GEOTECHNICAL UPDATE

BUILDER'S MAX
APN 371-150-001 & 371-150-002
GRAND AVENUE AT KATHRYN WAY
LAKE ELSINORE, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

BUILDER'S MAX
TEMECULA, CALIFORNIA

FEBRUARY 22, 2022
PROJECT NO. T2719-22-02

Project No. T2719-22-02
February 22, 2022

Builder's Max
31938 Temecula Parkway, Unit A369
Temecula, CA 92592

Attention: Mr. Steve Galvez

Subject: GEOTECHNICAL UPDATE
BUILDER'S MAX
APNS 371-150-001 & 371-150-002
GRAND AVENUE AT KATHRYN WAY
LAKE ELSINORE, CALIFORNIA

Dear Mr. Galvez:

In accordance with your authorization of Proposal No. IE-2887 dated December 1, 2021, Geocon West Inc. (Geocon) herein submits the results of our geotechnical update and percolation testing for the subject site. The accompanying report presents the results of our previous study, results of recent percolation testing, and updated geotechnical parameters in accordance with the 2019 California Building Code for the proposed light industrial development. The site is considered suitable for development provided the recommendations of this report are followed.

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

Very truly yours,

GEOCON WEST, INC.

PROFESSIONAL GEOLOGIST
LISA A. BATTIATO
No. 2316
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Lisa A. Battiato
CEG 2316

Petrina Zen

REGISTERED PROFESSIONAL ENGINEER
PETRINA ZEN
C87489
CIVIL
STATE OF CALIFORNIA

Petrina Zen
PE 87489

REGISTERED PROFESSIONAL ENGINEER
JOSEPH J. VETTEL
No. 2401
GEOTECHNICAL
STATE OF CALIFORNIA

Joseph J. Vettel
GE 2401

LAB:PZ:JJV:hd
(e-mail)

TABLE OF CONTENTS

TABLE OF CONTENTS (Concluded)

GEOTECHNICAL UPDATE

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical update and percolation testing for design of site storm water infiltration structures for the proposed light industrial development to be constructed within APNs 371-150-001 & -002 north of Grand Avenue and Kathryn Way in Lake Elsinore, California. Four buildings are planned within the central and southwestern area of the site. A building materials storage yard is planned for the northeastern area of the site, within the recommended building set back zone along the Elsinore fault. The site location is depicted on the *Vicinity Map*, Figure 1. The purposes of this geotechnical update are to provide geotechnical and seismic design parameters per the 2019 California Building Code (CBC) and present infiltration rates at four locations proposed for site BMPs. In addition, this report provides recommendations for remedial grading, shallow foundations, concrete slab-on-grade, concrete flatwork, preliminary pavement sections, lateral loading, and retaining walls. This update also includes a review of readily available published and unpublished geologic literature (see *List of References*).

The scope of this update included performing a site reconnaissance, field exploration, percolation testing, laboratory testing, engineering analyses and preparing this report. We drilled four percolation borings to depths of four feet on the site on February 2, 2022 and percolation testing was performed on February 3, 2022. The recent boring logs and percolation data sheets are presented in Appendix A along with previous geotechnical borings within the proposed development area of the site (Borings 1 through 6, and 10). The *Geologic Map*, Figure 2, presents the approximate locations of the borings and percolation tests. Appendix A provides a detailed discussion of the field investigation and includes logs of the borings with moisture and dry density results and percolation test results. Details of the laboratory tests and a summary of the test results are presented in *Appendix B* and on the boring logs in *Appendix A*.

Recommendations presented herein are based on analyses of data obtained from this study and the previous investigation performed by Geocon in 2016 and our understanding of proposed site development. References reviewed to prepare this report are provided in the *List of References*. If project details vary significantly from those described herein, Geocon should be contacted to evaluate the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The site is located northeast of Grand Avenue and northwest of Kathryn Way in Lake Elsinore, California. The property is bounded on the northwest by rural properties and northeast by Lake Elsinore and habitat mitigation land. The existing grades range from approximately 1,283 feet above mean sea level (MSL) near Grand Avenue to approximately 1,271 feet MSL near the northeastern most proposed building. The site is generally flat with a few isolated trees and a very low growth of recent grass. The site is at approximately latitude 33.63384 and longitude -117.3329.

A site plan depicting the building locations and finish floor elevations indicates finish grades will be 1,271 to 1,267 feet MSL resulting in cuts of approximately 5 to 12 feet. The site BMPs will be located along Kathryn Way and northeast of the paved storage yard. The *Preliminary Grading Plan* dated December 31, 2021, provided by Grant Becklund, Civil Engineer was utilized as the base for our *Geologic Map*, Figure 2.

We expect that the proposed light industrial buildings will either be concrete tilt up structures or wood/steel framed buildings with stucco exteriors. We expect spread footing foundations and slab on grade floors.

Due to the preliminary nature of the design at this time, wall and column loads were not available. We anticipate that column loads for the proposed structures will be up to 150 kips, and wall loads will be up to 2 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located within the Peninsular Ranges Geomorphic Province (Province) at the boundary of the Perris and Santa Ana Mountain Blocks. In the vicinity of the site, the Perris Block is characterized by highlands which display elevated erosional surfaces surrounded by alluviated and fault bounded valleys. The Santa Ana Mountains Block is characterized by heterogeneous granitic bedrock with a moderate amount of volcanic and metamorphic rocks, and some terrestrial sedimentary rocks. The Peninsular Ranges are bound by the Transverse Ranges (San Gabriel and San Bernardino Mountains) to the north and the Colorado Desert Geomorphic Province to the east. The Peninsular Ranges Geomorphic Province extends westward into the Pacific Ocean and southward to the tip of Baja California. Overall, the Province is characterized by Cretaceous-age granitic rock and a lesser amount of Mesozoic-age metamorphic rock overlain by terrestrial and marine sediments. Faulting within the Province is typically northwest trending and includes the San Andreas, San Jacinto, Elsinore, and Newport-Inglewood faults. Locally, the site is in the Elsinore Valley, a pull-apart basin as a result of a step-over within the Elsinore Fault Zone. Conversely, Rome Hill located just to the southeast (and the moderate relief hill located in the northeastern portion of the site) are the result of compressional stresses of strike slip faulting within the zone. The Willard strand of the Elsinore fault zone is mapped in the eastern portion of the site. The entire site is within a Riverside County Fault Hazard Zone.

4. GEOLOGIC MATERIALS

4.1 General

Site geologic materials encountered consist of undocumented artificial fill, lacustrine deposits, alluvial fan deposits and the fanglomerate member of the Pauba Formation. The descriptions of the soil and geologic conditions are shown on the excavation logs located in *Appendix A*, the *Geotechnical Map* (Figure 2), and described herein; generally following the nomenclature of Morton and Weber, 2003 (see *List of References*).

4.2 Undocumented Artificial Fill (afu)

Undocumented fill was observed in the excavations to depths of approximately 14 feet within the central and eastern portions of the site. We understand that the fill was placed on the site after flooding in the 1980's. There is no documentation of removals beneath the fill or geotechnical testing and observation of fill placement. Therefore, it is considered undocumented. The unit consists of silty to clayey sands, silts, and clays which are loose (soft) to medium dense (firm) and dry to moist, gray to brown, and has varying amounts of debris, including rocks and asphaltic concrete.

