Appendix 5.3

Geology and Soils

PRELIMINARY GEOTECHNICAL INVESTIGATION AND PERCOLATION TESTING

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

GEOCON
WEST, INC.
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

THE HOMESTEAD, LLC
NEWPORT BEACH, CALIFORNIA

April 19, 2019
PROJECT NO. T2857-22-01

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. T2857-22-01
April 19, 2019

The Homestead, LLC
280 Newport Center Drive, Suite 240
Newport Beach, California 92660

Attention: Mr. Grant Ross

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
AND PERCOLATION TESTING
THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

Dear Mr. Ross:

In accordance with your authorization of our Proposal IE-2226 dated August 24, 2018, Geocon West, Inc. (Geocon) herein submits the results of our preliminary geotechnical investigation and percolation testing for the proposed retail development and industrial business park at the northeast corner of Limonite Avenue and Archibald Avenue in Eastvale, California. The geotechnical investigation is being issued as preliminary as portions of the site are inaccessible due to the recent rain and active use of the site by livestock.

The accompanying report presents our findings, conclusions and recommendations pertaining to the geotechnical aspects of the proposed development. 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,

GEOCON WEST, INC.

CHET E. ROBINSON
No. 2890

LISA A. BATTIATO
No. 2316

Chet E. Robinson
GE 2890

Lisa A. Battiato
CEG 2316

CER:LAB:hd

Distribution: Addressee (email)

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

PRELIMINARY GEOTECHNICAL INVESTIGATION AND PERCOLATION TESTING

1. PURPOSE AND SCOPE

This report presents the results of our preliminary geotechnical investigation and percolation testing for the proposed industrial business park located west of Limonite Avenue and Archibald Avenue in Eastvale, California (see *Vicinity Map*, Figure 1). The purpose of the investigation was 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 as presently proposed.

The scope of our investigation included a review of available historic aerial photographs, 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 the drilling of six small-diameter geotechnical borings and four percolation test borings. Our initial proposed scope included additional borings. However, portions of the site were not accessible due to active use by the livestock, wet soils at the site from recent rains limited access to the drilling equipment, and storm water was ponded in the southwestern portion of the site. An update geotechnical investigation is planned once the site is clear of livestock and the site is accessible to the drilling equipment.

Appendix A presents a discussion of the field investigation, logs of the borings, and percolation test data. The approximate locations of the exploratory borings are presented on the *Geologic Map* (Figure 2). We performed laboratory tests on soil samples obtained from the exploratory borings to evaluate pertinent physical and chemical properties for engineering analysis. The results of the laboratory testing are presented in *Appendix B*.

2. SITE AND PROJECT DESCRIPTION

The site is currently being utilized as a dairy. Residences are in the eastern portion of the site, and the western portion of the site is an open field with a stormwater pond. The general site conditions are shown on Figure 3, *Aerial Photograph*. Based on our review of historic aerial photographs, the site has been utilized for agriculture since at least 1938 and was converted to a dairy in the 1980's or early 1990's (Continental; NETR, 2019).

The area totals approximately 50 acres and is located at latitude 33.9746 and longitude -117.5970. Site grades are relatively level with elevations ranging from approximately 633 feet above mean sea level (MSL) in the southwest corner to 647 feet above MSL in the northeast portion of the site. The property is bounded on the east by Archibald Avenue and on the south by an industrial development. A storm water channel is immediately north and west of the site.

Several stockpiles of soil were observed in the western portion of the site. Manure (organic rich soil) was present in the pen areas and within the western portion of the site. The manure was observed at the ground surface.

Grading plans were not available for our review at the time of this preliminary investigation. The *Conceptual Site Plan* by HP Architecture dated June 21, 2018 was utilized as the base for our *Geologic Map*, Figure 2. The plan indicates four industrial buildings will be constructed in the site, and an extension of Limonite will bisect the site with three buildings to the north and one building south of the roadway. The industrial developments will include associated utility, parking, driveway and flat work improvements. Storm water infiltration structures currently under consideration include one retention basin in the southwest corner, and one retention basin on the eastern portion of the site north of Limonite Avenue.

Based on the site and surrounding grades, we expect that rough grading will result in cuts and fills of up to 10 feet to level the site and fill in the pond. Due to the relatively level topography for the development, graded slopes are expected to be less than 10 feet high. Structural plans were not provided for the buildings; however, we have assumed that the industrial business park will consist of one- or two-story buildings using concrete tilt-up construction. The buildings will likely be supported by shallow foundations with concrete slab-on-grade floors.

Due to the preliminary nature of the design currently, wall and column loads were not available. We expect that column loads for the proposed structures will be up to 100 kips, and wall loads will be up to 10 kips per linear foot. Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary.

If project details differ significantly from those described, Geocon should be contacted for review and possible revision to this report.

3. GEOLOGIC SETTING

The site is located within an alluvial fan and flood plain within the southern part of the Chino Basin, that is part of the Corona-Chino Valley crustal block, a major structural low. This crustal block is bounded on the west by the Chino fault and the Chino and San Jose Hills, on the north by the Cucamonga fault zone and the San Gabriel Mountains, on the east by the Rialto-Colton fault, and on the south by the La Sierra and Pedley Hills. This structural low was filled with late Tertiary to early Quaternary non-marine sedimentary deposits derived from the San Gabriel Mountains, the Chino Hills, Puente Hills, and the San Bernardino Mountains via the Santa Ana River, and capped by a relatively thin layer of windblown sand. At depth, the basin consists of impermeable sedimentary and igneous rocks that are exposed at the surface in the surrounding mountains and hills.

Locally, the site is underlain by several hundred feet of young alluvial fan deposits from the San Gabriel Mountains and flood plain deposits from the Santa Ana River to the south, resulting in interlayered fine- and coarse-grained deposits of clays, silts, and sands. No faults are geologically mapped within or adjacent to the site.

4. GEOLOGIC MATERIALS

4.1 General

Based on our field investigation and published geologic maps of the area, the soils underlying the site consist of undocumented artificial fill and young alluvial fan deposits (Morton and Gray, 2002). Undocumented artificial fill was encountered to depths of 3 to 4 feet in the southern portion of the site and is likely present in other areas from the dairy improvements. The site soils are described in detail on the boring logs in *Appendix A*. The soil and geologic units encountered at the site are discussed below.

4.2 Undocumented artificial fill (afu)

Undocumented artificial fill was encountered within Borings B-1, B-5, P-1, and P-2 to depths of 3 to 4 feet within the southern portion of the site. The fill encountered is fine silty sand to silt which is brown to grey, moist and stiff/medium dense. Fill is likely present in other areas of the site that were not explored.

4.3 Young Alluvial Fan Deposits (Qyf_a)

Holocene alluvial fan deposits with interlayered fluvial flood plain deposits were encountered across the site to depths of 51.5 feet. These soils are collectively referred to as young alluvial fan deposits herein for simplicity. The alluvial fan units consist of silty to clayey sands which are moist and generally medium dense. The fluvial deposits are the fine-grained units of silt and clay which are moist to wet and soft to very stiff.

5. GROUNDWATER

Seepage or perched water was encountered within B-2 at 24½ feet below ground surface and in B-4 at a depth of 18¼ feet below the ground surface. Seepage or groundwater were not encountered in the other borings to depths of 30 to 50 feet below ground surface. At the time of our investigation, the stormwater pond in the western portion of the site had standing water. Perched water was not encountered in the borings near the pond but should be expected in the area and along the storm water channel.

Based on data from the California Department of Water Resources, groundwater was reported at depths of greater than 128 feet BGS at a well approximately 0.8 mi east-northeast of the site between 2011 and 2017. 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 therefore vary.

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). An active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a currently established State of California Alquist-Priolo Earthquake Fault Zone (CDC, 2018a) or a Riverside County Fault Hazard Zone for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is in the seismically active southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active southern California faults.

The closest active faults to the site are the Chino-Central Avenue fault, located approximately 5.4 miles to the southwest, and the Elsinore Glen Ivy fault, located 8.8 miles south of the site (CDC, 2018b). Faults within a 50-mile radius of the site are listed in Table 6.1.1. Historic earthquakes in southern California of magnitude 6.0 and greater, their magnitude, distance, and direction from the site are listed in Table 6.1.2.

TABLE 6.1.1
Active Faults within 50 Miles of the Site

Fault Name	Maximum Magnitude (Mw)	Geometry (Slip Character)	Slip Rate (mm/yr)	Information Source	Distance from Site (mi)	Direction from Site
Chino Fault	6.7	RL-R-O	1.0	a	4.3	WSW
Elsinore Fault (Glen Ivy North)	6.8	RL-SS	5.0	a	8.8	S
Whittier Fault	6.8	RL-R-O	2.5	a	9.1	SW
Red Hill (Etiwanda Ave)	n/a	n/a	n/a	b	12	NNE
Cucamonga Fault	6.9	R	5.0	a	13	N
San Jacinto Fault (San Bernardino)	6.8	RL-SS	5.0	a	17	NE
San Andreas (San Bernardino Mountains)	7.5	RL-SS	24	a	21	NE
San Jacinto (San Jacinto Valley)	6.9	RL-SS	12	a	24	E
Raymond	6.5	LL-R-O	1.5	a	27	NW
San Jacinto (Casa Loma)	6.9	RL-SS	12	a	28	ESE
Elsinore (Wildomar)	6.8	RL-SS	5.0	a	28	SE
Crafton Hills	n/a	n/a	n/a	b	28	ENE
Newport-Inglewood	7.1	RL-SS	1.0	a	31	SW
Beaumont Plain	n/a	n/a	n/a	b	34	E
North Frontal Thrust	7.2	R	1.0	a	35	NE
San Andreas (Mojave Section)	7.4	RL-SS	30.0	a	36	NNW
Verdugo	6.9	R	0.5	a	38	WNW
Llano	6.1	RO	1.0	a	38	NNW
San Gorgonio Pass	n/a	THRUST	n/a	b	39	E
Palos Verdes	7.3	RS-SS	3.0	a	39	SW
Hollywood	6.4	LL-R-O	1.0	a	40	WNW
Sierra Madre	7.2	R	2.0	a	42	NW
Coronado Bank	7.6	RL-SS	3.0	a	42	SW
San Jacinto (Anza)	7.2	RL-SS	12	a	44	SE
Sierra Madre (San Fernando Section)	6.7	R	2.0	a	45	NW
Redondo Canyon	n/a	n/a	n/a	b	48	WSW
Helendale	7.3	RL-SS	0.6	a	48	NE
Santa Monica	6.6	LL-R-O	1.0	a	48	W

Geometry: BT = blind thrust, LL = left lateral, N = normal, O = oblique, R = reverse, RL = right lateral, SS = strike slip.

Information Sources: a = Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, including Appendices A, B, and C, dated June; b = online Fault Activity Map of California website, maps.conservation.ca.gov/cgs/fam/, as of 1/2017.

n/a = data not available

TABLE 6.1.2
Historic Earthquake Events with Respect to the Site

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto	April 21, 1918	6.8	37	ESE
Loma Linda Area	July 22, 1923	6.3	20	E
Long Beach	March 10, 1933	6.4	33	SW
Buck Ridge	March 25, 1937	6.0	86	ESE
Imperial Valley	May 18, 1940	6.9	74	E
Desert Hot Springs	December 4, 1948	6.0	69	E
Arroyo Salada	March 19, 1954	6.4	101	ESE
Borrego Mountain	April 8, 1968	6.5	107	ESE
San Fernando	February 9, 1971	6.6	59	WNW
Joshua Tree	April 22, 1992	6.1	80	E
Landers	June 28, 1992	7.3	74	ENE
Big Bear	June 28, 1992	6.4	50	ENE
Northridge	January 17, 1994	6.7	62	WNW
Hector Mine	October 16, 1999	7.1	93	ENE

6.2 Liquefaction

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not.

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.

The site is within an area mapped as having very high liquefaction potential per Riverside County (RCIT, 2018).

As discussed in the Groundwater Section of this report, groundwater is expected in excess of 100 feet below the ground surface, however seepage or perched water was encountered in two of the borings at depths of 18¼ to 24½ feet. The depth of the perched groundwater was used in our liquefaction analysis.

We performed a liquefaction analysis of the soils underlying the site 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 a magnitude 6.7 earthquake, and the site-specific peak horizontal acceleration for the site. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance

Based on the medium dense to dense consistency of the granular alluvial soils and the relatively cohesive nature of the fine-grained alluvial deposits, the potential for liquefaction and seismic settlement at the site is negligible and not a design consideration. An analysis of the liquefaction potential and seismic induced settlement is included on Figures 4 and 5.

6.3 Expansive Soil

The soils encountered within the site consist of clays, silts, and sands. Laboratory testing results indicate samples of the near surface soils exhibit “very low” expansion potential (expansion index [EI] of 20 or less) with expansion index test results of 0 and 1.

6.4 Hydrocompression

Hydrocompression is the tendency of unsaturated soil structure to collapse upon wetting resulting in the overall settlement of the affected soil and overlying foundations or improvements supported thereon. Potentially compressible soils underlying the site are typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydrocompression of the soil exists. Alluvial soils obtained during our investigation were tested for hydrocompression and exhibited a collapse potential of 0.01 to 0.3 percent when loaded to the expected post-grading pressures.

6.5 Landslides

The site is not located near a hillside. Therefore, landslides are not a design consideration.

6.6 Rock Fall Hazards

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

6.7 Slope Stability

Graded slopes up to 10 feet in height and inclined as steep as 2:1 (horizontal:vertical) are expected at the site. In general, graded fill slopes constructed of on-site soils with gradients of 2:1 (horizontal to vertical) or flatter will possess factors of safety of 1.5 or greater. Geocon should be contacted for additional evaluation is steeper slopes or slopes greater than 10 feet in height are planned for the development.

6.8 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.*, 2003). The site is located approximately 31 miles from the nearest coastline; therefore, the negligible 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 or below reservoirs or other standing bodies of water. Therefore, seiche hazard is not a design consideration.

6.9 Organic Rich Soil

Samples of soil tested for organic content indicated that the subsurface site soils have between 1.0 and 3.6 percent organics by weight. Soils with a higher organic content are expected near the ground surface and in stockpiles at the site due to previous agricultural activities and where manure has been mixed with the soils.

7. SITE INFILTRATION

Percolation testing was performed in general accordance with the procedures in *Riverside County Flood Control and Water Conservation District LID BMP, Appendix A* (the Handbook) at locations and depths selected by the design team. The percolation test locations are depicted on the *Geologic Map* (see Figure 2). The percolation tests had to be modified due to the operations of the dairy at the time of our investigation. The sandy soil criteria test had to be halted in percolation tests P-3 and P-4 because of livestock within the test area. The tests were resumed later that day once the dairy was able to relocate the animal.

Approximately 2 inches of gravel was placed at the bottom of each percolation test hole and a 3-inch diameter perforated PVC pipe in silt filter sock was placed atop the gravel. The test locations were pre-saturated prior to testing. Percolation data sheets are presented in *Appendix A* of this report. Calculations to convert the percolation test rate to infiltration test rates are presented in Table 7 below. The Handbook requires a factor of safety of 3 be applied to the values below based on the test method used.

TABLE 7
INFILTRATION TEST RATES FOR PERCOLATION AREAS

Parameter	P-1	P-2	P-3	P-6
Depth (inches)	96.0	96.0	97.0	96.0
Test Type	Modified	Modified	Modified	Modified
Change in head over time: ΔH (inches)	1.7	5.0	1.0	1.1
Average head: Havg (in)	23.2	21.5	24.4	23.6
Time Interval (minutes): Δt (minutes)	30	10	10	10
Radius of test hole: r (inches)	4	4	4	4
Tested Infiltration Rate: It (inches/hour)	0.27	2.58	0.44	0.51

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 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 design and construction.

8.1.2 Potential geologic hazards at the site include seismic shaking, compressibility of the near surface soils, and organic soils. Based on our investigation and available geologic information, active, potentially active, or inactive faults are not present underlying or trending toward the site.

8.1.3 The undocumented artificial fill and the upper portion of the alluvial soil are not considered suitable for the support of additional compacted fill or settlement-sensitive improvements. Remedial grading of the surficial soil will be required as discussed herein. The existing site soils, except as indicated below, are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed.

8.1.4 The manure impacted soils at the site are not suitable for use as compacted fill. The manure impacted soils should be removed from the site as part of the clearing and grubbing operations.

8.1.5 Following removal of the manure impacted soils, our laboratory tests indicate that the subsurface soils to be used as fill contain organic contents between 1.0 and 3.6 percent. Processing of the site soils during grading is expected to result in an average organic content of approximately 2 to 3 percent. Additional compactive effort should be planned during grading to mitigate the settlement potential due to the organic content of the soils at the site.

8.1.6 Perched water was encountered in B-2 at 24½ and in B-4 at 18¼ feet during our subsurface investigation. It is likely that this condition is a result of water from recent precipitation flowing along a silty sand unit and perched on the underlying silt layer. However, based on the variability of the soil types encountered, it is possible that perched water will be encountered at shallower depths, depending on after agricultural irrigation, precipitation during rainy seasons, infiltration from the stormwater pond, and other factors.

8.1.7 Moisture contents are expected to vary based on the season and amount of precipitation. Special handling of the soil should be anticipated, particularly if grading occurs during the rainy season, as drying back of the existing materials should be anticipated prior to their use as fill.

8.1.8 Given the loose or soft consistency of the site soils and high moisture contents, relatively soft soils should be expected in the site excavation walls and bottoms, and subgrade stabilization will be required within site excavations during grading or installation of utilities.

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

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

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

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

8.1.13 Recommended grading specifications are provided in Appendix C.

8.2 Soil Characteristics

8.2.1 The near surface site soils encountered in the field investigation are "non-expansive" (Expansion Index [EI] of 20 or less) as defined by 2016 California Building Code (CBC) Section 1803.5.3 with a "Very Low" expansion potential. Table 8.2.1 presents soil classifications based on the EI.

TABLE 8.2.1
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

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

8.2.2 Based on the material classifications and laboratory testing, the near surface site soils are generally expected to possess a low expansion potential (EI of 50 or less). Medium to highly expansive soils should not be placed within 4 feet of the proposed foundations, flatwork or paving improvements. Additional testing for expansion potential should be performed once final grades are achieved.