Undocumented fill is also present within the fault trench excavations as loose backfill.

4.3 Lacustrine Deposits (Ql) – not a mapped unit

Lacustrine (lake) deposits underlie the undocumented fill in the central and eastern portions of the site at depths of 4 to 30 feet (Borings B-5 and B-6). This unit consists of silty sands and silts that are thickly bedded and have abundant carbonate throughout the unit. Porosity was observed within these deposits at depths of 13 to 23 feet. The lacustrine deposits are fine to coarse, moist to wet, and loose (soft) to dense (stiff). The soil color is generally olive grey to yellowish brown. Consolidation test results generally showed 0 to 0.6 percent collapse with one sample showing 11.8 percent collapse when inundated with water.

4.4 Alluvial Fan Deposits (Qyf)

Alluvial fan deposits were encountered at the surface in the western portion of the site and underlying the undocumented fill and lacustrine deposits in the central and eastern portions of the site to depths of 51½ feet. This unit consists predominately of silty sand that is medium dense to very dense, moist to wet, reddish brown to dark gray with some mottling. The unit contained variable amounts of clay and gravel in localized deposits.

4.5 Pauba Formation (Qpfs)

The fanglomerate member of the Pauba Formation was encountered at depths of 19½ to 51½ feet within the western and central portions of the site. As encountered, the Pauba consists of well graded to silty sandstone which is moist to wet, and poorly to non-indurated. The Pauba is moderately hard to very hard.

5. GROUNDWATER

Groundwater was encountered in boring B-1 at a depth of 36 feet, 11 inches, and it stabilized at 26 feet, 6 inches during drilling. It was also encountered in boring B-12 (in the northeastern area of the property, outside of the current project limits) at a depth of 40 feet and rose to a height of 35 feet during drilling. California Department of Water Resources well data indicates groundwater has been measured at depths of approximately 56 to 58 feet below ground surface in nearby wells. Depth to groundwater may also fluctuate based on the water level in Lake Elsinore during annual variations. Groundwater should be anticipated at or slightly above the lake water elevation. During the rainy season, localized perched water conditions may develop above less permeable units that may require special consideration during grading operations. Groundwater elevations and seepage are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result.

6. GEOLOGIC HAZARDS

6.1 Seismic Hazard Analysis

The site is located within a Riverside County Fault Zone. The Willard branch of the Elsinore Fault Zone bisects the site as shown on the *Regional Geologic Map* (Figure 3). Terra Geosciences performed fault trenching for the project and reported their results under separate cover (2015, 2016). These reports were approved by the County Geologist in November, 2016. We understand that the locations of the faults and fault trenches were surveyed and building setback zones were established for the project. The approximate building setback zone is depicted on Figure 2 herein.

Significant active faults within a 100 kilometer radius of the site are listed in Table 6.1.1 and include directions and distances from the site, and the potential earthquake magnitudes. Historic earthquakes of magnitude 6.0 and greater within 100 miles of the site are listed in Table 6.1.2 and include the fault names, directions and distances from the site, and the magnitudes of the seismic events.

TABLE 6.1.1
SIGNIFICANT ACTIVE FAULTS WITHIN 100 KM OF THE SITE

Fault	Direction	Distance from Site (km)	Magnitude
Elsinore	E	0	6.8
Casa Loma (San Jacinto)	NE	34	6.9
Claremont (San Jacinto)	NE	37	6.9
Chino-Central Avenue	NNW	40	6.7
Newport Inglewood	W	48	7.1
Whittier	NW	51	6.8
San Andreas	NNE	56	7.5
Cucamonga	NW	61	6.9
Coronado Bank	SW	69	7.2
San Diego Trough	SW	85	7.2
Rose Canyon	SW	90	7.2

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.

6.2 Seismicity

As with all of southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. A number of earthquakes of moderate to major magnitude have occurred in the southern California area within the last 100 years. A partial list of these earthquakes is included in the Table 6.2.

TABLE 6.2
HISTORIC EARTHQUAKE EVENTS WITH REPECT TO THE SITE

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	26	NNE
Long Beach	March 10, 1933	6.4	36	W
Tehachapi	July 21, 1952	7.5	134	NW
San Fernando	February 9, 1971	6.6	81	NW
Whittier Narrows	October 1, 1987	5.9	52	WNW
Sierra Madre	June 28, 1991	5.8	58	NW
Landers	June 28, 1992	7.3	65	NE
Big Bear	June 28, 1992	6.4	49	NE
Northridge	January 17, 1994	6.7	80	WNW
Hector Mine	October 16, 1999	7.1	90	NE
Ridgecrest China Lake Fault	July 5, 2019	7.1	148	N

6.3 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 and density 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.

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

A review of the County of Riverside Land Information System indicates that the site is located within an area designated as having a high potential for liquefaction.

Liquefaction analysis of the soils underlying the site (approximate elevation of 1,262 through 1,212 feet MSL and 1,270 to 1,250 feet MSL) was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

The liquefaction analysis was performed for a Design Earthquake level by using a “model” historic high groundwater table of 5 feet below the ground surface, a magnitude 6.49 earthquake, and a peak horizontal acceleration of 0.585g (⅔PGA_M). The enclosed liquefaction analyses, included herein for boring B-1, indicates that the alluvial and lacustrine soils below the proposed foundation level would be prone to up to approximately 3 inches of liquefaction settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 3 and 4). The site could be prone to differential settlements up to approximately 1.5 inches over a distance of 50 feet at the ground surface.

We understand that the intent of the Building Code is to maintain “Life Safety” during Maximum Considered Earthquake level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structure.

The liquefaction analysis was also performed for Maximum Considered Earthquake levels by using a “model” historic high groundwater table of 5 feet below the ground surface, a magnitude 7.71 earthquake, and a peak horizontal acceleration of 0.877g (PGA_M). The enclosed liquefaction analysis, included herein for boring B-1, indicates that the alluvial and lacustrine soils below the proposed foundation would be prone to up to approximately 3 inches of liquefaction settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 5 and 6).

6.4 Expansive Soil

The geologic units generally consist of silty sands with some clay deposits. Laboratory testing results indicate the soils tested exhibit a “very low” expansion potential (EI of 20 or less) as defined by ASTM International (ASTM) D4829.

6.5 Landslides

There are no steep slopes on or adjacent to the site. Therefore, landslides are not a design consideration for the site.

6.6 Rock Fall Hazards

The closest mountains are the Santa Ana Mountains, approximately 1,000 feet to the west. Due to shallow moderate slope angle, the moderate soil development, and abundant brush on the slopes, rock falls are not a design consideration for the site.

6.7 Slope Stability

Based on the preliminary site plan and relatively flat site topography, it does not appear that significant slopes will be constructed. Therefore, slope stability will not be a design consideration for the site.

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 more than 22 miles from the nearest coastline with a mountain range in between; therefore, 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. Lake Elsinore is located approximately 1,600 feet north of the site and has a water surface elevation of approximately 1,238 feet MSL. Lake water surface elevations are 1,244 feet MSL and outflow channel elevations are 1,255 feet MSL. The hill located just northwest of the proposed development acts as a barrier between the site and the lake, with a peak of approximately 1,283 feet above MSL. Further, the proposed development elevations will be approximately 1,268 feet above MSL. The civil engineer should use these criteria to evaluate if there is a hazard for a seiche to affect the site.

7. SITE INFILTRATION

Percolation testing was performed in accordance with the procedures outlined in *Riverside County Flood Control and Water Conservation District LID BMP*, *Appendix A* for the proposed infiltration structures along the eastern area of the site. The percolation test locations are depicted on the *Geologic Map*, Figure 2.

Percolation borings P-1 through P-4 were excavated at the proposed BMP locations at depths of 4 feet per Grant Becklund. Three-inch diameter perforated pipe wrapped in filter fabric was placed within the borings. Gravel was placed at the bottom of the hole and around the pipe. Percolation testing began within 24 hours after the holes were presaturated. Percolation data sheets are presented in *Appendix A* of this report. Results of the converted percolation test rates to infiltration test rates are presented in Table 7.

TABLE 7
INFILTRATION TEST RATES FOR PERCOLATION AREAS

Parameter	P-1	P-2	P-3	P-4
Depth (inches)	48	48	48	48
Test Type	Sandy	Normal	Sandy	Normal
Change in head over time: ∆H (inches)	4.2	2.2	3.8	1.3
Average head: Havg (inches)	30.9	22.9	25.1	21.5
Time Interval (minutes): ∆t (minutes)	10	30	10	30
Radius of test hole: r (inches)	4	4	4	4
Tested Infiltration Rate: It (inches/hour)	1.5	21.4	1.7	13.5

The results of the infiltration testing indicate that infiltration at the locations tested ranged from 1.5 to 21.4 inches per hour.

The in-situ field percolation tests performed provide short-term infiltration rates, which apply mainly to the initiation of the infiltration process due to the short time of the test (hours instead of days) and the amount of water used. Where appropriate, the short-term infiltration rates shall be converted to long-term infiltration rates using reduction factors depending on the degree of infiltrate quality, maintenance access and frequency, site variability, subsurface stratigraphy variation, and other factors. The small-scale percolation testing cannot model the complexity of the effect of interbedded layers of different soil composition, and our test results should be considered only as index values of infiltration rates.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude construction of the proposed project provided the recommendations presented herein are followed and implemented during design and construction.

8.1.2 Up to 12 feet of undocumented artificial fill was encountered during the site investigation within the planned structural improvement areas. The undocumented fill encountered is believed to have been placed to elevate the land surface after flooding in the 1980's. Deeper fill may exist in other areas of the site that were not directly explored. The undocumented fill, in its present condition, is not suitable for direct support of proposed foundations, slabs, or additional fill. The undocumented fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed.

8.1.3 Several fault trenches have been excavated within the site. We understand that these excavations were surveyed and loosely backfilled. Trenches within area to receive structural or flatwork improvements should be located, re-excavated, and backfilled in accordance with the recommendations herein during grading.

8.1.4 Active faulting is present in the eastern portion of the site. Building setbacks were recommended by Terra Geoscience. The fault and building setback locations should be plotted on the project grading plans and clearly staked by survey in the field for compliance during site construction.

8.1.5 The enclosed seismically-induced settlement analysis indicates that alluvial soils underlying the site could be prone to up to approximately 3 inches of liquefaction settlement as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$). The resulting differential settlement at the ground surface is anticipated to be up to approximately 1.5 inches over a horizontal distance of 50 feet. The foundation recommendations presented in this report are intended to reduce the effects of differential settlement on proposed structures.

8.1.6 The upper 8 to 12 feet of site soils consist of undocumented fill or compressible alluvial soils which should be removed and replaced with compacted fill within areas to receive improvements during grading.

8.1.7 Where relatively loose wet, or soft soils are encountered in the site excavations, subgrade stabilization may be required prior to placing fill or installing utilities. The contractor should be prepared to mitigate these conditions.

8.1.8 Subsequent to the recommended grading, the proposed structures may be supported on a reinforced concrete mat foundation system deriving support on newly placed engineered fill. A mat foundation is more capable of mitigating the effects of differential settlement of the underlying soils, as well as distributing the structural loads of the building, thereby minimizing loads imposed on the supporting soils.

8.1.9 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. However, if excavations in proximity to an adjacent property line, utility lines, and/or structures are required, special excavation measures may be necessary in order to maintain lateral support of existing improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report.

8.1.10 Where new paving is to be placed, we recommend that existing undocumented fill and soft lacustrine/alluvial soils be excavated and properly compacted for paving support. As a minimum, the upper 24 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report.

8.2 Soil Characteristics

8.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are present.

8.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

8.2.3 Onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping, shoring, or slot cutting. Excavation recommendations are provided in the *Temporary Excavations* section of this report.

8.2.4 The upper 5 feet of site soils encountered in the field investigation are considered to be "Non-Expansive" (very low Expansion Index [EI] of 20 or less based on ASTM D4829, See Figure B-3) as defined by 2019 California Building Code (CBC) Section 1803.5.3. Table 8.2.4 presents soil classifications based on the EI. Recommendations presented herein assume that the building foundations and slabs will derive support in these materials

TABLE 8.2.4
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2019 CBC Expansion Classification
0 – 20	**Very Low**	**Non-Expansive**
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

8.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on a representative sample of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are not considered a corrosive environment in accordance with Caltrans corrosion criteria (Caltrans, 2021). The results are presented in *Appendix B* and should be considered for design of underground structures.

8.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in *Appendix B* and indicate that the on-site materials possess "negligible" sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-19 Chapter 19.

TABLE 8.3.2
REQUIREMENTS FOR CONCRETE
EXPOSED TO SULFATE-CONTAINING SOLUTIONS

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

8.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion.

8.4 Grading

8.4.1 Grading is anticipated to include preparation of building pads, excavation of site soils for proposed foundations, utility trenches, and placement of backfill for utility trenches.

8.4.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon. The existing fill encountered during exploration is suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris is removed.

8.4.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and, if applicable, building official in attendance. Special soil handling requirements can be discussed at that time.

8.4.4 Grading should commence with the removal of existing vegetation, improvements, and undocumented fill, and previous fault trench backfill from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. Existing underground improvements planned for removal should be excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer.

8.4.5 As a minimum, we recommend that the undocumented fill and/or the upper 8 to 12 feet of existing site soils within the proposed building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove deeper artificial fill or soft alluvial or lacustrine soil at the direction of the Geotechnical Engineer. The excavation should extend laterally a minimum distance equal to the depth of the over excavation beyond the building footprint area, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Proposed foundations should be underlain by at least 3 feet of newly compacted engineered fill. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities.

8.4.6 Excavations must be observed and approved in writing by the Geotechnical Engineer. Prior to placing fill, the excavation bottom must be proof-rolled with heavy equipment in the presence of the Geotechnical Engineer.

8.4.7 Where relatively loose or soft soils are encountered in the site excavation walls and bottom, subgrade stabilization may 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- to 6-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.

8.4.8 We expect that stable excavations can be achieved with sloping measures. Excavation recommendations are provided in the *Temporary Excavations* section of this report.

8.4.9 Fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near optimum moisture content, and properly compacted to a minimum 90 percent of the maximum dry density per ASTM D 1557 (latest edition).