8.2.3 Laboratory testing was performed on samples of the site soils to evaluate the percentage of water-soluble sulfate content. Results indicate that the on-site materials at the locations tested possess a sulfate content of 0.000 to 0.044% (less than 10 to 440 parts per million [ppm]) equating to an exposure class of S0 to concrete structures as defined by 2016 CBC Section 1904.3 and ACI 318. Table 8.2.3 presents a summary of concrete requirements set forth by 2016 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 8.2.3
REQUIREMENTS FOR CONCRETE
EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
S0	**0.00-0.10**	**--**	**--**	**2,500**
S1	0.10-0.20	II	0.50	4,000
S2	0.20-2.00	V	0.45	4,500
S3	> 2.00	V+ Pozzolan or Slag	0.45	4,500

8.2.4 Laboratory testing indicates the site soils have a minimum electrical resistivity of 320 to 26,000 ohm-cm, possess 40 to 180 parts per million chloride, less than 10 to 440 ppm sulfate, and have a pH of 7.24 and 8.32. As shown in Table 8.2.4 below, the site would be classified as "corrosive" to buried improvements, in accordance with the Caltrans Corrosion Guidelines (Caltrans, 2018) based on the electrical resistivity. Additionally, the site historic and current use is for agriculture and as a dairy farm. Several areas of the site were not accessible for our exploration. The client should anticipate corrosive soils will be encountered on the site, particularly where manure or drainage from the cow pens are present.

TABLE 8.2.4
CALTRANS CORROSION GUIDELINES

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

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

8.3 Grading

8.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in *Appendix C* and the Grading Ordinances of the City of Eastvale.

8.3.2 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.

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

8.3.4 Undocumented fill and alluvium within a 1:1 (h:v) projection of the limits of grading should be removed to expose competent alluvium with a relative compaction of at least 85 percent (ASTM D1557). Removals in the existing fill and alluvium should be expected on the order of 6 to 8 feet below existing grades. The removals should also extend at least 3 feet below the bottom of the planned foundations. Areas of loose, dry, or compressible soils will require deeper excavation and processing prior to fill placement. Removals in pavement and walkway areas should extend at least 3 feet beneath the pavement or flatwork subgrade elevation. The actual depth of removal should be evaluated by the engineering geologist during grading operations. Where over excavation and compaction is to be conducted, the excavations should be extended laterally a minimum distance of 5 feet beyond the building footprint or for a distance equal to the depth of removal, whichever is greater. Patios and building appurtenances should be considered a part of the building footprint when determining the limits of lateral excavation. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned as necessary, and properly compacted to 95 percent of the maximum dry density as determined by ASTM 1557.

8.3.5 Geocon should observe the removal bottoms to check the competence at the bottom of the removal. Deeper excavations may be required if dry, loose, or soft materials are present at the base of the removals. Excavation bottoms require written approval by a Geocon representative.

8.3.6 The site soils are expected to have an average organic content on the average of 2 to 3 percent by weight when placed as compacted fill. Riverside County guidelines (RTLMA, 2000) indicate that fill soils should have an organic content of 1 percent or less. To mitigate the potential settlement from the organic soils at the site, fill should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557.

8.3.7 The fill placed within 4 feet of proposed foundations should possess a “low” expansion potential (EI of 50 or less).

8.3.8 If perched groundwater, wet, or saturated materials are encountered during remedial grading, extensive drying and mixing with dryer soil will be required. The excavated materials should then be moisture conditioned as necessary to 0 to 2 percent above optimum moisture content prior to placement as compacted fill.

8.3.9 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 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content as determined by ASTM D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

8.3.10 Where relatively loose, soft, or wet soils are encountered in the site excavations, subgrade stabilization will be required prior to placing fill or installing utilities. Where required, subgrade stabilization can be achieved by over excavating the loose or soft materials and replacing with compacted fill, placing 3-inch diameter rock in the soft bottom and working it into soil until it is stabilized, or placing gravel wrapped in filter fabric at the bottom of the excavation. Where used, gravel should consist of 12 to 18 inches of washed angular ¾ inch gravel atop a filter fabric (Mirafi 500X or equivalent) on the excavation bottom. The filter fabric should be placed in a manner so that the gravel does not have direct contact with the soil. Once the gravel is placed and vibrated to a relatively dense state, a top layer of filter fabric should be placed to cover the gravel. Recommendations for stabilizing excavation bottoms should be based on an evaluation in the field by Geocon at the time of construction.

8.3.11 Import fill (if necessary) should consist of granular materials with a "low" expansion potential (EI of 50 or less), non-corrosive, generally free of deleterious material and contain no rock fragments larger than 6 inches. Geocon should be notified of the import soil source and should perform laboratory testing of import soil to evaluate its suitability prior to its arrival at the site for use as fill material.

8.4 Earthwork Grading Factors

8.4.1 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 95 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 with similar site soils, the shrinkage of the undocumented fill and upper portion of the alluvium is expected to be 5 to 10 percent when compacted to at least 95 percent of the laboratory maximum dry density. This estimate is for preliminary quantity estimates only. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate variations.

8.5 Utility Trench Backfill

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

8.5.2 Utility trench backfill should be placed in layers no thicker than will allow for adequate bonding and compaction. Utility backfill should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density and moisture conditioned at 0 to 2 percent above optimum moisture content as determined by ASTM D 1557. Backfill materials placed below the recommended moisture content may require additional moisture conditioning prior to placing additional fill.

8.6 Seismic Design Criteria

8.6.1 We used the computer program *U.S. Seismic Design Maps*, provided by the California Office of Statewide Health Planning and Development (OSHPD) to evaluate the seismic design criteria. Table 8.6.1 summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements as currently proposed should be designed using a Site Class D in accordance with ASCE 7-10 Section 20.3.1. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10 using blow count data presented on the boring logs in *Appendix A*. The values presented in Table 8.6.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 8.6.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.500g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.600g	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}	1.500g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.900g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.000g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.600g	Section 1613.3.4 (Eqn 16-40)

8.6.2 Table 8.6.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10.

TABLE 8.6.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.500	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.500g	Section 11.8.3 (Eqn 11.8-1)

8.6.3 The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2016 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spread, and seismic settlements. We understand the intent of the building code is to maintain “Life Safety” during an MCE event.

8.6.4 Deaggregation of the MCE peak ground acceleration was performed using the online *Unified Hazard Tool* (USGS, 2018b) provided by the USGS. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.7 magnitude event occurring at a hypocentral distance of 17.3 kilometers from the site

8.6.5 Conformance to the criteria in the above tables 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.

8.7 Foundation and Concrete Slabs-On-Grade

8.7.1 The foundation recommendations presented herein are for the proposed buildings subsequent to the recommended grading assuming that the buildings are founded in soils with a low expansion potential. If soils with a medium or high expansion potential are placed within 4 feet of finish grade, then Geocon should be contacted for additional recommendations. We understand that future buildings will be supported on conventional shallow foundations with a concrete slab-on-grade deriving support in newly placed engineered fill.

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

8.7.3 From a geotechnical engineering standpoint, concrete slabs-on-grade for the structure should be at least 5 inches thick and be reinforced with No. 4 steel reinforcing bars placed 18 inches on center in both directions. The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slab for supporting equipment and storage loads. A thicker concrete slab may be required for heavier loading conditions. To reduce the effects of differential settlement on the foundation system, thickened slabs and/or an increase in steel reinforcement can provide a benefit to reduce concrete cracking

8.7.4 Following remedial grading, foundations for the buildings may be designed for an allowable soil bearing pressure of 3,000 pounds per square foot (psf) (dead plus live load). The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

8.7.5 The maximum expected static settlement for the planned structures supported on conventional foundation systems with the above allowable bearing pressures and deriving support in engineered fill is estimated to be 1 inch and to occur below the heaviest loaded structural element.

8.7.6 Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a horizontal distance of 40 feet.

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

8.7.8 Steel reinforcement for continuous footings should consist of at least four No. 4 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.

8.7.9 Foundations near slopes should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

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

8.7.11 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). The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.

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

8.7.13 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.

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

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

8.8 Exterior Concrete Flatwork

8.8.1 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 95 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 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.

8.8.2 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade or differential settlement. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork.

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

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

8.9 Conventional Retaining Walls

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

8.9.2 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 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 60 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 50 or less. For walls where backfill materials do not conform to the criteria herein, Geocon should be consulted for additional recommendations.

8.9.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, the walls should be designed for a soil pressure equivalent to the pressure exerted by a fluid density of 55 pcf.

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

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

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

8.9.7 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140N (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. Alternatively, a drainage panel, such as a Miradrain 6000 or equivalent, can be placed along the back of the wall. Typical retaining wall drainage details are shown on Figure 7. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted backfill (EI of 50 or less) with no hydrostatic forces or imposed surcharge load. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations.

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

8.10 Lateral Design

8.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. A passive pressure exerted by an equivalent fluid weight of 325 pounds per cubic foot (pcf) with a maximum earth pressure of 3,250 psf should be used for the design of footings or shear keys poured neat against newly compacted fill. 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.

8.10.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between newly compacted fill soil and concrete of 0.35 should be used for design. When combining passive pressure and friction for lateral resistance, the passive component should be reduced by one-third.

8.11 Pavement Design

8.11.1 The final pavement design should be based on R-value testing of soils at the subgrade following grading at the site. Streets should be designed in accordance with the city of Eastvale and Riverside County *Standard Drawings and Specifications* when final Traffic Indices and R-Value test results of subgrade soil are completed. Roadway classifications and traffic indices are based on Riverside County Standard No. 114. The civil engineer should evaluate the final traffic index for the pavements. Laboratory testing indicated that the site soils possess an R-value of 55 and 70. For the preliminary analysis, we have used an R-value of 50, the maximum allowed by Caltrans. Preliminary flexible pavement sections are presented in Table 8.11.1. We have included TI's for areas within the industrial business park as well as Limonite Avenue. Geocon should be contacted for additional recommendations if other traffic loading is appropriate for the roadways.

TABLE 8.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Road Classification/Use	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Aggregate Base (inches)
Local Street/Parking Areas/Light Duty Vehicles	5.5	50	3.5	6.0
Enhanced Local Street/Moderate Traffic	6.5	50	4.0	6.0
Industrial Collector/Heavy Truck Areas	8.0	50	5.0	6.0
Major Highway	9.0	50	5.5	6.5
Arterial Highway	9.5	50	6.0	7.0

8.11.2 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 at 0 to 2 percent above optimum moisture content beneath pavement sections.

8.11.3 The crushed aggregated base and asphalt concrete materials should conform to Section 200-2.2 and Section 203-6, respectively, of the Greenbook and the County of Riverside Standard Drawings and Specifications. Base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content. Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

8.11.4 Where prefabricated concrete pavers (80 mm thick) will be used in site roadways and parking areas, it is acceptable from a geotechnical standpoint to construct the pavers over 1 inch of sand underlain by a properly prepared subgrade and aggregate base per the following table. The aggregate base should be compacted to at least 95 percent relative compaction as evaluated by ASTM D 1557 (latest edition). Pavers should be constructed in accordance with the manufacture's guidelines.

TABLE 8.11.4
PAVER DESIGN SECTIONS

Road Classification/Use	Estimated Traffic Index (TI)	Prefabricated Concrete Paver (inches)	Class 2 Aggregate Base (inches)
Local Street/Parking Areas/Light Duty Vehicles	5.5	3⅛	6

8.11.5 Where concrete pavers will be placed in pedestrian walkway areas, and will not be subject to vehicle loading, the inclusion of a 4-inch thick layer of base over properly compacted subgrade underlying the pavers is acceptable from a geotechnical standpoint.

8.11.6 Where different pavement sections are to be constructed adjacent to each other, we recommend that consideration be given to the use of deepened base sections to maintain a uniform base thickness and avoid stepped cuts for placement of base material. This condition is expected to occur across the transition across the areas of asphalt paving and prefabricated pavers.

8.11.7 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 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 8.11.7.

TABLE 8.11.7
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	150 pci
Modulus of rupture for concrete, M_R	550 psi
Traffic Category, TC	A, B, C and D
Average daily truck traffic, ADTT	10, 25, 100 and 700

8.11.8 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.11.8.

TABLE 8.11.8
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Car Parking Areas and Access Lanes (TC=A)	5.0
Entrance and Service Lanes (TC=B)	6.0
Moderate Truck Traffic (TC=C)	6.5
Bus Stops and Heavy Truck Traffic (TC=D)	7.5

8.11.9 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch). Base material will not be required beneath concrete improvements.

8.11.10 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.

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

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

8.12 Temporary Excavations

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

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

8.12.3 Where sufficient space is available, temporary unsurcharged embankments may be sloped back at a uniform 1.5:1 (h:v) slope gradient or flatter. A uniform slope does not have a vertical portion.

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

8.13 Site Drainage and Moisture Protection

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

8.13.2 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.

8.13.3 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 be used to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes. 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.

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

8.14 Plan Review

8.14.1 Geocon should be provided the opportunity to review the grading and structural/foundation plans for the project prior to final submittal, to verify that the plans have been prepared in substantial conformance with the recommendations of this report. Additional analyses may be required after review of the project 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 their representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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

1. American Concrete Institute, 2011, *Building Code Requirements for Structural Concrete*, Report by ACI Committee 318.

2. American Concrete Institute, 2008, *Guide for Design and Construction of Concrete Parking Lots*, Report by ACI Committee 330.

3. ASCE 7-10, 2011, *Minimum Design Loads for Buildings and Other Structures,* Secon Printing, April 6

4. Boore, D. M. and G. M Atkinson, *Ground-Motion Prediction for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods Between 0.01 and 10.0 S*, Earthquake Spectra, Volume 24, Issue 1, pages 99-138, February 2008.

5. Bryant, William A., and Earl W. Hart, 2007, *Fault-Rupture Hazard Zones in California: Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps*; California Geological Survey Special Publication 42, 54 p.

6. California Building Standards Commission, 2016, *California Building Code (CBC)*, California Code of Regulations Title 24, Part 2.

7. California Department of Water Resources (CDWR), 2018, *Water Data Library* website, www.water.ca.gov/waterdatalibrary/index.cfm, accessed 02/08/2018.

8. California Department of Conservation (CDC), 2018a, *CGS Information Warehouse: Regulatory Maps* website for Alquist-Priolo Earthquake Fault Zone Maps, http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps, accessed 02/08/2018.

9. California Department of Conservation (CDC), 2018b, *Fault Activity Map of California (2010)* website, http://maps.conservation.ca.gov/cgs/fam/, accessed 02/09/2018.

10. California Department of Transportation (Caltrans), Division of Engineering Services, Materials Engineering and Testing Services, *Corrosion Guidelines, Version 2.0,* dated November 2012.

11. California Geological Survey (CGS), *Earthquake Shaking Potential for California*, from USGS/CGS Seismic Hazards Model, CSSC No. 03-02, 2003.

12. California Department of Transportation (Caltrans), 2015, *Corrosion Guidelines, Version 2.1,* Division of Engineering Services, Materials Engineering and Testing Services, dated January.

13. Campbell, K. W. and Y. Bozorgnia, *NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s*, Preprint of version submitted for publication in the NGA Special Volume of Earthquake Spectra, Volume 24, Issue 1, pages 139-171, February 2008.

LIST OF REFERENCES (CONTINUED)

14. Chiou, Brian S. J. and Robert R. Youngs, *An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra*, preprint for article to be published in NGA Special Edition for Earthquake Spectra, Spring 2008.

15. Continental Aerial Photograph: 05/21/49 AXL-7F-136, -137, -138, -139; 11/6/59 AXL-18W-156, -157; 5/8/65 65200 27, 98; 5/15/67 AXM-4HH-172, -173; 3/3/71 71000 21, 22; 10/24/75 75000 58, 59; 2/15/77 RIV 2-1, 2-02; 1/80 SBD 16-13, -15; 1/8/87 F 159, 160, 161; 6/12/90 C83-12-27; 7/11/95 C114-30-73, -74; 10/16/97 C119-30-169, -170, -171; 2/2/99 C-131-30-116, -117.

16. Google, Inc., 2017, *Google Earth* online mapping software, accessed 02/08/2018.

17. HPA Architecture, 2018*, Conceptual Site Plan, Limonite Ave. and Archibald Ave. – Phase II, Scheme 4, Phase 2*, dated June 21.

18. Legg, M. R., J. C. Borrero, and C. E. Synolakis, 2003*, Evaluation of Tsunami Risk to Southern California Coastal Cities*, 2002 NEHRP Professional Fellowship Report, dated January.

19. Morton, Douglas M., and C.H. Gray, Jr., 2002, *Geologic Map of the Corona North 7.5' Quadrangle, Riverside and San Bernardino Counties, California*, version 1.0; U.S. Geological Survey Open-File Report 02-22; scale 1:24,000.

20. NETRonline, 2019, *Historic Aerials* photograph database, www.historicaerials.com, accessed April.

21. OSHPD, 2018, Seismic Design Maps, https://seismicmaps.org, accessed April 2, 2019.

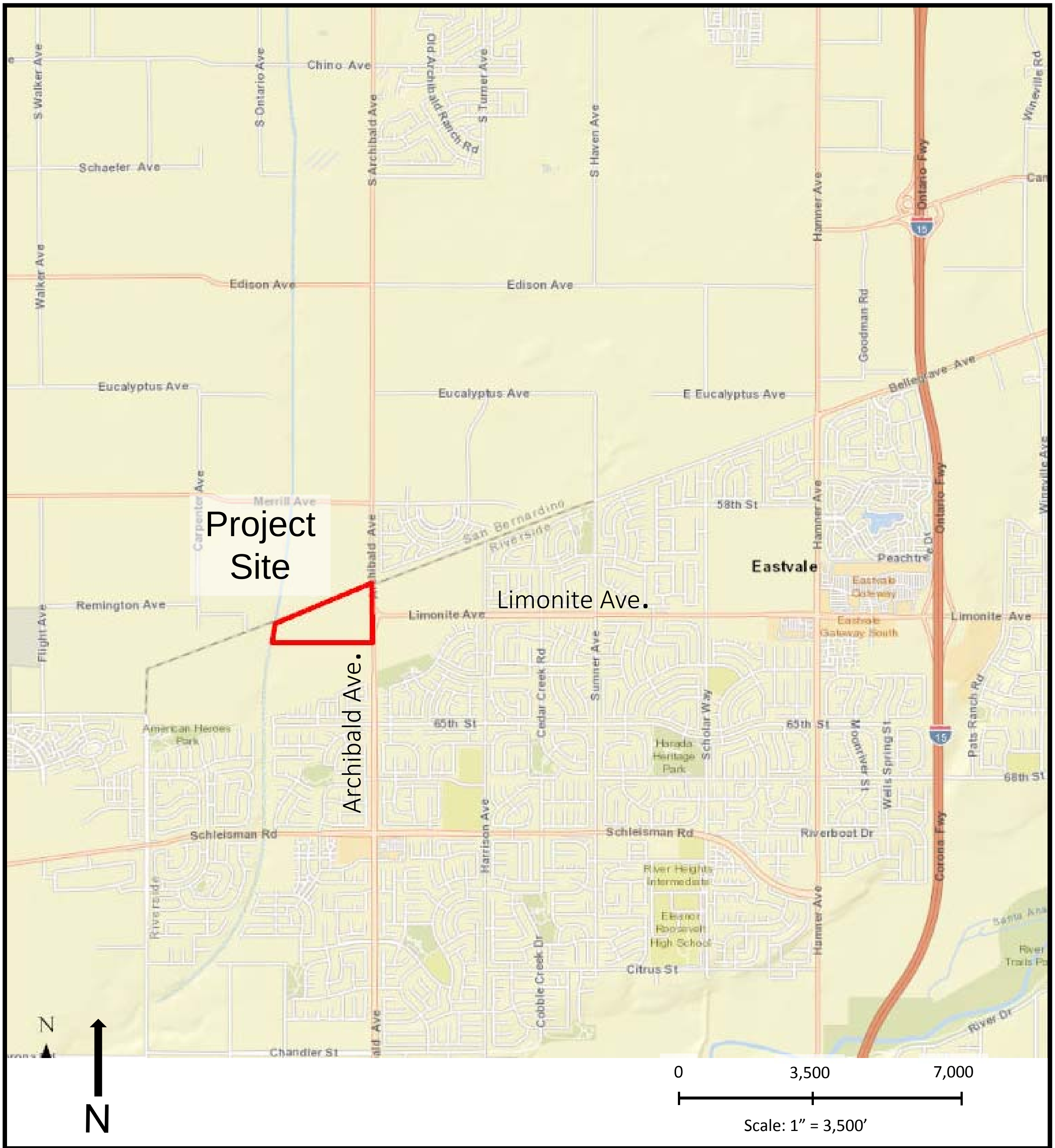
22. Public Works Standards, Inc., 2015, *Standard Specifications for Public Works Construction "Greenbook,"* Published by BNi Building News.