8.4.10 Where new paving is to be placed, we recommend that undocumented fill and soft native soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of undocumented fill and soft soils in the area of new paving is not required; however, paving constructed over existing undocumented fill or unsuitable alluvial or lacustrine soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 24 inches of soil should consist of properly compacted fill as described in Section 8.4.9. The upper 12 inches of subgrade in pavement areas should be moisture conditioned and compacted to at least 95 percent relative compaction near optimum moisture content. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report.

8.4.11 Imported fill shall be observed, tested, and approved by Geocon prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils. If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer. Soils can be borrowed from non-building pad areas and later replaced with imported soils.

8.4.12 If import soils will be utilized, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

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

8.5 Earthwork Grading Factors

8.5.1 Estimates of shrinkage factors are based on empirical judgments comparing the material in its existing or natural state as encountered in the exploratory excavations to a compacted state. Variations in natural soil density and in compacted fill density render shrinkage value estimates very approximate. As an example, the contractor can compact the fill to a dry density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has an approximately 10 percent range of control over the fill volume. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate variations.

8.6 Utility Trench Backfill

8.6.1 Utility trenches should be properly backfilled in accordance with the requirements of the latest edition of the *Standard Specifications for Public Works Construction* (Greenbook) and applicable agency requirements. The pipes should be bedded with well-graded crushed rock or clean sand (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe.

8.6.2 If open graded rock is used, it should be wrapped in filter fabric to prevent finer soils from migrating into the rock voids. The remainder of the trench backfill may be derived from onsite soil or approved import soil. 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.

8.6.3 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 90 percent of the laboratory maximum dry density and moisture conditioned at or slightly above optimum moisture content (as determined by ASTM D1557). Backfill at the finish subgrade elevation of new pavements should be compacted to at least 95 percent of the maximum dry density. Backfill materials placed below the recommended moisture content may require additional moisture conditioning prior to placing additional fill.

8.7 Seismic Design Criteria

8.7.1 Table 8.7.1 summarizes site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. We used the computer program *Seismic Design Maps*, provided by the Structural Engineers Association (SEA) to calculate the seismic design parameters. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. Although the site is liquefiable, we expect the planned structures will have a period of ½ second or less and therefore can be classified as Site Class D. The values presented herein are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 8.7.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.855g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.673g	Figure 1613.3.1(2)
Site Coefficient, F_A	1	Table 1613.3.3(1)
Site Coefficient, F_V	1.7*	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.855g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	1.144g*	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.237g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.763g*	Section 1613.3.4 (Eqn 16-40)
Note: Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed. *See Section 11.4.8		

8.7.2 Table 8.7.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16 for the mapped maximum considered geometric mean (MCE_G).

TABLE 8.7.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Site Class	D	Section 1613.2.2 (2019 CBC)
Mapped MCE_G Peak Ground Acceleration, PGA	0.797g	Figure 22-9
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.877g	Section 11.8.3 (Eqn 11.8-1)

8.7.3 Conformance to the criteria in Tables 8.7.1 and 8.7.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.7.4 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 2019 California Building Code and ASCE 7-16, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the building code is to maintain "Life Safety" during an MCE event. The Design Earthquake

8.7.5 Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

8.7.6 Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 7.71 magnitude event occurring at a hypocentral distance of 2.1 kilometers from the site.

8.7.7 Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.49 magnitude occurring at a hypocentral distance of 6.85 kilometers from the site.

8.7.8 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.

8.8 Mat Foundation

8.8.1 Subsequent to the recommended grading, a mat foundation system may be utilized for support of the proposed structure provided foundations derive support on a blanket of newly placed engineered fill. Proposed foundations should be underlain by at least 3 feet of newly compacted engineered fill.

8.8.2 We expect that the mat foundation will impart an average pressure of less than 1,000 pounds per square foot (psf), with locally higher pressures up to 3,000 psf. The recommended maximum allowable bearing value is 3,000 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.8.3 A modulus of subgrade reaction of 100 to 150 pounds per cubic inch (pci) may be used in the design of mat foundations deriving support in newly placed engineered fill. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

8.8.4 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.

8.8.5 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

8.8.6 For seismic design purposes, a coefficient of friction of 0.40 may be utilized between the concrete mat and newly placed engineered fill, and 0.15 for slabs underlain by a moisture barrier.

8.8.7 Foundation excavations should be observed by the Geotechnical Engineer, prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.8.8 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

8.9 Foundation Settlement

8.9.1 The enclosed liquefaction analysis indicates that the alluvial soils could be prone to up to 3 inches of liquefaction settlement as a result of the Design Earthquake ground motion. The resulting differential settlement at the ground surface is anticipated to be up to approximately 1.5 inches over a horizontal distance of 50 feet. These settlements are in addition to the static settlements indicated below and must be considered in the structural design.

8.9.2 The maximum expected static settlement for the structures supported on a mat foundation system deriving support in newly compacted engineered fill and utilizing an average bearing pressure of 1,000 psf is estimated to be less than 0.75 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed 0.5 inch over a distance of 40 feet.

8.9.3 Based on seismic considerations, the proposed structure supported on a mat foundation system should be designed for a combined static and seismically induced differential settlement of less than 1.25 inch over a distance of 40 feet.

8.9.4 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

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. An allowable coefficient of friction of 0.40 may be used with the dead load forces in the undisturbed alluvial soils and newly compacted engineered fill.

8.10.2 Passive earth pressure for the sides of foundations poured against undisturbed alluvium may be computed as an equivalent fluid having a density of 300 pounds per cubic foot (pcf) with a maximum earth pressure of 3,000 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.11 Exterior Concrete Flatwork

8.11.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 8.11.1. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 8.11.1
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Steel Reinforcement* Options	Minimum Thickness
EI $\leq$ 50	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 18 inches on center, Both Directions	

*In excess of 8 feet square.

8.11.2 The subgrade soil should be properly moisturized and compacted prior to the placement of steel and concrete. The subgrade soil should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557.

8.11.3 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

8.11.4 Concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

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

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

8.12 Retaining Walls

8.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 6 feet. In the event that walls significantly higher than 6 feet are planned, Geocon should be contacted for additional recommendations.

8.12.2 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 30 pcf.

8.12.3 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 54 pcf.

8.12.4 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils or engineered fill derived from onsite soils. If import soil will be used to backfill proposed retaining walls, revised earth pressures may be required to account for the geotechnical properties of the import soil used as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

8.12.5 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

8.12.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

8.12.7 Retaining wall foundations may be supported on conventional foundations deriving support on a minimum of 24 inches of newly placed engineered fill.

8.12.8 Retaining wall footings may be designed for an allowable bearing capacity of 2,500 psf, and should be a minimum of 12 inches in width and 18 inches in depth below the lowest adjacent grade.

8.12.9 The soil bearing pressure above may be increased by 250 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 3,000 psf. The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

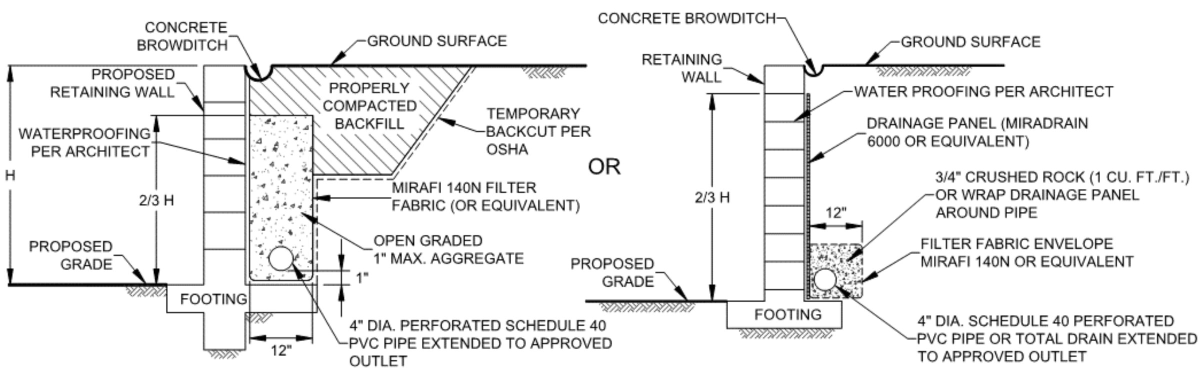
8.12.10 Reinforcement for retaining wall footings should be designed by the project structural engineer.

8.12.11 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

8.12.12 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.12.13 Retaining walls should be designed to ensure stability against overturning sliding, and excessive foundation pressure. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, it is not necessary to consider active pressure on the keyway.

8.12.14 Drainage openings through the base of the wall (weep holes) should not be used where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. The retaining wall should be properly drained as shown in the Typical Retaining Wall Drainage Detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon should be contacted for additional recommendations.



Typical Retaining Wall Drainage Detail

8.13 Preliminary Pavement Recommendations

8.13.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We have assumed an R-Value of 20 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 8.13.1 presents the preliminary flexible pavement sections.

TABLE 8.13.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	20	3	7
Driveways for automobiles and light-duty vehicles	5.5	20	3	9
Medium truck traffic areas	6.0	20	3.5	10
Driveways for heavy truck traffic	7.0	20	4	12

8.13.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

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

8.13.4 The base thickness can be reduced if a reinforcement geogrid is used during the installation of the pavement. Geocon should be contact for additional recommendations if alternate design parameters are requested.

8.13.5 A rigid Portland cement concrete (PCC) pavement section should be placed in heavy truck areas, driveway aprons, and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 8.13.5.

TABLE 8.13.5
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	150 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	C and D
Average daily truck traffic, ADTT	100 and 700

8.13.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.13.6.

TABLE 8.13.6
RIGID PAVEMENT RECOMMENDATIONS

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

8.13.7 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 near optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch).

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

8.13.9 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.

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

8.14 Temporary Excavations

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

8.14.2 Excavations of up to 12 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.

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

8.14.4 Where sufficient space is available, temporary unsurcharged embankments in soil may be sloped back at a uniform 1.5:1 (h:v) slope gradient or flatter. Excavations in bedrock may be steepened per Cal OSHA requirements. Note, a uniform slope does not have a vertical portion.

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

8.14.6 Where temporary constructions slopes 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 slopes 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 in accordance with OSHA regulations 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.

8.15 Site Drainage and Moisture Protection

8.15.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

8.15.2 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 can infiltrate the soil for prolonged periods of time.

8.15.3 Storm water mitigation systems should be offset a minimum of 20 feet from the outside edge of structural footings, so as to reduce the occurrence of water migrating within the structures' load projection.

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

8.15.5 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. Downgradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration.

8.16 Grading and Foundation Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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 this investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that expected herein, Geocon West, Inc. 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 West, Inc.

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.

The requirements for concrete and steel reinforcement presented in this report are preliminary recommendations from a geotechnical perspective. The Structural Engineer should provide the final recommendations for structural design of concrete and steel reinforcement for foundation systems, floor slabs, exterior concrete, or other systems where concrete and steel reinforcement are utilized, in accordance with the latest version of applicable codes.

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.

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 7-16, 2019, *Minimum Design Loads for Buildings and Other Structures*.

4. California Building Standards Commission, 2019, *California Building Code (CBC)*, California Code of Regulations Title 24, Part 2.

5. California Department of Transportation (Caltrans), 2021, Division of Engineering Services, Materials Engineering and Testing Services, *Corrosion Guidelines, Version 3.2*, dated May.

6. California Department of Water Resources, *Water Data Library* website, https://wdl.water.ca.gov/ ; accessed February 2022.

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

8. California Geologic Survey, 2008, Special Publication 117A, *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Revised and Re-adopted September 11.

9. Morton, D.M. and F.H. Weber, Jr., 2003, *Preliminary Geologic Map of the Elsinore 7.5' Quadrangle, Riverside County, California*, USGS Open-File Report 03-281, version 1.0, Scale 1:24,000.

10. Google Earth Pro, 2021, accessed February 2022.

11. Harden, Deborah R., 1998, *California Geology*, Prentice Hall Publishing.

12. Jennings, C. W., 2010, California Division of Mines and Geology, *Fault Activity Map of California and Adjacent Areas*, California Geologic Data Map Series Map No. 6.

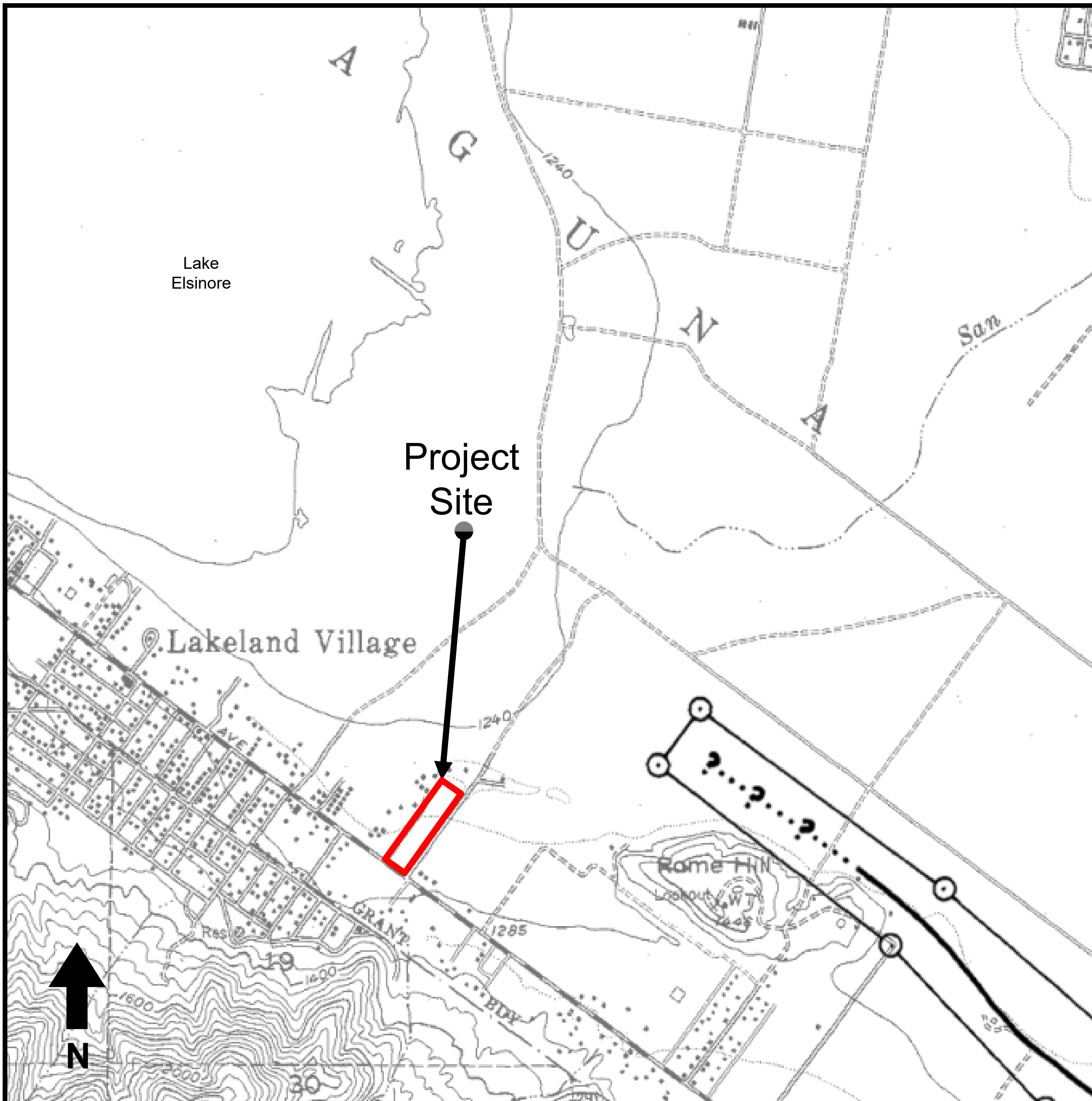
13. Legg, M. R., J. C. Borrero, and C. E. Synolakis, *Evaluation of Tsunami Risk to Southern California Coastal Cities*, 2002 NEHRP Professional Fellowship Report, dated January 2003.