23. Riverside County Flood Control, 2011, *Design Handbook for Low Impact Development Best Management Practices*, 162 p., dated September.

24. Riverside County Transportation and Land Management Agency, 2000, *Technical Guidelines for Review of Geotechnical and Geologic Reports*, 2000 Edition.

25. Riverside County RCIT, 2019, *Map My County* online GIS database, http://mmc.rivcoit.org/MMC_Public/Viewer.html?Viewer=MMC_Public, accessed April.

26. U.S. Geological Survey (USGS), *Deaggregation of Seismic Hazard for PGA and 2 Periods of Spectral Acceleration*, 2002, USGS Website: www.earthquake.usgs.gov/research/hazmaps.



VICINITY MAP

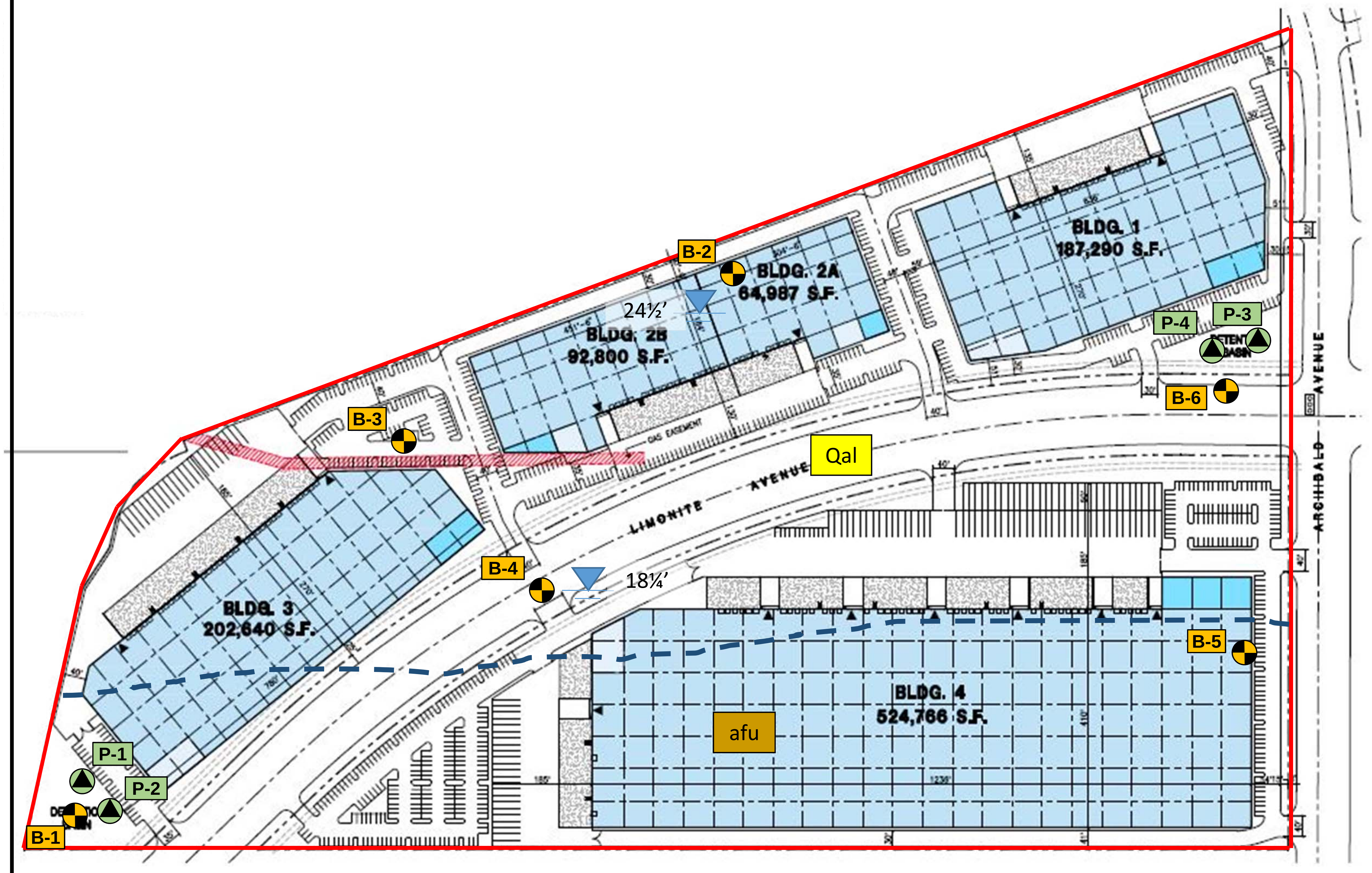
GEOCON
WEST, INC.

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

DF

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019 | PROJECT NO. T2857-22-01 | FIG. 1



GEOCON LEGEND

Locations are approximate

——	……. PROJECT BOUNDARY
- - -	……. INFERRED GEOLOGIC CONTACT
B-5 ●	……. BORING LOCATION
P-4 ▲	……. PERCOLATION TEST LOCATION
afu	……. UNDOCUMENTED FILL
Qyfa	……. ALLUVIUM
▼	……. PERCHED GROUNDWATER OR SEEPAGE DEPTH

N

0 200 400

SCALE 1" = 200'

SOURCE: HPA ARCHITECTURE, *CONCEPTUAL SITE PLAN, SCHEME 4 – PHASE 2*, DATED JUNE 21, 2018.

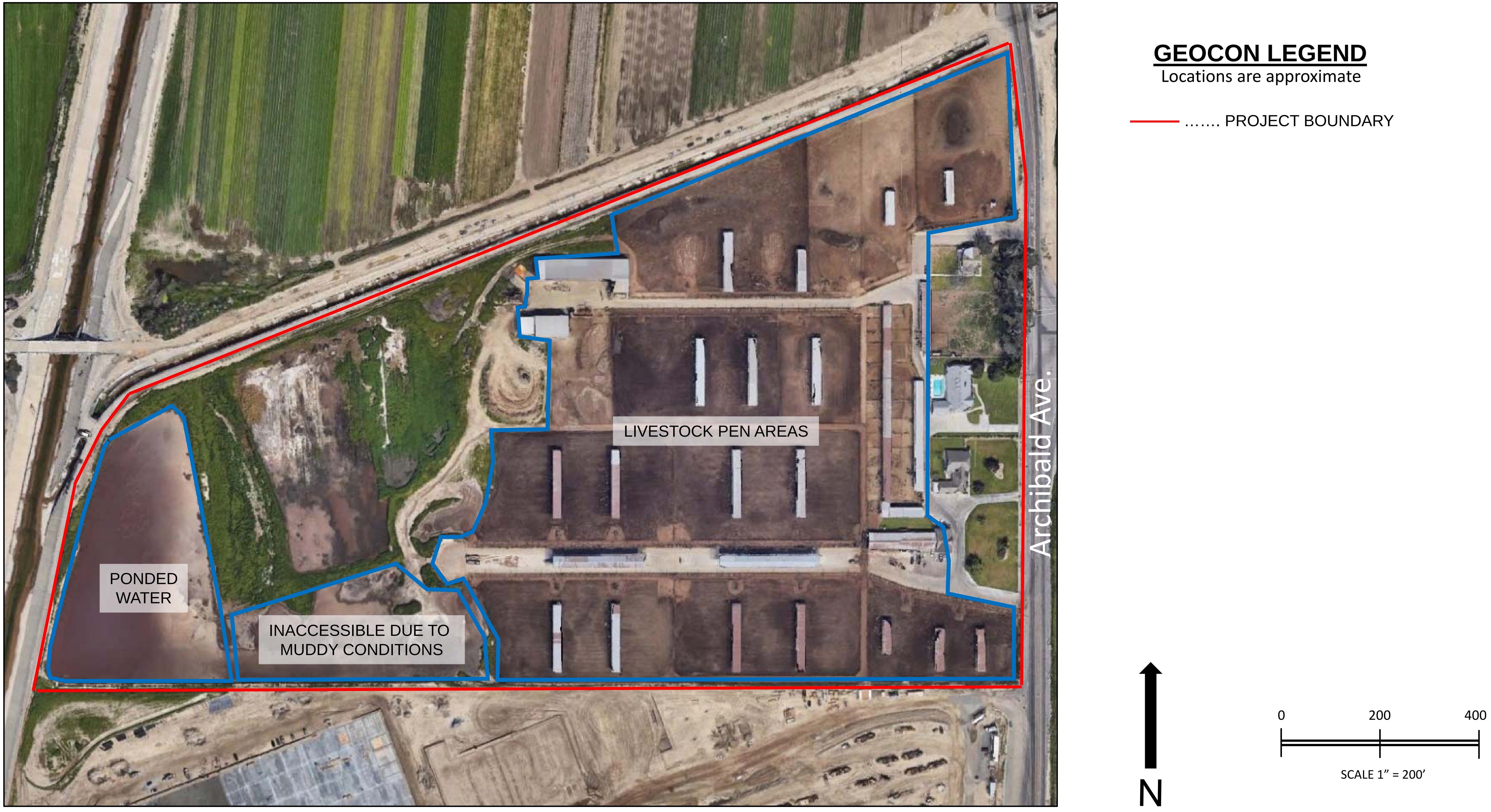
GEOCON
WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF

GEOLOGIC MAP

THE HOMESTEAD BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019 | PROJECT NO. T2857-22-01 | FIG. 2



SOURCE: GOOGLE EARTH PRO IMAGERY, DATED MARCH, 2017.

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

DF

AERIAL PHOTOGRAPH

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019	PROJECT NO. T2857-22-01	FIG. 3

GEOCON

Project : The Homestead
File No. : T2857-22-01
Boring : B-2 & B-5

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.73
PGA_M (g):	0.500
Calculated Mag.Wtg.Factor:	0.762
Historic High Groundwater:	18.0
Groundwater @ Exploration:	24.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'$_o$	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1	13	127	0.032	0.032	82	39	0.325	--	0.00	0.00
2	13	127	0.095	0.095	82	39	0.325	--	0.00	0.00
3	13	127	0.159	0.159	82	39	0.325	--	0.00	0.00
4	13	127	0.222	0.222	82	39	0.325	--	0.00	0.00
5	13	127	0.286	0.286	82	38	0.325	--	0.00	0.00
6	24	127	0.349	0.349	106	60	0.325	--	0.00	0.00
7	24	127	0.413	0.413	106	55	0.325	--	0.00	0.00
8	18	127	0.476	0.476	86	41	0.325	--	0.00	0.00
9	18	127	0.540	0.540	86	39	0.325	--	0.00	0.00
10	18	127	0.603	0.603	86	38	0.325	--	0.00	0.00
11	11	130	0.668	0.668	64	25	0.325	--	0.00	0.00
12	11	130	0.733	0.733	64	24	0.325	--	0.00	0.00
13	11	130	0.798	0.798	64	23	0.325	--	0.00	0.00
14	11	130	0.863	0.863	64	23	0.325	--	0.00	0.00
15	36	130	0.928	0.928	106	60	0.325	--	0.00	0.00
16	36	130	0.993	0.993	106	58	0.325	--	0.00	0.00
17	36	130	1.058	1.058	106	57	0.325	--	0.00	0.00
18	36	130	1.123	1.107	106	55	0.330	Non-Liq.	0.00	0.00
19	22	130	1.188	1.141	78	38	0.338	Non-Liq.	0.00	0.00
20	22	130	1.253	1.175	78	37	0.347	Non-Liq.	0.00	0.00
21	22	130	1.318	1.208	78	36	0.354	Non-Liq.	0.00	0.00
22	22	130	1.383	1.242	78	35	0.362	Non-Liq.	0.00	0.00
23	22	130	1.448	1.276	78	35	0.369	Non-Liq.	0.00	0.00
24	22	130	1.513	1.310	78	34	0.375	Non-Liq.	0.00	0.00
25	22	130	1.578	1.344	78	34	0.382	Non-Liq.	0.00	0.00
26	38	130	1.643	1.377	99	53	0.388	Non-Liq.	0.00	0.00
27	38	130	1.708	1.411	99	52	0.393	Non-Liq.	0.00	0.00
28	38	130	1.773	1.445	99	52	0.399	Non-Liq.	0.00	0.00
29	38	130	1.838	1.479	99	51	0.404	Non-Liq.	0.00	0.00
30	19	130	1.903	1.513	68	33	0.409	Non-Liq.	0.00	0.00
31	19	130	1.968	1.546	68	32	0.414	Non-Liq.	0.00	0.00
32	19	130	2.033	1.580	68	32	0.418	Non-Liq.	0.00	0.00
33	19	130	2.098	1.614	68	32	0.422	Non-Liq.	0.00	0.00
34	19	130	2.163	1.648	68	32	0.427	Non-Liq.	0.00	0.00
35	19	130	2.228	1.682	68	31	0.431	Non-Liq.	0.00	0.00
36	19	130	2.293	1.715	65	31	0.434	Non-Liq.	0.00	0.00
37	19	130	2.358	1.749	65	31	0.438	Non-Liq.	0.00	0.00
38	19	130	2.423	1.783	65	31	0.442	Non-Liq.	0.00	0.00
39	19	130	2.488	1.817	65	31	0.445	Non-Liq.	0.00	0.00
40	19	130	2.553	1.851	65	30	0.448	Non-Liq.	0.00	0.00
41	45	130	2.618	1.884	98	62	0.451	Non-Liq.	0.00	0.00
42	45	130	2.683	1.918	98	62	0.455	Non-Liq.	0.00	0.00
43	45	130	2.748	1.952	98	61	0.457	Non-Liq.	0.00	0.00
44	45	130	2.813	1.986	98	61	0.460	Non-Liq.	0.00	0.00
45	45	130	2.878	2.020	98	60	0.463	Non-Liq.	0.00	0.00
46	24	130	2.943	2.053	69	35	0.466	Non-Liq.	0.00	0.00
47	24	130	3.008	2.087	69	35	0.468	Non-Liq.	0.00	0.00
48	24	130	3.073	2.121	69	35	0.471	Non-Liq.	0.00	0.00
49	66	130	3.138	2.155	112	82	0.473	Non-Liq.	0.00	0.00
50	66	130	3.203	2.189	112	81	0.476	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

GEOCON

Project : The Homestead
File No. : T2857-22-01
Boring : B-2 & B-5

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9
EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS
MAXIMUM CONSIDERED EARTHQUAKE

MCE EARTHQUAKE INFORMATION:

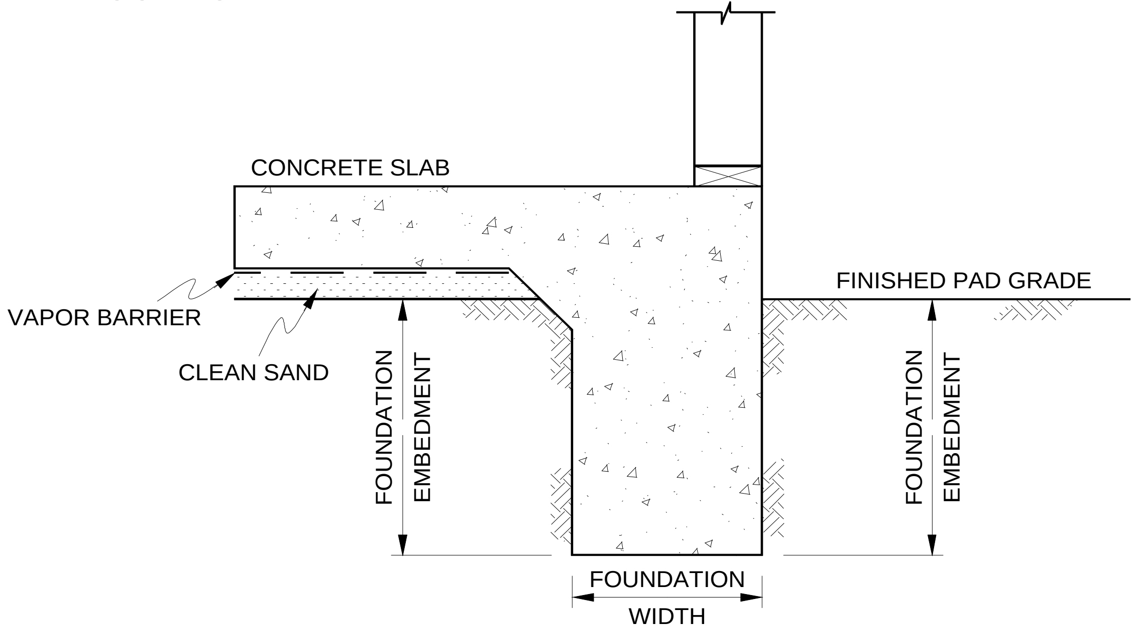
Earthquake Magnitude:	6.73
Peak Horiz. Acceleration (g):	0.500