14. OSPD, 2018, *Seismic Design Maps, https://seismicmaps.org* Accessed February 2022.

15. Public Works Standards, Inc., 2021, *Standard Specifications for Public Works Construction "Greenbook,"* Published by BNi Building News.

16. Riverside County, *Map My County*, accessed February 2022.

17. Riverside County Flood Control and Water Conservation District, 2011, *Design Handbook for Low Impact Development Best Management Practices*, dated September.



1 INCH = 1500 FEET
(SCALE IS APPROXIMATE)

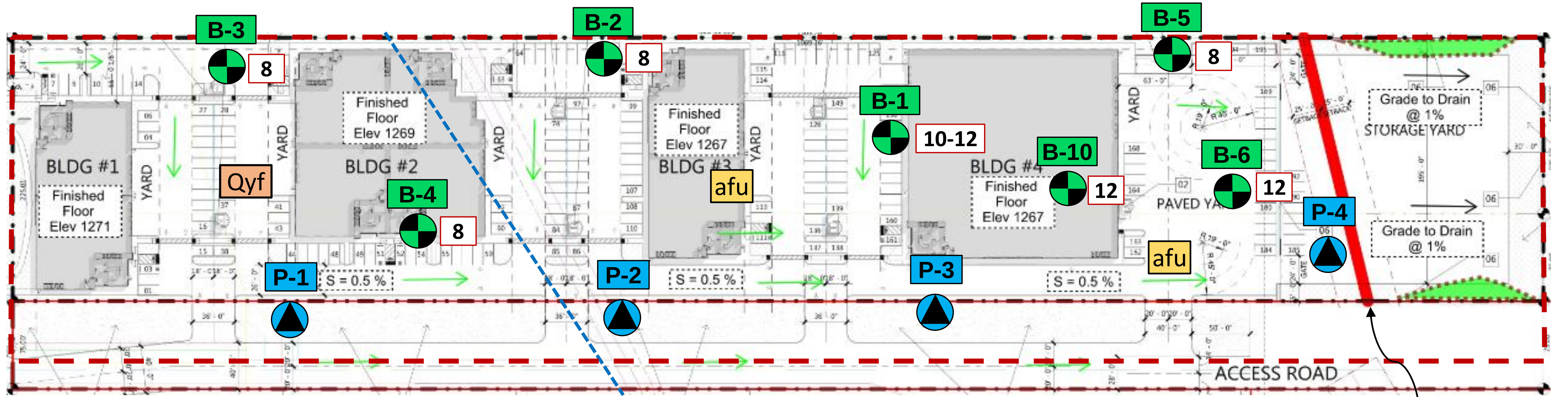
SOURCE: CGS, 1980, Elsinore AP Quadrangle.

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

GRAND AVE. AT KATHRYN WAY
LAKE ELSINORE, CALIFORNIA

AMO			FEB. 2022	PROJECT NO. T2719-22-02	FIG. 1



SOURCE: Grant Becklund Civil Engineer, 2022, *Preliminary Grading Plan PA 2021-19*, dated January.

Willard Fault with building setback zone

GEOCON LEGEND

Locations are approximate

B-10 ……. GEOTECHNICAL BORING LOCATION, 2016

12 ……. ESTIMATED REMOVAL DEPTH (ft)

P-4 ……. PERCOLATION TEST LOCATION

……. PROJECT LIMITS

afu ……... UNDOCUMENTED FILL

Qyf ……... ALLUVIAL FAN DEPOSITS

……... GEOLOGIC CONTACT

0 25' 50'

SCALE: 1" = 25'

N

GEOCON WEST, INC. GEOTECHNICAL ENVIRONMENTAL MATERIALS 41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562 PHONE 951-304-2300 FAX 951-304-2392			GEOLOGIC MAP APN 371.150.001 & 371.150.002 GRAND AVENUE AT KATHRYN WAY LAKE ELSINORE, CALIFORNIA		
LCW			FEBRUARY 2022	PROJECT NO. T2719-22-02	FIG. 2

GEOCON

Client : Lake Resort Apartments
File No. : T2719-22-02
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.49
Peak Horiz. Acceleration PGA_M (g):	0.877
2/3 PGA_M (g):	0.585
Calculated Mag.Wtg.Factor:	0.694
Historic High Groundwater:	5.0
Groundwater Depth During Exploration:	26.5

By Thomas F. Blake (1994-1996)