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[yeff]*[Geff] [Gmax]	yeff Shear Strain	[yeff]*100%	Volumetric Strain M7.5 [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	127.0	0.03	0.02	0.010	13	1.25	82.3	2.0	39.5	1.0	221.977	4.60E-05	6.00E-05	0.006	2.65E-03	8.8404	2.09E-03	0.001
2.0	1.0	1.5	127.0	0.10	0.06	0.031	13	1.25	82.3	2.0	39.5	1.0	384.476	7.82E-05	1.40E-04	0.014	6.19E-03	8.8404	4.88E-03	0.001
3.0	1.0	2.5	127.0	0.16	0.11	0.052	13	1.25	82.3	2.0	39.5	1.0	496.357	9.89E-05	1.60E-04	0.016	7.08E-03	8.8404	5.58E-03	0.001
4.0	1.0	3.5	127.0	0.22	0.15	0.072	13	1.25	82.3	2.0	39.5	1.0	587.297	1.15E-04	1.70E-04	0.017	7.52E-03	8.8404	5.93E-03	0.001
5.0	1.0	4.5	127.0	0.29	0.19	0.093	13	1.25	82.3	1.9	38.0	1.0	657.447	1.29E-04	1.70E-04	0.017	7.87E-03	8.8404	6.21E-03	0.001
6.0	1.0	5.5	127.0	0.35	0.23	0.113	24	1.25	106.2	1.7	59.5	1.0	844.230	1.21E-04	1.50E-04	0.015	4.05E-03	8.8404	3.19E-03	0.001
7.0	1.0	6.5	127.0	0.41	0.28	0.134	24	1.25	106.2	1.6	55.2	1.0	895.107	1.32E-04	1.50E-04	0.015	4.43E-03	8.8404	3.50E-03	0.001
8.0	1.0	7.5	127.0	0.48	0.32	0.154	18	1.25	85.8	1.5	41.5	1.0	874.055	1.53E-04	1.50E-04	0.015	6.25E-03	8.8404	4.93E-03	0.001
9.0	1.0	8.5	127.0	0.54	0.36	0.174	18	1.25	85.8	1.4	39.4	1.0	914.592	1.63E-04	1.50E-04	0.015	6.65E-03	8.8404	5.24E-03	0.001
10.0	1.0	9.5	127.0	0.60	0.40	0.194	18	1.25	85.8	1.3	37.6	1.0	952.341	1.72E-04	1.50E-04	0.015	7.02E-03	8.8404	5.54E-03	0.001
11.0	1.0	10.5	130.0	0.67	0.45	0.215	11	1.25	64.4	1.3	24.8	1.0	871.716	2.04E-04	4.50E-04	0.045	3.48E-02	8.8404	2.74E-02	0.007
12.0	1.0	11.5	130.0	0.73	0.49	0.235	11	1.25	64.4	1.2	24.0	0.9	903.146	2.12E-04	4.50E-04	0.045	3.62E-02	8.8404	2.85E-02	0.007
13.0	1.0	12.5	130.0	0.80	0.53	0.255	11	1.25	64.4	1.1	23.3	0.9	933.015	2.19E-04	3.70E-04	0.037	3.08E-02	8.8404	2.43E-02	0.006
14.0	1.0	13.5	130.0	0.86	0.58	0.275	11	1.25	64.4	1.1	22.7	0.9	961.523	2.26E-04	3.70E-04	0.037	3.19E-02	8.8404	2.51E-02	0.006
15.0	1.0	14.5	130.0	0.93	0.62	0.295	36	1.25	106.2	1.1	60.1	0.9	1380.551	1.67E-04	1.60E-04	0.016	4.27E-03	8.8404	3.37E-03	0.001
16.0	1.0	15.5	130.0	0.99	0.66	0.315	36	1.25	106.2	1.0	58.4	0.9	1413.961	1.71E-04	1.60E-04	0.016	4.43E-03	8.8404	3.49E-03	0.001
17.0	1.0	16.5	130.0	1.06	0.71	0.335	36	1.25	106.2	1.0	56.8	0.9	1446.036	1.75E-04	1.60E-04	0.016	4.58E-03	8.8404	3.61E-03	0.001
18.0	1.0	17.5	130.0	1.12	0.75	0.354	36	1.25	106.2	1.0	55.3	0.9	1476.909	1.79E-04	1.60E-04	0.016	4.72E-03	8.8404	3.72E-03	0.001
19.0	1.0	18.5	130.0	1.19	0.80	0.373	22	1.25	78.2	0.9	37.7	0.9	1336.683	2.06E-04	3.70E-04	0.037	1.73E-02	8.8404	1.36E-02	0.000
20.0	1.0	19.5	130.0	1.25	0.84	0.392	22	1.25	78.2	0.9	36.8	0.9	1362.533	2.09E-04	3.70E-04	0.037	1.78E-02	8.8404	1.40E-02	0.000
21.0	1.0	20.5	130.0	1.32	0.88	0.411	22	1.25	78.2	0.9	36.1	0.9	1387.578	2.13E-04	3.70E-04	0.037	1.82E-02	8.8404	1.44E-02	0.000
22.0	1.0	21.5	130.0	1.38	0.93	0.429	22	1.25	78.2	0.9	35.3	0.9	1411.882	2.16E-04	3.70E-04	0.037	1.87E-02	8.8404	1.47E-02	0.000
23.0	1.0	22.5	130.0	1.45	0.97	0.448	22	1.25	78.2	0.8	34.7	0.9	1435.501	2.18E-04	3.70E-04	0.037	1.91E-02	8.8404	1.51E-02	0.000
24.0	1.0	23.5	130.0	1.51	1.01	0.466	22	1.25	78.2	0.8	34.2	0.9	1460.577	2.21E-04	3.00E-04	0.030	1.58E-02	8.8404	1.24E-02	0.000
25.0	1.0	24.5	130.0	1.58	1.06	0.484	22	1.25	78.2	0.8	33.9	0.9	1487.038	2.22E-04	3.00E-04	0.030	1.59E-02	8.8404	1.26E-02	0.000
26.0	1.0	25.5	130.0	1.64	1.10	0.501	38	1.25	99.0	0.8	52.8	0.9	1759.034	1.93E-04	1.30E-04	0.013	4.06E-03	8.8404	3.20E-03	0.000
27.0	1.0	26.5	130.0	1.71	1.14	0.519	38	1.25	99.0	0.8	52.2	0.9	1787.323	1.94E-04	1.30E-04	0.013	4.11E-03	8.8404	3.24E-03	0.000
28.0	1.0	27.5	130.0	1.77	1.19	0.536	38	1.25	99.0	0.8	51.7	0.9	1814.883	1.95E-04	1.30E-04	0.013	4.16E-03	8.8404	3.28E-03	0.000
29.0	1.0	28.5	130.0	1.84	1.23	0.552	38	1.25	99.0	0.8	51.2	0.9	1841.758	1.96E-04	1.30E-04	0.013	4.21E-03	8.8404	3.32E-03	0.000
30.0	1.0	29.5	130.0	1.90	1.27	0.569	19	1.25	67.6	0.8	32.7	0.9	1613.605	2.28E-04	3.00E-04	0.030	1.66E-02	8.8404	1.31E-02	0.000
31.0	1.0	30.5	130.0	1.97	1.32	0.585	19	1.25	67.6	0.8	32.4	0.9	1636.716	2.29E-04	3.00E-04	0.030	1.68E-02	8.8404	1.32E-02	0.000
32.0	1.0	31.5	130.0	2.03	1.36	0.601	19	1.25	67.6	0.8	32.2	0.9	1659.344	2.30E-04	3.00E-04	0.030	1.69E-02	8.8404	1.34E-02	0.000
33.0	1.0	32.5	130.0	2.10	1.41	0.617	19	1.25	67.6	0.8	32.0	0.9	1681.513	2.31E-04	3.00E-04	0.030	1.71E-02	8.8404	1.35E-02	0.000
34.0	1.0	33.5	130.0	2.16	1.45	0.632	19	1.25	67.6	0.8	31.7	0.8	1703.246	2.32E-04	3.00E-04	0.030	1.72E-02	8.8404	1.36E-02	0.000
35.0	1.0	34.5	130.0	2.23	1.49	0.647	19	1.25	67.6	0.7	31.5	0.8	1724.565	2.32E-04	3.00E-04	0.030	1.74E-02	8.8404	1.37E-02	0.000
36.0	1.0	35.5	130.0	2.29	1.54	0.662	19	1.25	65.4	0.7	31.3	0.8	1745.489	2.32E-04	3.00E-04	0.030	1.75E-02	8.8404	1.38E-02	0.000
37.0	1.0	36.5	130.0	2.36	1.58	0.676	19	1.25	65.4	0.7	31.1	0.8	1766.038	2.33E-04	3.00E-04	0.030	1.77E-02	8.8404	1.39E-02	0.000
38.0	1.0	37.5	130.0	2.42	1.62	0.690	19	1.25	65.4	0.7	30.9	0.8	1786.227	2.33E-04	3.00E-04	0.030	1.78E-02	8.8404	1.41E-02	0.000
39.0	1.0	38.5	130.0	2.49	1.67	0.704	19	1.25	65.4	0.7	30.7	0.8	1806.072	2.33E-04	3.00E-04	0.030	1.80E-02	8.8404	1.42E-02	0.000
40.0	1.0	39.5	130.0	2.55	1.71	0.718	19	1.25	65.4	0.7	30.5	0.8	1825.589	2.34E-04	3.00E-04	0.030	1.81E-02	8.8404	1.43E-02	0.000
41.0	1.0	40.5	130.0	2.62	1.75	0.731	45	1.25	97.7	0.7	62.1	0.8	2344.243	1.84E-04	1.30E-04	0.013	3.34E-03	8.8404	2.63E-03	0.000
42.0	1.0	41.5	130.0	2.68	1.80	0.744	45	1.25	97.7	0.7	61.7	0.8	2367.501	1.84E-04	1.30E-04	0.013	3.37E-03	8.8404	2.65E-03	0.000
43.0	1.0	42.5	130.0	2.75	1.84	0.756	45	1.25	97.7	0.7	61.2	0.8	2390.383	1.84E-04	1.30E-04	0.013	3.39E-03	8.8404	2.68E-03	0.000
44.0	1.0	43.5	130.0	2.81	1.88	0.769	45	1.25	97.7	0.7	60.8	0.8	2412.906	1.84E-04	1.30E-04	0.013	3.42E-03	8.8404	2.70E-03	0.000
45.0	1.0	44.5	130.0	2.88	1.93	0.781	45	1.25	97.7	0.7	60.4	0.8	2435.082	1.84E-04	1.30E-04	0.013	3.45E-03	8.8404	2.72E-03	0.000
46.0	1.0	45.5	130.0	2.94	1.97	0.792	24	1.25	69.3	0.7	35.3	0.8	2058.095	2.19E-04	3.00E-04	0.030	1.52E-02	8.8404	1.20E-02	0.000
47.0	1.0	46.5	130.0	3.01	2.02	0.804	24	1.25	69.3	0.7	35.0	0.8	2076.549	2.19E-04	1.00E-02	1.000	5.10E-01	8.8404	4.02E-01	0.000
48.0	1.0	47.5	130.0	3.07	2.06	0.815	24	1.25	69.3	0.7	34.8	0.8	2094.746	2.19E-04	1.00E-02	1.000	5.14E-01	8.8404	4.05E-01	0.000
49.0	1.0	48.5	130.0	3.14	2.10	0.826	66	1.25	111.8	0.7	81.8	0.8	2813.887	1.64E-04	1.00E-02	1.000	1.84E-01	8.8404	1.45E-01	0.000
50.0	1.0	49.5	130.0	3.20	2.15	0.836	66	1.25	111.8	0.7	81.3	0.8	2836.589	1.64E-04	1.00E-02	1.000	1.86E-01	8.8404	1.46E-01	0.000

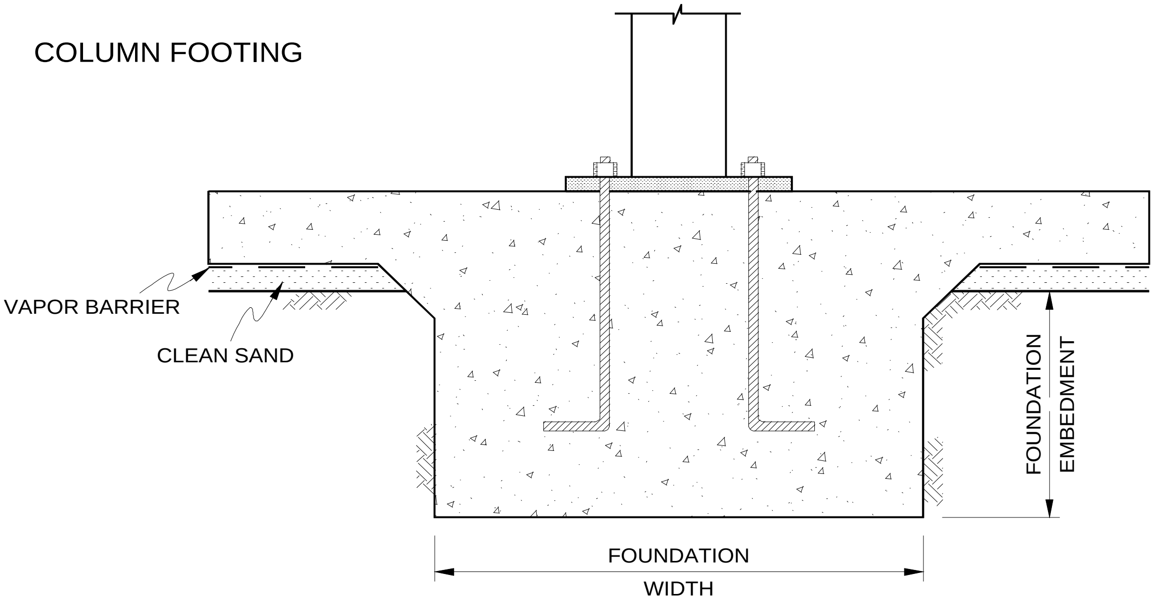
TOTAL SETTLEMENT = **0.04**

Figure 5

WALL FOOTING



COLUMN FOOTING



NOTE: SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

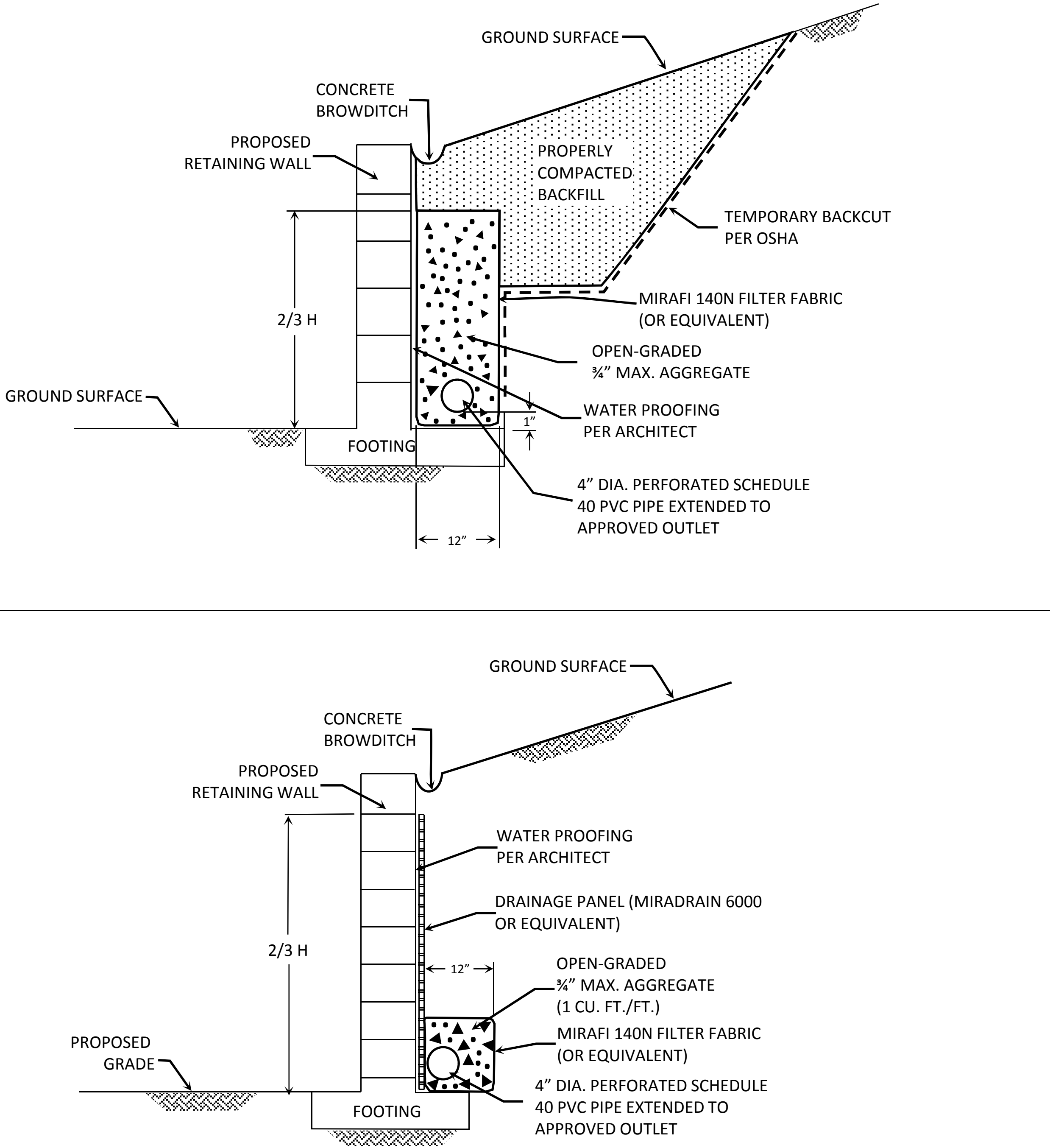
GEOCON
W E S T, I N C.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF		

WALL / COLUMN FOOTING DETAIL

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL 2019	PROJECT NO. T2857-22-01	FIG. 6



NOTES:

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

CONCRETE BROW DITCH RECOMMENDED FOR SLOPE HEIGHTS
GREATER THAN 6 FEET

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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

DF		

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019	PROJECT NO. T2857-22-01	FIG. 7

APPENDIX A

APPENDIX A

EXPLORATORY EXCAVATIONS

Geocon performed the field investigation on March 14 and 15, 2019. Our subsurface exploration consisted of drilling six small-diameter borings and four percolation tests at the site. The borings were drilled to depths of 30 to 51 feet below the existing ground surface and the percolation tests were advanced to depths of approximately 8 feet below the existing ground surface using a track-mounted, hollow stem auger drill rig. We collected bulk and relatively undisturbed samples from the borings by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound hammer falling 30 inches or a slide hammer. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch inside diameter brass sampler rings to facilitate removal and testing. Standard Penetration Test samples were also collected by driving a 2-inch diameter sampler 18 inches into the soil to retrieve small bulk samples. Relatively undisturbed samples and bulk samples of disturbed soils were transported to our laboratory for testing.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A-1 through A-10. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the excavations are indicated the *Geologic Map*, Figure 2.