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	19.0	5.0	1		95	2.000	49.2	125.0	Infin.	0.998	0.263	--
2.0	125.0	0	19.0	5.0	1		95	2.000	49.2	125.0	Infin.	0.993	0.262	--
3.0	125.0	0	19.0	5.0	1		95	2.000	49.2	125.0	Infin.	0.989	0.261	--
4.0	125.0	0	19.0	5.0	1		95	2.000	49.2	125.0	Infin.	0.984	0.260	--
5.0	125.0	1	19.0	5.0	1		95	1.927	47.4	62.6	Infin.	0.979	0.274	Non-Liq.
6.0	125.0	1	19.0	5.0	1		95	1.743	42.8	62.6	Infin.	0.975	0.298	Non-Liq.
7.0	125.0	1	12.0	7.5	1	36	73	1.603	31.9	62.6	Infin.	0.970	0.317	Non-Liq.
8.0	125.0	1	12.0	7.5	1	36	73	1.492	30.2	62.6	Infin.	0.966	0.332	Non-Liq.
9.0	125.0	1	12.0	7.5	1	36	73	1.402	28.8	62.6	0.375	0.961	0.345	1.09
10.0	125.0	1	12.0	7.5	1	36	73	1.326	27.6	62.6	0.338	0.957	0.355	0.95
11.0	125.0	1	12.0	7.5	1	36	73	1.261	26.6	62.6	0.315	0.952	0.364	0.87
12.0	125.0	1	6.0	12.5	1	36	49	1.205	16.4	62.6	0.178	0.947	0.371	0.48
13.0	125.0	1	6.0	12.5	1	36	49	1.156	16.0	62.6	0.174	0.943	0.377	0.46
14.0	125.0	1	6.0	12.5	1	36	49	1.112	15.6	62.6	0.170	0.938	0.382	0.45
15.0	125.0	1	10.0	17.5	1	42	60	1.073	22.8	62.6	0.253	0.934	0.386	0.66
16.0	125.0	1	10.0	17.5	1	42	60	1.038	22.3	62.6	0.246	0.929	0.389	0.63
17.0	125.0	1	10.0	17.5	1	42	60	1.006	21.8	62.6	0.240	0.925	0.392	0.61
18.0	125.0	1	10.0	17.5	1	42	60	0.977	21.4	62.6	0.235	0.920	0.395	0.59
19.0	125.0	1	10.0	17.5	1	42	60	0.950	21.0	62.6	0.230	0.915	0.397	0.58
20.0	125.0	1	10.0	17.5	1	42	60	0.926	20.7	62.6	0.226	0.911	0.398	0.57
21.0	125.0	1	10.0	17.5	1	42	60	0.903	20.3	62.6	0.222	0.906	0.400	0.55
22.0	125.0	1	10.0	17.5	1	42	60	0.881	20.0	62.6	0.218	0.902	0.401	0.54
23.0	125.0	1	7.0	22.5	1	42	48	0.862	16.7	62.6	0.181	0.897	0.401	0.45
24.0	125.0	1	7.0	22.5	1	42	48	0.843	16.4	62.6	0.179	0.893	0.402	0.45
25.0	125.0	1	22.0	27.5	1	35	82	0.826	37.7	62.6	Infin.	0.888	0.402	Non-Liq.
26.0	125.0	1	22.0	27.5	1	35	82	0.809	37.1	62.6	Infin.	0.883	0.402	Non-Liq.
27.0	125.0	1	22.0	27.5	1	35	82	0.798	36.6	62.6	Infin.	0.879	0.402	Non-Liq.
28.0	125.0	1	22.0	27.5	1	35	82	0.790	36.4	62.6	Infin.	0.874	0.402	Non-Liq.
29.0	125.0	1	22.0	27.5	1	35	82	0.783	36.1	62.6	Infin.	0.870	0.402	Non-Liq.
30.0	125.0	1	22.0	27.5	1	35	82	0.776	35.8	62.6	Infin.	0.865	0.402	Non-Liq.
31.0	125.0	1	22.0	27.5	1	35	82	0.769	35.6	62.6	Infin.	0.861	0.401	Non-Liq.
32.0	125.0	1	22.0	27.5	1	35	82	0.762	35.3	62.6	Infin.	0.856	0.400	Non-Liq.
33.0	125.0	1	9.0	32.5	1	39	50	0.756	18.7	62.6	0.201	0.851	0.400	0.50
34.0	125.0	1	9.0	32.5	1	39	50	0.749	18.6	62.6	0.200	0.847	0.399	0.50
35.0	125.0	1	26.0	37.5	1	38	83	0.743	40.3	62.6	Infin.	0.842	0.398	Non-Liq.
36.0	125.0	1	26.0	37.5	1	38	83	0.737	40.1	62.6	Infin.	0.838	0.397	Non-Liq.
37.0	125.0	1	26.0	37.5	1	38	83	0.731	39.8	62.6	Infin.	0.833	0.396	Non-Liq.
38.0	125.0	1	26.0	37.5	1	38	83	0.725	39.5	62.6	Infin.	0.829	0.395	Non-Liq.
39.0	125.0	1	26.0	37.5	1	38	83	0.720	39.3	62.6	Infin.	0.824	0.393	Non-Liq.
40.0	125.0	1	22.0	40.0	1	40	75	0.714	34.1	62.6	Infin.	0.819	0.392	Non-Liq.
41.0	125.0	1	22.0	40.0	1	40	75	0.709	33.9	62.6	Infin.	0.815	0.391	Non-Liq.
42.0	125.0	1	32.0	42.5	1		89	0.703	38.8	62.6	Infin.	0.810	0.389	Non-Liq.
43.0	125.0	1	32.0	42.5	1		89	0.698	38.5	62.6	Infin.	0.806	0.388	Non-Liq.
44.0	125.0	1	32.0	42.5	1		89	0.693	38.3	62.6	Infin.	0.801	0.387	Non-Liq.
45.0	125.0	1	32.0	42.5	1		89	0.688	38.0	62.6	Infin.	0.797	0.385	Non-Liq.
46.0	125.0	1	32.0	42.5	1		89	0.683	37.7	62.6	Infin.	0.792	0.384	Non-Liq.
47.0	125.0	1	62.0	47.5	1		120	0.679	72.6	62.6	Infin.	0.787	0.382	Non-Liq.
48.0	125.0	1	62.0	47.5	1		120	0.674	72.1	62.6	Infin.	0.783	0.380	Non-Liq.
49.0	125.0	1	62.0	47.5	1		120	0.669	71.6	62.6	Infin.	0.778	0.379	Non-Liq.
50.0	125.0	1	62.0	47.5	1		120	0.665	71.1	62.6	Infin.	0.774	0.377	Non-Liq.

GEOCON

Client : Lake Resort Apartments
File No. : T2719-22-02
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.49
PGAM (g):	0.877
2/3 PGAM (g):	0.58
Calculated Mag.Wtg.Factor:	0.694
Historic High Groundwater:	5.0
Groundwater @ Exploration:	26.5