Percolation testing was performed on March 28, 2018 in general accordance with *Riverside County Flood Control and Water Conservation District, LID BMP Manual, Appendix A.* The testing procedures were modified because of site constraints from the active dairy. The percolation tests were run in accordance with *Section 2.3., Shallow Percolation Test*. The percolation test data is presented on Figures A-11 through A-14.

PROJECT NO. T2857-22-01

BORING B-1

ELEV. (MSL.) **637** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-1@0-5'			SM	**UNDOCUMENTED FILL (Afu)** Silty SAND, medium dense, moist, grayish brown; fine sand			
2								
	B-1@2.5'					38	106.7	19.2
4				SM	**YOUNG ALLUVIAL FAN DEPOSITS(Qyfa)** Silty SAND, dense, moist, light olive brown; fine sand			
	B-1@5'					92	107.6	10.1
6								
8	B-1@7.5'					71	113.7	15.4
10	B-1@10'			ML	Sandy SILT, very stiff, moist, dark brown	42	106.8	20.3
12								
14				SM	Silty SAND, dense, damp, orangish brown; fine to medium sand			
	B-1@15'					68	118.9	12.2
16								
18								
				CL	CLAY with sand, very stiff, moist, brown with orange and gray			
20	B-1@20'					75	113.9	17.7
22								
24								
	B-1@25'				-becomes fine to medium sand; iron oxide staining	61		
26								
28								

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

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

BORING B-1

ELEV. (MSL.) **637** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	BULK	DR/SPT	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B-1@30'		■			SM	Silty SAND, very dense, moist, orangish brown; fine to medium sand; trace gravel	91/10'	121.2	11.0
32										
34						SC	Clayey SAND, very dense, olive brown; fine sand			
	B-1@35'		■					50/6'		
36										
38										
40	B-1@40'		■					50/6'		

Total depth 41'
No Groundwater encountered
Penetration resisance for 140-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 03/14/19

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

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◣ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

PROJECT NO. T2857-22-01

BORING B-2

ELEV. (MSL.) **642** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	BULK	DR/SPT	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					ML	**TOPSOIL** SILT with sand, soft, wet, dark brown with orange			
2	B-2@2.5'		■		SM	**YOUNG ALLUVIAL FAN DEPOSITS(Qyfa)** Silty SAND, medium dense, wet, olive brown; fine sand	20	111.1	14.5
6	B-2@5-7' B-2@5'	⊠	■		ML	Sandy SILT, stiff, wet, olive brown; fine to medium sand	36	103.8	22.0
8	B-2@7.5'		■				27		
10	B-2@10'		■		ML	SILT, stiff, wet, brown; trace fine sand	17	111.5	16.7
16	B-2@15'		■			-becomes very stiff	54	92.8	31.1
18					SM	Silty SAND, medium dense, moist, brown with gray and dark brown; micaceous			
20	B-2@20'		■				33	105.6	20.9
26	B-2@25'		◧		SM	Silty SAND, dense, saturated, grayish brown; medium sand	38		
28				▽					
					CL	Sandy CLAY, stiff, saturated, dark brown			

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

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. T2857-22-01

BORING B-2

ELEV. (MSL.) **642** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B-2@30'			CL	Sandy CLAY, stiff, saturated, dark brown	29		
32								
34								
	B-2@35'					19		
36				ML	Sandy SILT, very stiff, saturated, bluish gray; fine sand			
38								
40	B-2@40'			SM	Silty SAND, very dense, saturated, brown; fine to medium sand	50/4"		

Total depth 40' 3"
Seepage or perched water encountered at 24' 5" during drilling
Penetration resisance for 140-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 03/14/19

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

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. T2857-22-01

BORING B-3

ELEV. (MSL.) **638** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**TOPSOIL** Organic, loose, wet			
	B-3@1-5'	SM	**YOUNG ALLUVIAL FAN DEPOSITS(Qyfa)** Silty SAND, dense, moist, grayish brown	4		
	B-3@2.5'			58	112.7	16.0
	B-3@5'	CL-ML	Sandy silty CLAY, stiff, wet, grayish brown	12	101.5	23.7
	B-3@7.5'			15	96.8	27.5
10	B-3@10'	SC	Clayey SAND, dense, wet, grayish brown	44	123.1	14.6
		CL	CLAY with sand, stiff, moist, brown			
	B-3@15'			25	101.4	24.7
20	B-3@20'			41	116.1	17.7
	B-3@25'			46		

Figure A-3,
Log of Boring B-3, Page 1 of 2

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ☒ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

BORING B-3

ELEV. (MSL.) **638** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	BULK	DR/SPT	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B-3@30'		■			SM	Silty SAND, very dense, moist, brown	50/4"	113.7	14.6
							Total depth 30' 10" No Groundwater encountered Penetration resisance for 140-lb hammer falling 30" by auto-hammer Backfilled with cuttings 03/14/19			

Figure A-3,
Log of Boring B-3, Page 2 of 2

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. T2857-22-01

BORING B-4

ELEV. (MSL.) 636 DATE COMPLETED 03/14/2019

EQUIPMENT HOLLOW STEM AUGER BY: C. Robinson

DEPTH IN FEET	SAMPLE NO.	BULK	DR/SPT	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0						SM	**FILL (Disturbed soils)** Silty SAND, loose, moist, brown			
	B-4@1-5'	☒				SM	**YOUNG ALLUVIAL FAN DEPOSITS(Qyfa)** Silty SAND, medium dense, moist, brown			
2	B-4@2.5'	☒	■					28		
		☒	■			ML	SILT with sand, stiff, moist, olive		94.5	23
4		☒								
	B-4@5'		■					63	100.7	26.9
6						CL	CLAY, very stiff, moist, greenish brown			
8	B-4@7.5'		■					18	102.4	22.4
							-becomes light brown			
10	B-4@10'		■			SM	Silty SAND, medium dense, wet, brown with light brown	21		
12										
14										
	B-4@15'		■					20		
16										
18					▽					
						ML	SILT with sand, very stiff, saturated, brown			
20	B-4@20'		■					28	100.3	28.0
22										
24										
	B-4@25'		■			SC	Clayey SAND, medium dense, saturated, brown	20		
26										
28										

Figure A-4,
Log of Boring B-4, Page 1 of 2

SAMPLE SYMBOLS			
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. T2857-22-01

BORING B-4

ELEV. (MSL.) **636** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	BULK	DR/SPT	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B-4@30'		■	▨		SC	-becomes very dense	50/6"	117.5	14.2

Total depth 31'
Seepage or perched water encountered at 18'3" during drilling
Penetration resisance for 140-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 03/14/19

Figure A-4,
Log of Boring B-4, Page 2 of 2

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◩ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. T2857-22-01

BORING B-5

ELEV. (MSL.) **643** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**TURF AND TOPSOIL** Loose, wet, dark brown			
	B-5@1-5'			SM	**UNDOCUMENTED FILL (afu)** Silty SAND, medium dense, moist, dark brown; fine sand			
2	B-5@2.5'					77		
				SM	**YOUNG ALLUVIAL FAN DEPOSITS(Qyfa)** Silty SAND, dense, moist, dark brown; fine sand			
4								
	B-5@5'					20	100.2	9.0
6				ML	SILT with sand, very stiff, moist, olive brown			
8	B-5@7.5'				-becomes wet; trace roots	28		
10	B-5@10'			CL	CLAY, stiff, wet, olive brown	25	90.7	31.4
12								
14								
	B-5@15'				-becomes greenish brown	35	104.6	22.8
16								
18				CL	CLAY with sand,and gravel size cemented pieces, stiff, wet, light olive brown			
20	B-5@20'					13	90.5	34.6
22								
24				ML	Clayey SILT with sand, stiff, wet, dark brown; fine sand			
	B-5@25'					19		
26								
28								

Figure A-5,
Log of Boring B-5, Page 1 of 2

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. T2857-22-01

BORING B-5

ELEV. (MSL.) **643** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	BULK	DR/SPT	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B-5@30'		■			ML	-becomes fine to medium sand; very stiff	34		
32										
						SC	Clayey SAND, very dense, wet, olive brown; fine to medium sand			
34										
	B-5@35'		■					50/6"	121.4	14.0
36										
38										
						ML	SILT with sand, hard, moist, olive brown; iron oxide staining; fine sand			
40										
	B-5@40'		■					67		
42										
44										
	B-5@45'		■				-becomes very stiff	36	86.8	39.3
46										
48										
						SM	Silty SAND, very dense, moist, olive brown; iron oxide staining			
50										
	B-5@50'		■					50/6"		

Total depth 51'
No Groundwater encountered
Penetration resisance for 140-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 03/14/19

Figure A-5,
Log of Boring B-5, Page 2 of 2

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. T2857-22-01

BORING B-6

ELEV. (MSL.) **645** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

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	**TOPSOIL** Silty SAND, loose, damp, light brown			
2	B-6@1-5'			SM	**YOUNG ALLUVIAL FAN DEPOSITS(Qyfa)** Silty SAND, medium dense, damp, light brown; fine sand			
	B-6@2.5'					23	102.7	5.9
4					-becomes olive brown			
	B-6@5'					22	106.9	13.3
6								
8	B-6@7.5'			ML	SILT with sand, very stiff, moist, olive brown; fine sand	43	95.1	27.6
10	B-6@10'					43		
12								
				SM	Silty SAND, medium dense, damp, olive brown; fine to medium sand			
14								
	B-6@15'					44	110.7	11.6
16				CL	Sandy CLAY, stiff, wet, light brown; fine to medium sand			
18								
				SM	Silty SAND, very dense, damp, olive brown; fine sand			
20	B-6@20'					50/5"		

Total depth 20' 5"
No Groundwater encountered
Penetration resisance for 140-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 03/14/19

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

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. T2857-22-01

BORING P-1

ELEV. (MSL.) **637** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	BULK	DR/SPT	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0						ML	**UNDOCUMENTED FILL (afu)** SILT with sand, stiff, moist, medium brown; fine sand			
2										
4						SM	**YOUNG ALLUVIAL FAN DEPOSITS(Qyfa)** Silty SAND, medium dense, moist, light brown; fine sand			
6	P-1@6-8'	☒								
8							Total depth 8' No Groundwater encountered Penetration resisance for 140-lb hammer falling 30" by auto-hammer Backfilled with cuttings 03/14/19			

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

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. T2857-22-01

BORING P-2

ELEV. (MSL.) **637** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	BULK	DR/SPT	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0						ML	**UNDOCUMENTED FILL (afu)** SILT with sand, stiff, moist, medium brown; fine sand			
2										
4						SM	**YOUNG ALLUVIAL FAN DEPOSITS(Qyfa)** Silty SAND, medium dense, moist, light brown			
6	P-2@6-8'	☒								
8							Total depth 8' No Groundwater encountered Penetration resisance for 140-lb hammer falling 30" by auto-hammer Backfilled with cuttings 03/14/19			

Figure A-8,
Log of Boring P-2, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. T2857-22-01

BORING P-3

ELEV. (MSL.) **645** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	BULK	DR/SPT	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0						SM	**UNDOCUMENTED FILL (afu)** Silty SAND, loose, damp, light brown			
						SM	**YOUNG ALLUVIAL FAN DEPOSITS(Qyfa)** Silty SAND, medium dense, damp, light brown			
2										
4										
6	P-3@6-8'	☒								
8										

Total depth 8' 2"
No Groundwater encountered
Penetration resisance for 140-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 03/14/19

Figure A-9,
Log of Boring P-3, Page 1 of 1

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. T2857-22-01

BORING P-4

ELEV. (MSL.) **645** DATE COMPLETED **03/14/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **C. Robinson**

DEPTH IN FEET	SAMPLE NO.	BULK	DR/SPT	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0						SM	**UNDOCUMENTED FILL (afu)** Silty SAND, loose, damp, light brown			
						SM	**YOUNG ALLUVIAL FAN DEPOSITS(Qyfa)** Silty SAND, medium dense, damp, light brown			
2										
4										
6	P-4@6-8'	☒								
8										

Total depth 8' 2"
No Groundwater encountered
Penetration resisance for 140-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 03/14/19

Figure A-10,
Log of Boring P-4, Page 1 of 1

T2857-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PERCOLATION TEST REPORT

Project Name:	The Homestead	**Project No.:**	T2857-22-01
Test Hole No.:	P-1	**Date Excavated:**	3/14/2019
Length of Test Pipe:	96.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	3/14/2019
Depth of Test Hole:	96.0 inches	**Perc Test Date:**	3/28/2019
Check for Sandy Soil Criteria Tested by:	SP	**Percolation Tested by:**	CER

Water level measured from BOTTOM of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	8:15 AM 8:40 AM	25	25	23.4	19.2	4.2	6.0
2	8:40 AM 9:05 AM	25	50	19.2	16.3	2.9	8.7

Soil Criteria: Normal

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	9:05 AM 9:35 AM	30	30	24.0	21.8	2.2	13.9
2	9:35 AM 10:05 AM	30	60	23.9	22.2	1.7	17.9
3	10:05 AM 10:35 AM	30	90	23.9	22.0	1.9	15.6
4	10:35 AM 11:05 AM	30	120	24.0	22.2	1.8	16.7
5	11:05 AM 11:35 AM	30	150	24.0	22.2	1.8	16.7
6	11:35 AM 12:05 PM	30	180	24.0	22.3	1.7	17.9
7	12:05 PM 12:35 PM	30	210	24.0	22.3	1.7	17.9

Infiltration Rate (in/hr):	0.27
Radius of test hole (in):	4
Average Head (in):	23.2

Figure A-11

PERCOLATION TEST REPORT

Project Name:	The Homestead	**Project No.:**	T2857-22-01
Test Hole No.:	P-2	**Date Excavated:**	3/14/2019
Length of Test Pipe:	96.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	3/14/2019
Depth of Test Hole:	96.0 inches	**Perc Test Date:**	3/28/2019
Check for Sandy Soil Criteria Tested by:	SP	**Percolation Tested by:**	CER

Water level measured from BOTTOM of hole

Sandy Soil Criteria Test

Trial No.	**Time**	**Time Interval (min)**	**Total Elapsed Time (min)**	**Initial Water Level (in)**	**Final Water Level (in)**	**Δ in Water Level (inches)**	**Percolation Rate (min/inch)**
1	8:15 AM 8:40 AM	25	25	25.2	13.1	12.1	2.1
2	8:40 AM 9:05 AM	25	50	13.1	4.8	8.3	3.0

Soil Criteria: Normal

Percolation Test

Reading No.	**Time**	**Time Interval (min)**	**Total Elapsed Time (min)**	**Initial Water Level (in)**	**Final Water Level (in)**	**Δ in Water Level (inches)**	**Percolation Rate (min/inch)**
1	9:15 AM 9:45 AM	30	30	25.2	8.3	16.9	1.8
2	9:45 AM 10:15 AM	30	60	24.1	8.6	15.5	1.9
3	10:15 AM 10:35 AM	20	80	24.2	11.2	13.1	1.5
4	10:35 AM 10:45 AM	10	90	24.0	19.8	4.2	2.4
5	10:45 AM 10:55 AM	10	100	24.0	19.0	5.0	2.0
6	10:55 AM 11:05 AM	10	110	23.8	19.0	4.8	2.1
7	11:05 AM 11:15 AM	10	120	24.1	19.1	5.0	2.0
8	11:15 AM 11:25 AM	10	130	24.1	19.0	5.2	1.9
9	11:25 AM 11:35 AM	10	140	24.0	19.0	5.0	2.0

Infiltration Rate (in/hr):	2.58
Radius of test hole (in):	4
Average Head (in):	21.5

Figure A-12

PERCOLATION TEST REPORT							
Project Name:		The Homestead			**Project No.:**		T2857-22-01
Test Hole No.:		P-3			**Date Excavated:**		3/14/2019
Length of Test Pipe:			109.0	inches	**Soil Classification:**		SM
Height of Pipe above Ground:			12.0	inches	**Presoak Date:**		3/14/2019
Depth of Test Hole:			97.0	inches	**Perc Test Date:**		3/28/2019
Check for Sandy Soil Criteria Tested by:				SP	**Percolation Tested by:**		CER
Water level measured from BOTTOM of hole							
Sandy Soil Criteria Test							
Trial No.	**Time**	**Time Interval (min)**	**Total Elapsed Time (min)**	**Initial Water Level (in)**	**Final Water Level (in)**	**Δ in Water Level (inches)**	**Percolation Rate (min/inch)**
1	8:50 AM			35.9			
2		Not measured due to livestock in test area					
			Soil Criteria: Normal				
			Percolation Test				
Reading No.	**Time**	**Time Interval (min)**	**Total Elapsed Time (min)**	**Initial Water Level (in)**	**Final Water Level (in)**	**Δ in Water Level (inches)**	**Percolation Rate (min/inch)**
1	1:00 PM 1:10 PM	10	10	24.5	23.8	0.7	13.9
2	1:10 PM 1:20 PM	10	20	24.9	23.8	1.1	9.3
3	1:20 PM 1:30 PM	10	30	25.0	23.8	1.2	8.3
4	1:30 PM 1:40 PM	10	40	25.0	23.9	1.1	9.3
5	1:40 PM 1:50 PM	10	50	24.9	23.4	1.4	6.9
6	1:50 PM 2:00 PM	10	60	24.9	23.7	1.2	8.3
7	2:00 PM 2:10 PM	10	70	25.0	23.8	1.2	8.3
8	2:10 PM 2:20 PM	10	80	24.9	23.9	1.0	10.4
9	2:20 PM 2:30 PM	10	90	24.9	23.7	1.2	8.3
10	2:30 PM 2:40 PM	10	100	25.0	23.9	1.1	9.3
11	2:40 PM 2:50 PM	10	110	25.0	24.0	1.0	10.4
12	2:50 PM 3:00 PM	10	120	24.9	23.9	1.0	10.4
Infiltration Rate (in/hr):			0.44				
Radius of test hole (in):			4				**Figure A-13**
Average Head (in):			24.4				

PERCOLATION TEST REPORT

Project Name:	The Homestead	**Project No.:**	T2857-22-01
Test Hole No.:	P-4	**Date Excavated:**	3/14/2019
Length of Test Pipe:	96.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	3/14/2019
Depth of Test Hole:	96.0 inches	**Perc Test Date:**	3/28/2019
Check for Sandy Soil Criteria Tested by:	SP	**Percolation Tested by:**	CER

Water level measured from BOTTOM of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	∆ in Water Level (inches)	Percolation Rate (min/inch)
1	8:55 AM			22.8			
2		Not measured due to livestock in test area					

Soil Criteria: Normal

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	∆ in Water Level (inches)	Percolation Rate (min/inch)
1	1:05 PM 1:15 PM	10	10	24.2	22.6	1.7	6.0
2	1:15 PM 1:25 PM	10	20	24.0	22.4	1.6	6.4
3	1:25 PM 1:35 PM	10	30	24.2	22.7	1.6	6.4
4	1:35 PM 1:45 PM	10	40	24.2	22.8	1.4	6.9
5	1:45 PM 1:55 PM	10	50	24.0	22.7	1.3	7.6
6	1:55 PM 2:05 PM	10	60	24.2	23.0	1.2	8.3
7	2:05 PM 2:15 PM	10	70	24.4	23.2	1.2	8.3
8	2:15 PM 2:25 PM	10	80	24.4	23.2	1.2	8.3
9	2:25 PM 2:35 PM	10	90	24.2	23.2	1.1	9.3
10	2:35 PM 2:45 PM	10	100	24.4	23.3	1.1	9.3
11	2:45 PM 2:55 PM	10	110	24.2	23.0	1.2	8.3
12	2:55 PM 3:05 PM	10	120	24.1	23.0	1.1	9.3

Infiltration Rate (in/hr):	0.51
Radius of test hole (in):	4
Average Head (in):	23.6

Figure A-14

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with current, generally accepted test methods of ASTM International (ASTM) or other suggested procedures. We analyzed selected soil samples for in-situ density and moisture content, maximum dry density and optimum moisture content, expansion index, corrosivity, grain size distribution, R-Value, plasticity, organic content, consolidation characteristics, and direct shear strength. The results of the laboratory tests are presented on Figures B-1 through B-13. The in-place dry density and moisture content of the samples tested are presented on the boring logs in *Appendix A*.

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% of dry wt.)
B-1 @ 0-5’	Silty SAND (SM), grayish brown	120.0	12.5
B-5 @ 1-5’	Silty SAND (SM), dark brown	111.5	12.5

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D4829

Sample No.	Moisture Content		After Test Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B-1 @ 0-5’	11.8	19.1	103.2	0
B-2 @ 5-7’	10.0	18.4	108.0	1

SUMMARY OF CORROSIVITY TEST RESULTS

Sample No.	Chloride Content (ppm)	Sulfate Content (%)	pH	Resistivity (ohm-centimeter)
B-4 @ 1-5’	40	0.044	7.24	320
B-5 @ 1-5’	180	0.000	8.32	26,000

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

SUMMARY OF LABORATORY R-VALUE TEST RESULTS
ASTM D2844

Sample No.	R-Value
B-4 @ 1-5’	55
B-6 @ 1-5’	70

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

DF

LABORATORY TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL 2019
PROJECT NO. T2857-22-01
FIG B-1

SUMMARY OF LABORATORY ORGANIC MATTER CONTENT TESTS
ASTM D2974 (Methods ‘A’ & ‘C’)

Sample No.	Organic Matter Content (%)
B-1 @ 2.5’	3.6
B-1 @ 7.5’	2.0
B-2 @ 2.5’	2.1
B-3 @ 2.5’	1.9
B-3 @ 10’	1.1
B-4 @ 2.5’	2.4
B-4 @ 5’	3.2
B-4 @ 20’	2.9
B-5 @ 2.5’	1.0
B-5 @ 7.5’	3.1
B-5 @ 10’	3.3
B-6 @ 2.5’	1.0
B-6 @ 5’	2.1

SUMMARY OF ONE-DIMENSIONAL CONSOLIDATION (COLLAPSE) TESTS
ASTM D2435

Sample No.	In-situ Dry Density (pcf)	Moisture Content Before Test (%)	Final Moisture Content (%)	Axial Load with Water Added (psf)	Percent Hydrocompression
B-2 @ 5’	103.8	22.0	20.6	2,000	0.02
B-2 @ 10’	111.5	16.7	15.2	2,000	0.03
B-3 @ 5’	101.5	23.7	22.7	2,000	0.02
B-3 @ 15’	101.4	24.7	22.9	4,000	0.10
B-5 @ 5’	100.2	9.0	20.5	2,000	0.30
B-5 @ 10’	90.7	31.4	30.8	2,000	0.01

LABORATORY TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

SUMMARY OF ATTERBERG LIMIT TEST RESULTS
ASTM D4318

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	USCS
B-2 @ 10’	**	**	0	ML
B-2 @ 35’	**	**	0	ML
B-3 @ 5’	23	19	4	CL-ML
B-4 @ 7.5’	26	17	9	CL
B-5 @ 10’	33	22	11	CL

** Non-plastic (NP): Material could not be rolled to 3 mm thread at any moisture content.

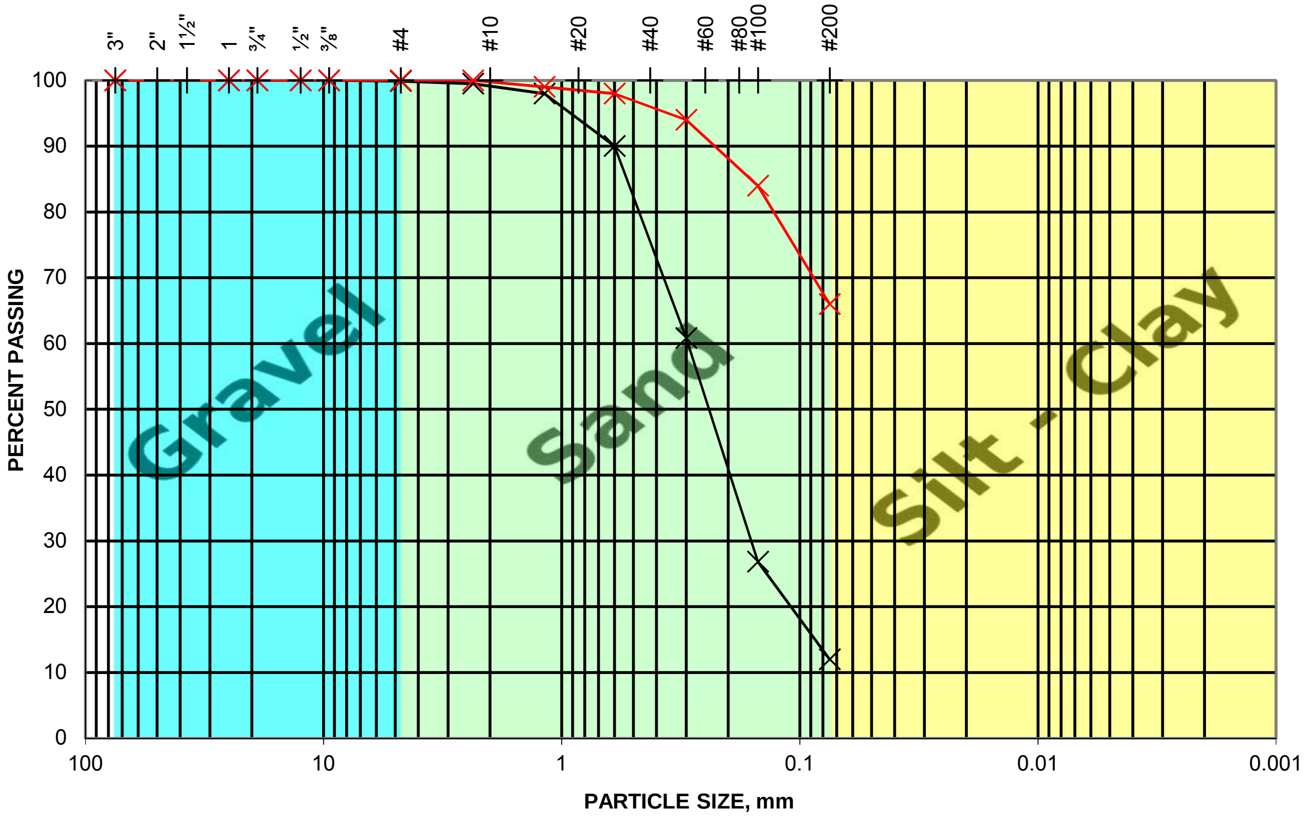
GEOCON
WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF

LABORATORY TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL 2019 | PROJECT NO. T2857-22-01 | FIG B-3



SAMPLE ID	SAMPLE DESCRIPTION
B-2 @ 25'	SM - Silty Sand
B-2 @ 35'	CL - Sandy Clay

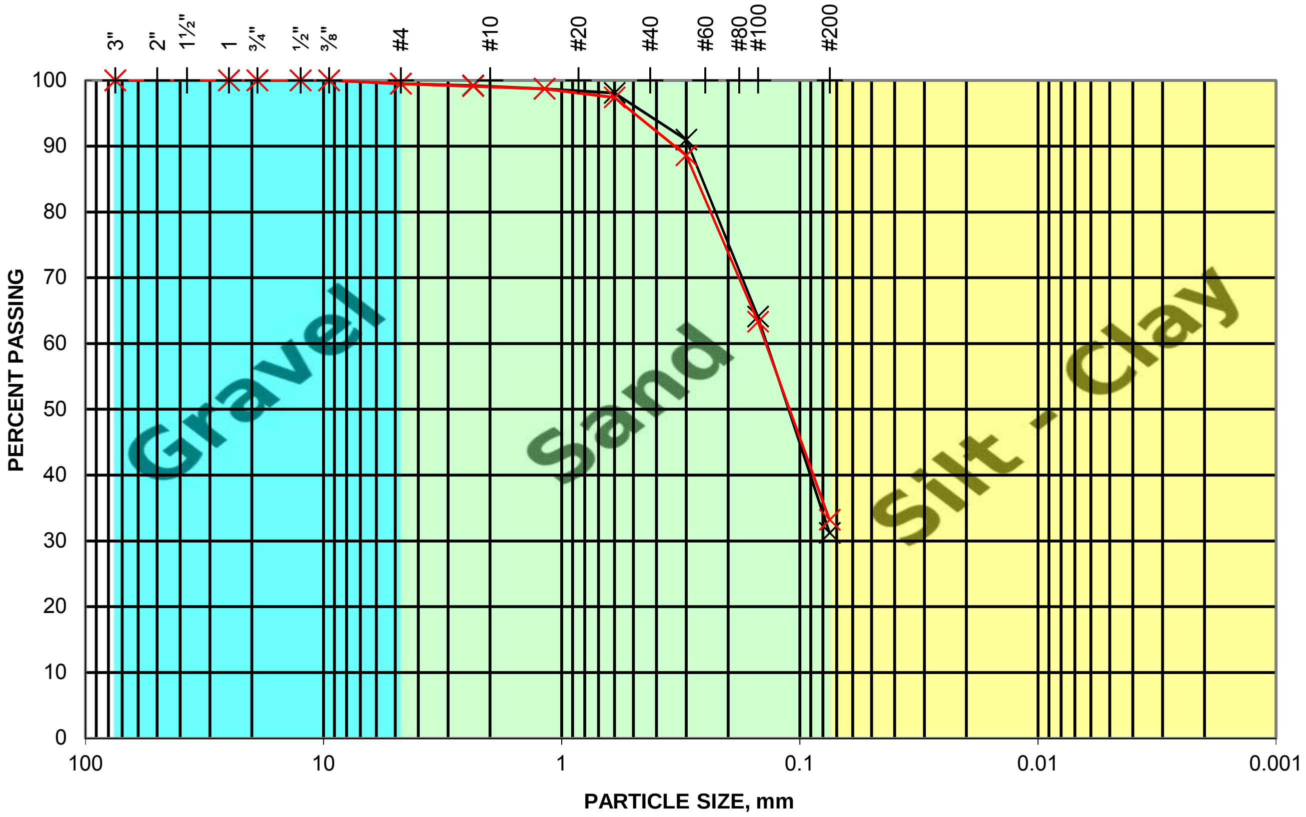
GEOCON
WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF		

GRAIN SIZE DISTRIBUTION

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019	PROJECT NO. T2857-22-01	FIG B-4



SAMPLE ID	SAMPLE DESCRIPTION
P-1 @ 6-8'	SM - Silty Sand
P-3 @ 6-8'	SM - Silty Sand

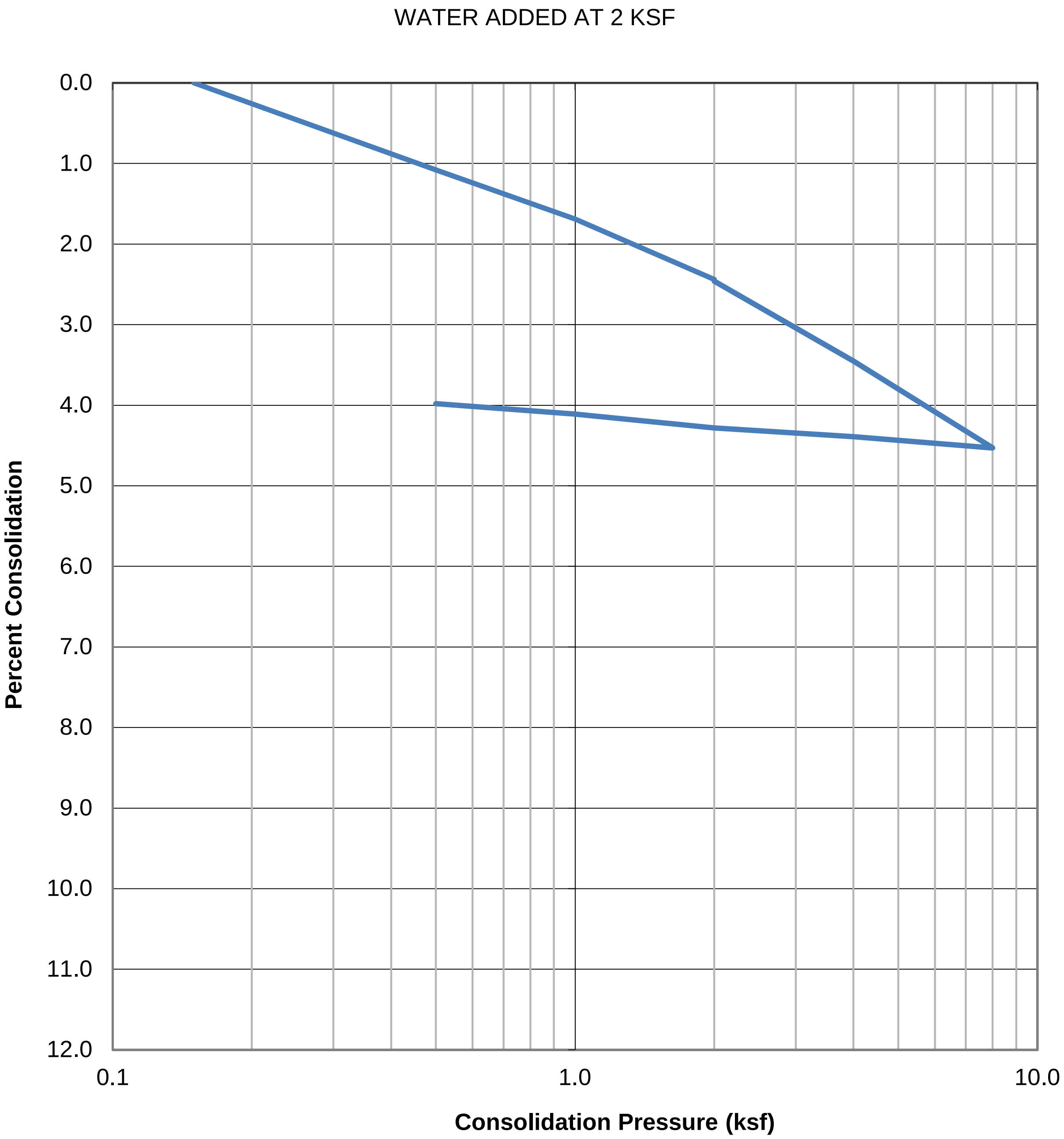
GEOCON
W E S T, I N C.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF		

GRAIN SIZE DISTRIBUTION

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019	PROJECT NO. T2857-22-01	FIG B-5



SAMPLE ID	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@5'	ML	103.8	22.0	20.6

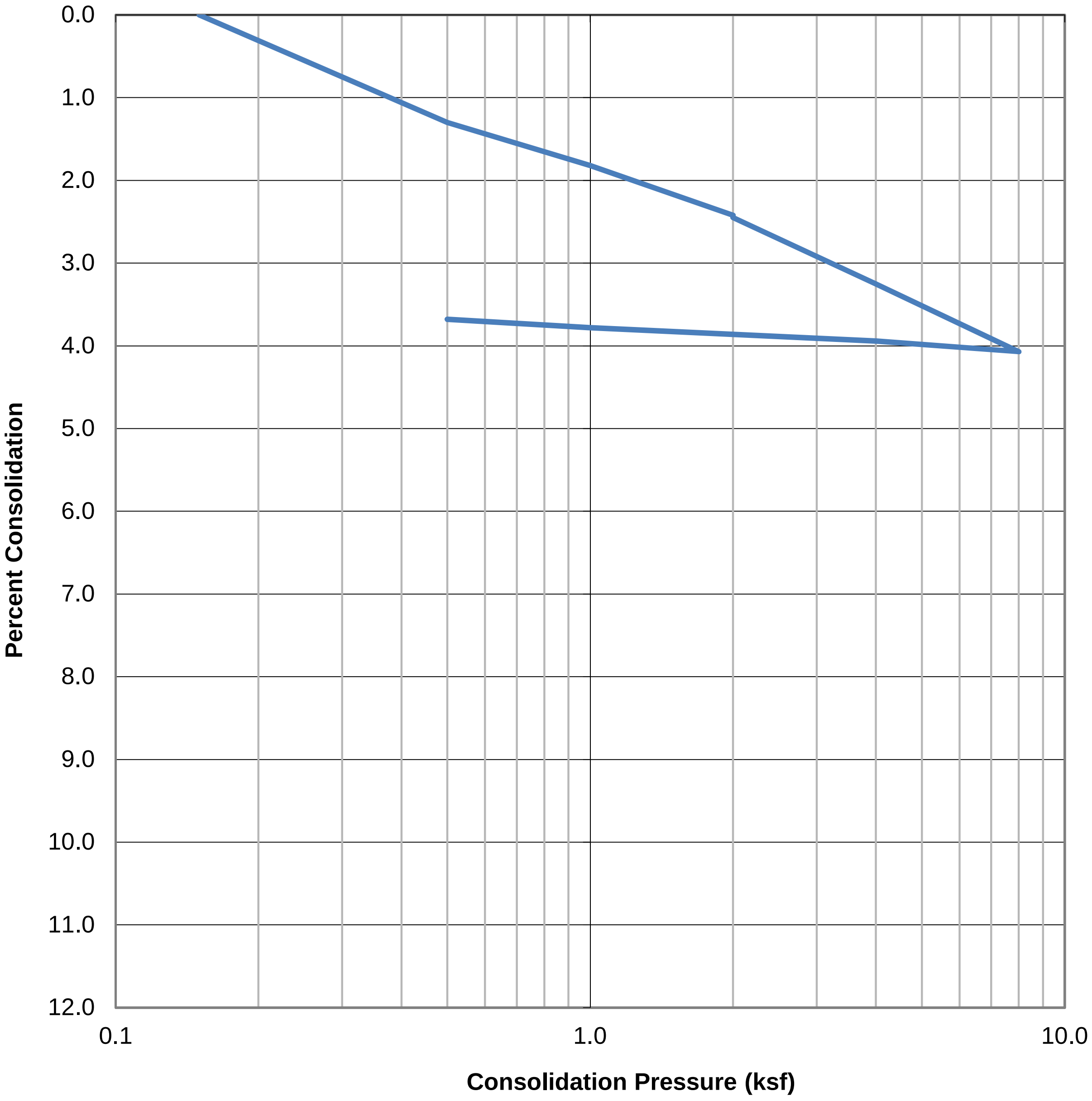
GEOCON
W E S T, I N C.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF

CONSOLIDATION TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
ANDARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019 | PROJECT NO. T2857-22-01 | FIG B-6



WATER ADDED AT 2 KSF

SAMPLE ID	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@10'	ML	111.5	16.7	15.2

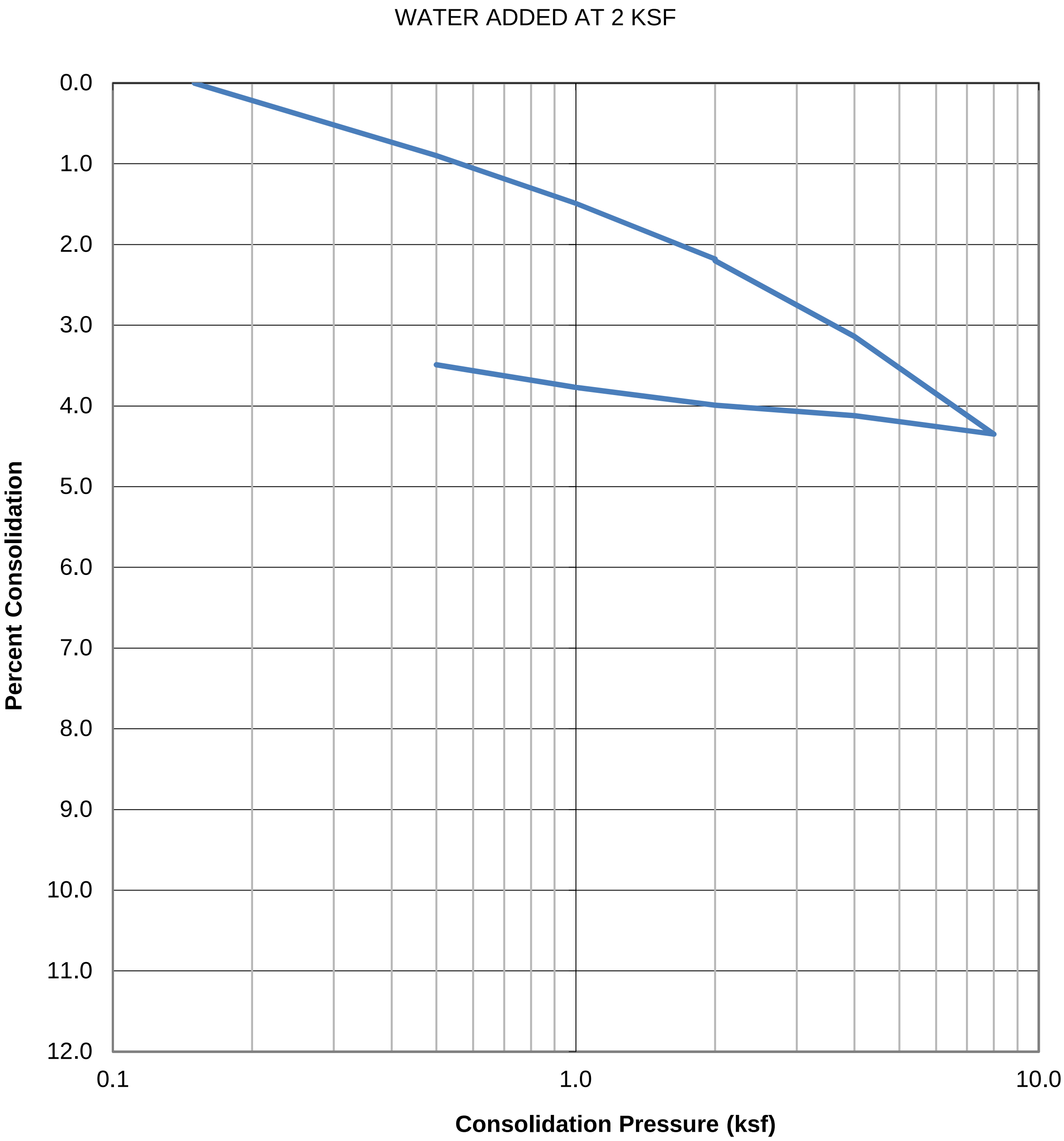
GEOCON
WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF

CONSOLIDATION TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019 | PROJECT NO. T2857-22-01 | FIG B-7



SAMPLE ID	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@5'	CL-ML	101.5	23.7	22.7

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

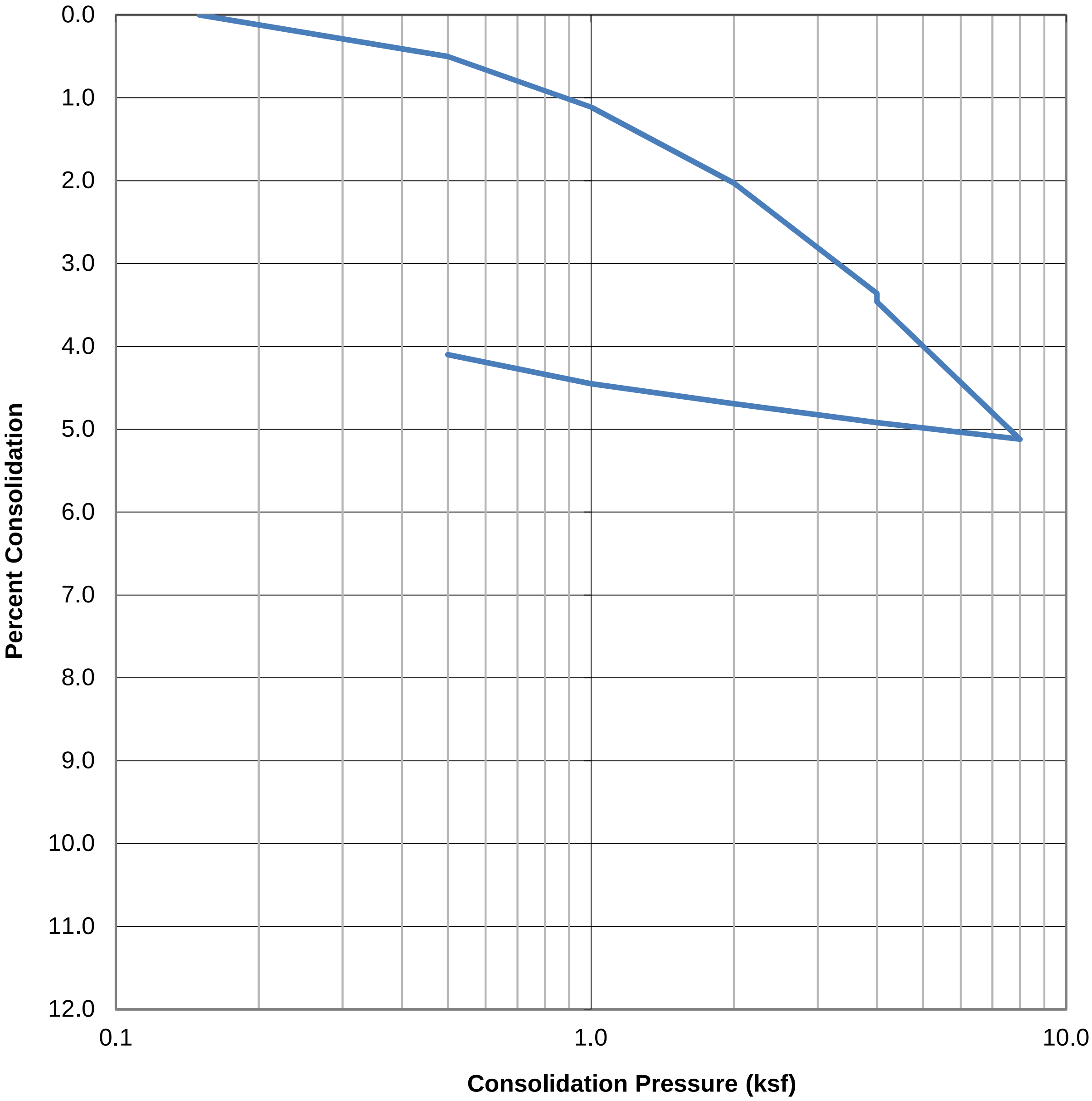
DF

CONSOLIDATION TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019 | PROJECT NO. T2857-22-01 | FIG B-8

WATER ADDED AT 4 KSF



SAMPLE ID	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@15'	CL	101.4	24.7	22.9

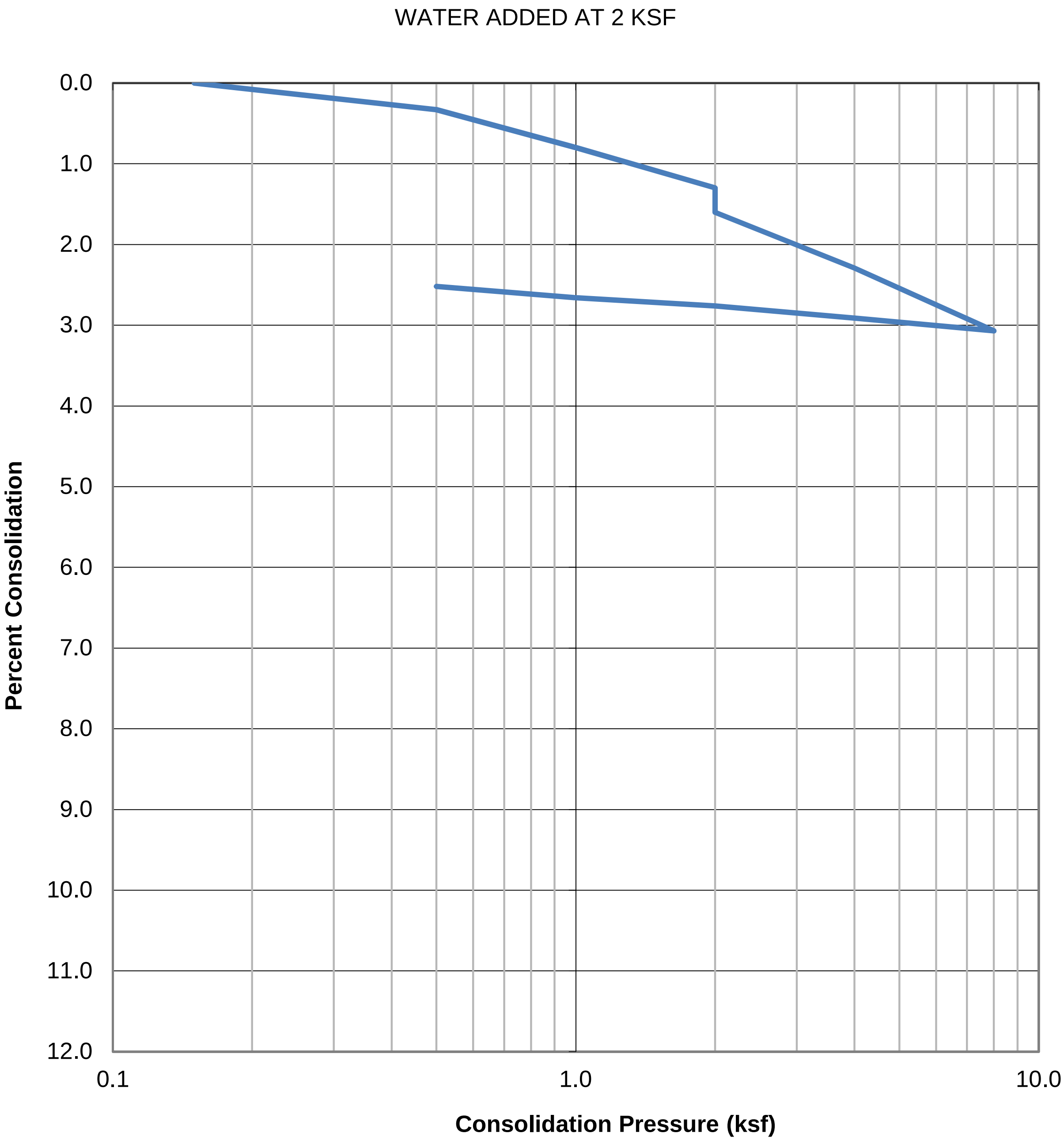
GEOCON
WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF

CONSOLIDATION TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019 | PROJECT NO. T2857-22-01 | FIG B-9



SAMPLE ID	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@5'	ML	100.2	9.0	20.5

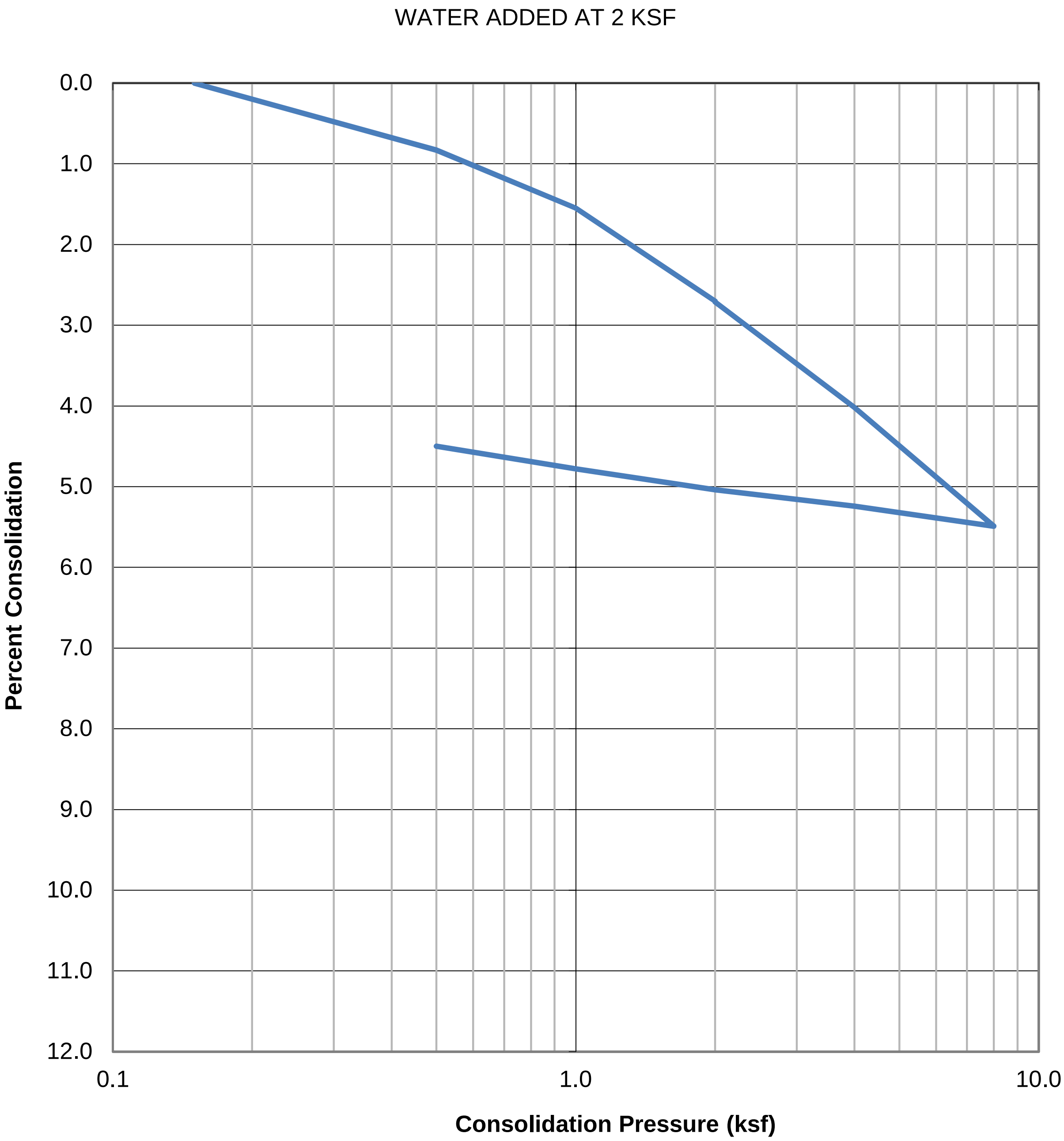
GEOCON
W E S T, I N C.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF

CONSOLIDATION TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019 | PROJECT NO. T2857-22-01 | FIG B-10



SAMPLE ID	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@10'	CL	90.7	31.4	30.8

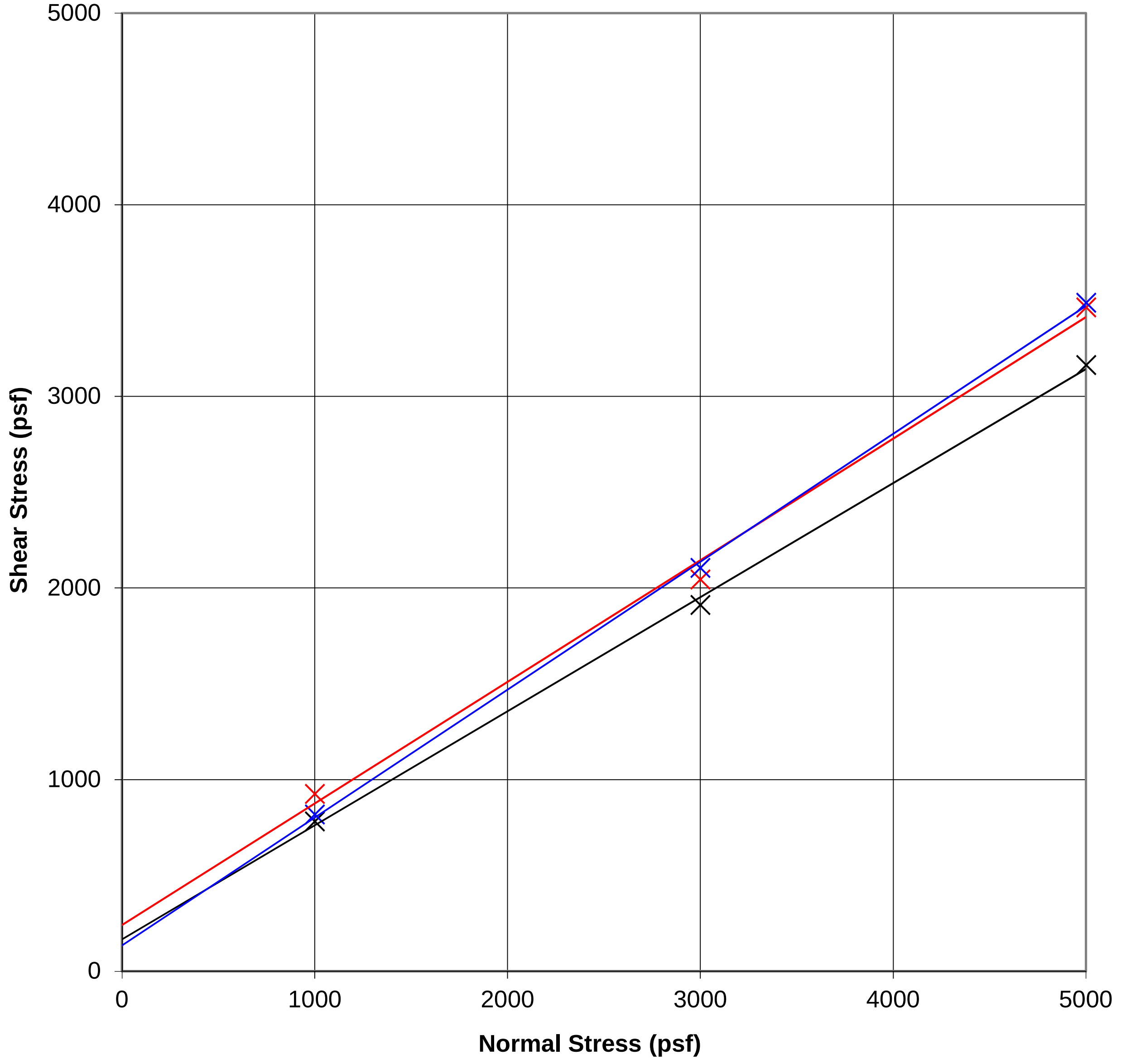
GEOCON
WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF

CONSOLIDATION TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019	PROJECT NO. T2857-22-01	FIG B-11



SAMPLE ID	SOIL TYPE	INITIAL DRY DENSITY (pcf)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)	C (psf)	ϕ (deg)
*B-1@0-5	SM	120.0	12.5	3.3	170	31
B-1@7.5	SM	113.7	15.4	20.6	240	32
B-3@7.5	CL-ML	96.8	27.5	24.3	130	34

*Sample remolded to approximately 90% of the test maximum dry density at optimum moisture content.

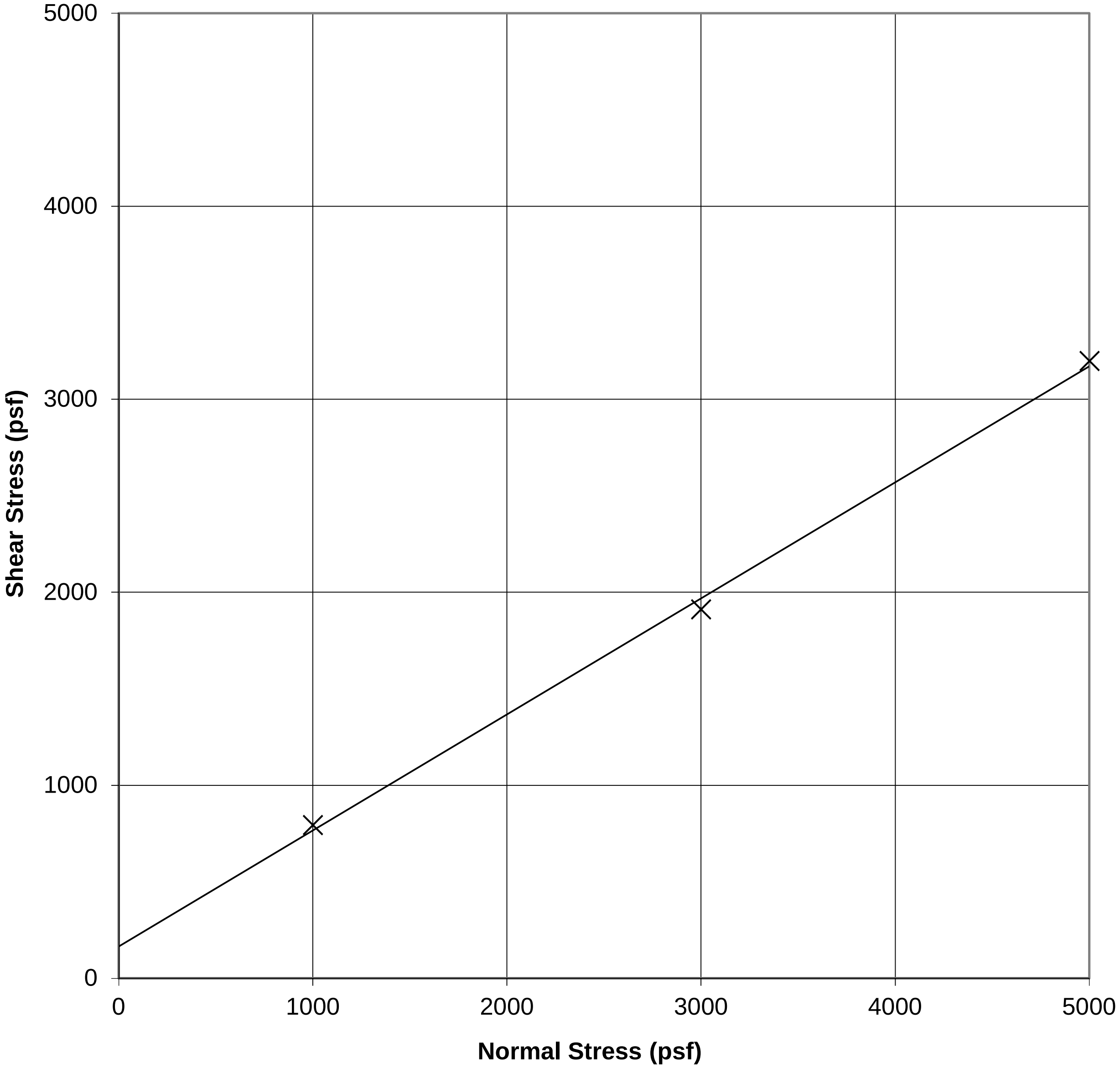
GEOCON
WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

DF

DIRECT SHEAR TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019	PROJECT NO. T2857-22-01	FIG B-12



SAMPLE ID	SOIL TYPE	INITIAL DRY DENSITY (pcf)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)	C (psf)	ϕ (deg)
*B-5@1-5	SM	111.5	12.5	11.2	160	31

*Sample remolded to approximately 90% of the test maximum dry density at optimum moisture content.

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

DF		

DIRECT SHEAR TEST RESULTS

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

APRIL, 2019	PROJECT NO. T2857-22-01	FIG B-13

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

THE HOMESTEAD INDUSTRIAL BUSINESS PARK
WEST OF LIMONITE AVENUE
AND ARCHIBALD AVENUE
EASTVALE, CALIFORNIA

PROJECT NO. T2857-22-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

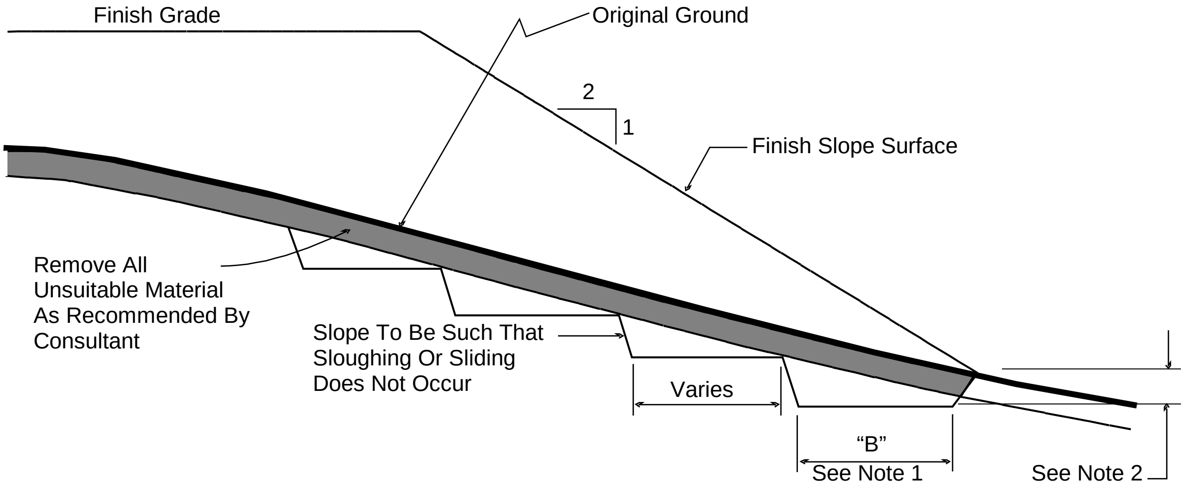
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



No Scale

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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

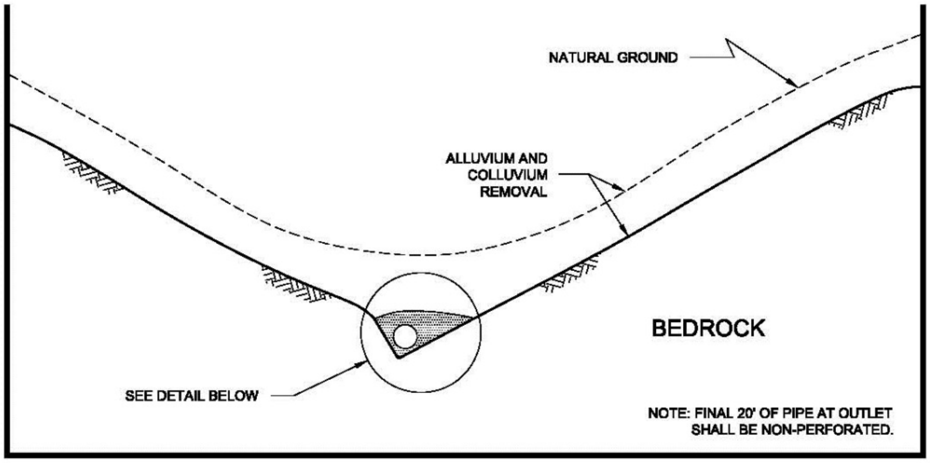
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

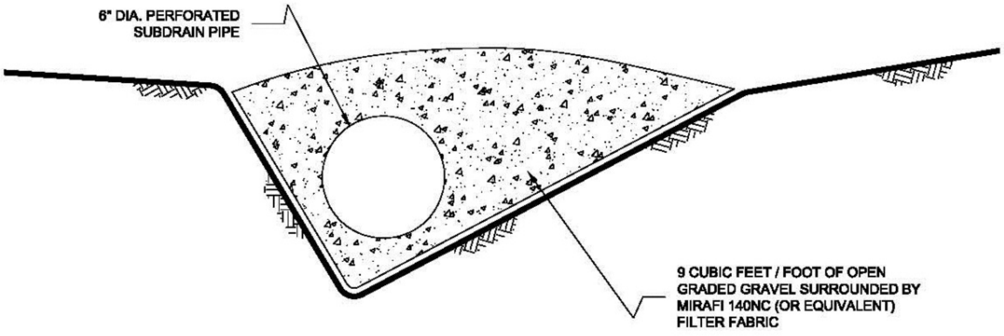
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL





NOTES:

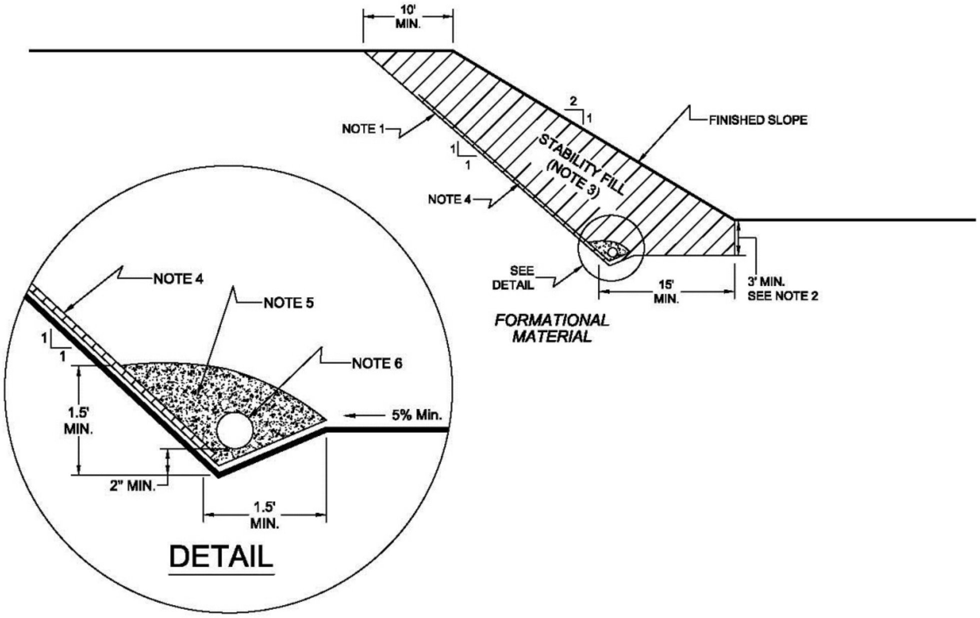
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

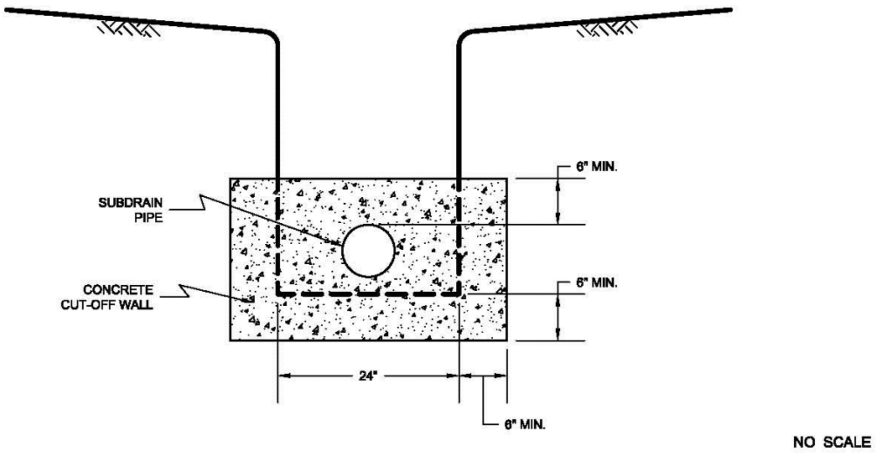
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

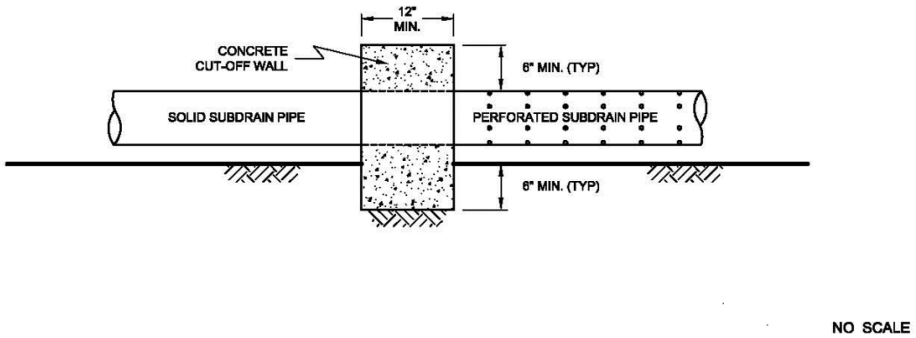
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



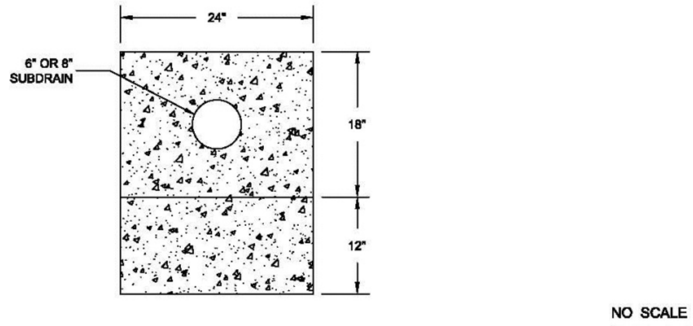
SIDE VIEW



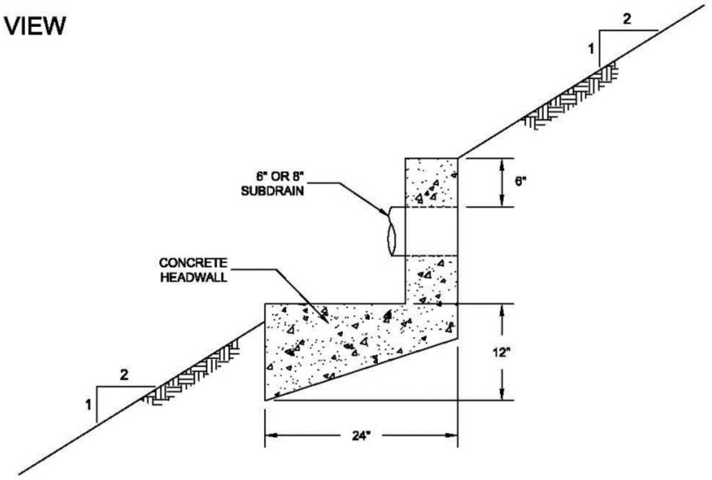
7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL

FRONT VIEW



SIDE VIEW



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

NO SCALE

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

8. OBSERVATION AND TESTING

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

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

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

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

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

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

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

9. PROTECTION OF WORK

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

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

10. CERTIFICATIONS AND FINAL REPORTS

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

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