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS	Tav/σ'o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1.0	19	125	0.031	0.031	95	49	0.380	--	0.00	Grading
2.0	19	125	0.094	0.094	95	49	0.380	--	0.00	Grading
3.0	19	125	0.156	0.156	95	49	0.380	--	0.00	Grading
4.0	19	125	0.219	0.219	95	49	0.380	--	0.00	Grading
5.0	19	125	0.281	0.266	95	47	0.403	Non-Liq.	0.00	Grading
6.0	19	125	0.344	0.297	95	43	0.440	Non-Liq.	0.00	Grading
7.0	12	125	0.406	0.328	73	32	0.471	Non-Liq.	0.00	0.00
8.0	12	125	0.469	0.360	73	30	0.496	Non-Liq.	0.00	0.00
9.0	12	125	0.531	0.391	73	29	0.517	1.09	0.75	0.09
10.0	12	125	0.594	0.422	73	28	0.535	0.95	0.75	0.09
11.0	12	125	0.656	0.453	73	27	0.550	0.87	1.10	0.13
12.0	6	125	0.719	0.485	49	16	0.564	0.48	1.70	0.20
13.0	6	125	0.781	0.516	49	16	0.576	0.46	1.70	0.20
14.0	6	125	0.844	0.547	49	16	0.586	0.45	1.70	0.20
15.0	10	125	0.906	0.579	60	23	0.595	0.66	1.30	0.16
16.0	10	125	0.969	0.610	60	22	0.604	0.63	1.40	0.17
17.0	10	125	1.031	0.641	60	22	0.611	0.61	1.40	0.17
18.0	10	125	1.094	0.673	60	21	0.618	0.59	1.40	0.17
19.0	10	125	1.156	0.704	60	21	0.625	0.58	1.40	0.17
20.0	10	125	1.219	0.735	60	21	0.630	0.57	1.40	0.17
21.0	10	125	1.281	0.766	60	20	0.636	0.55	1.40	0.17
22.0	10	125	1.344	0.798	60	20	0.640	0.54	1.40	0.17
23.0	7	125	1.406	0.829	48	17	0.645	0.45	1.70	0.20
24.0	7	125	1.469	0.860	48	16	0.649	0.45	1.70	0.20
25.0	22	125	1.531	0.892	82	38	0.653	Non-Liq.	0.00	0.00
26.0	22	125	1.594	0.923	82	37	0.657	Non-Liq.	0.00	0.00
27.0	22	125	1.656	0.954	82	37	0.660	Non-Liq.	0.00	0.00
28.0	22	125	1.719	0.986	82	36	0.663	Non-Liq.	0.00	0.00
29.0	22	125	1.781	1.017	82	36	0.666	Non-Liq.	0.00	0.00
30.0	22	125	1.844	1.048	82	36	0.669	Non-Liq.	0.00	0.00
31.0	22	125	1.906	1.079	82	36	0.671	Non-Liq.	0.00	0.00
32.0	22	125	1.969	1.111	82	35	0.674	Non-Liq.	0.00	0.00
33.0	9	125	2.031	1.142	50	19	0.676	0.50	1.60	0.19
34.0	9	125	2.094	1.173	50	19	0.678	0.50	1.60	0.19
35.0	26	125	2.156	1.205	83	40	0.681	Non-Liq.	0.00	0.00
36.0	26	125	2.219	1.236	83	40	0.683	Non-Liq.	0.00	0.00
37.0	26	125	2.281	1.267	83	40	0.684	Non-Liq.	0.00	0.00
38.0	26	125	2.344	1.299	83	40	0.686	Non-Liq.	0.00	0.00
39.0	26	125	2.406	1.330	83	39	0.688	Non-Liq.	0.00	0.00
40.0	22	125	2.469	1.361	75	34	0.690	Non-Liq.	0.00	0.00
41.0	22	125	2.531	1.392	75	34	0.691	Non-Liq.	0.00	0.00
42.0	32	125	2.594	1.424	89	39	0.693	Non-Liq.	0.00	0.00
43.0	32	125	2.656	1.455	89	39	0.694	Non-Liq.	0.00	0.00
44.0	32	125	2.719	1.486	89	38	0.695	Non-Liq.	0.00	0.00
45.0	32	125	2.781	1.518	89	38	0.697	Non-Liq.	0.00	0.00
46.0	32	125	2.844	1.549	89	38	0.698	Non-Liq.	0.00	0.00
47.0	62	125	2.906	1.580	120	73	0.699	Non-Liq.	0.00	0.00
48.0	62	125	2.969	1.612	120	72	0.700	Non-Liq.	0.00	0.00
49.0	62	125	3.031	1.643	120	72	0.702	Non-Liq.	0.00	0.00
50.0	62	125	3.094	1.674	120	71	0.703	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 3.0 INCHES

GEOCON

Client : Lake Resort Apartments
File No. : T2719-22-02
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.71
Peak Horiz. Acceleration PGA_M (g):	0.877
Calculated Mag.Wtg.Factor:	1.078
Historic High Groundwater:	5.0
Groundwater Depth During Exploration:	26.5

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	19.0	5.0	1		95	2.000	49.2	125.0	Infin.	0.998	0.613	--
2.0	125.0	0	19.0	5.0	1		95	2.000	49.2	125.0	Infin.	0.993	0.611	--
3.0	125.0	0	19.0	5.0	1		95	2.000	49.2	125.0	Infin.	0.989	0.608	--
4.0	125.0	0	19.0	5.0	1		95	2.000	49.2	125.0	Infin.	0.984	0.605	--
5.0	125.0	1	19.0	5.0	1		95	1.927	47.4	62.6	Infin.	0.979	0.638	Non-Liq.
6.0	125.0	1	19.0	5.0	1		95	1.743	42.8	62.6	Infin.	0.975	0.694	Non-Liq.
7.0	125.0	1	12.0	7.5	1	36	73	1.603	31.9	62.6	Infin.	0.970	0.738	Non-Liq.
8.0	125.0	1	12.0	7.5	1	36	73	1.492	30.2	62.6	Infin.	0.966	0.774	Non-Liq.
9.0	125.0	1	12.0	7.5	1	36	73	1.402	28.8	62.6	0.375	0.961	0.803	0.47
10.0	125.0	1	12.0	7.5	1	36	73	1.326	27.6	62.6	0.338	0.957	0.827	0.41
11.0	125.0	1	12.0	7.5	1	36	73	1.261	26.6	62.6	0.315	0.952	0.847	0.37
12.0	125.0	1	6.0	12.5	1	36	49	1.205	16.4	62.6	0.178	0.947	0.864	0.21
13.0	125.0	1	6.0	12.5	1	36	49	1.156	16.0	62.6	0.174	0.943	0.878	0.20
14.0	125.0	1	6.0	12.5	1	36	49	1.112	15.6	62.6	0.170	0.938	0.889	0.19
15.0	125.0	1	10.0	17.5	1	42	60	1.073	22.8	62.6	0.253	0.934	0.899	0.28
16.0	125.0	1	10.0	17.5	1	42	60	1.038	22.3	62.6	0.246	0.929	0.907	0.27
17.0	125.0	1	10.0	17.5	1	42	60	1.006	21.8	62.6	0.240	0.925	0.914	0.26
18.0	125.0	1	10.0	17.5	1	42	60	0.977	21.4	62.6	0.235	0.920	0.920	0.26
19.0	125.0	1	10.0	17.5	1	42	60	0.950	21.0	62.6	0.230	0.915	0.925	0.25
20.0	125.0	1	10.0	17.5	1	42	60	0.926	20.7	62.6	0.226	0.911	0.928	0.24
21.0	125.0	1	10.0	17.5	1	42	60	0.903	20.3	62.6	0.222	0.906	0.931	0.24
22.0	125.0	1	10.0	17.5	1	42	60	0.881	20.0	62.6	0.218	0.902	0.934	0.23
23.0	125.0	1	7.0	22.5	1	42	48	0.862	16.7	62.6	0.181	0.897	0.936	0.19
24.0	125.0	1	7.0	22.5	1	42	48	0.843	16.4	62.6	0.179	0.893	0.937	0.19
25.0	125.0	1	22.0	27.5	1	35	82	0.826	37.7	62.6	Infin.	0.888	0.938	Non-Liq.
26.0	125.0	1	22.0	27.5	1	35	82	0.809	37.1	62.6	Infin.	0.883	0.938	Non-Liq.
27.0	125.0	1	22.0	27.5	1	35	82	0.798	36.6	62.6	Infin.	0.879	0.938	Non-Liq.
28.0	125.0	1	22.0	27.5	1	35	82	0.790	36.4	62.6	Infin.	0.874	0.937	Non-Liq.
29.0	125.0	1	22.0	27.5	1	35	82	0.783	36.1	62.6	Infin.	0.870	0.937	Non-Liq.
30.0	125.0	1	22.0	27.5	1	35	82	0.776	35.8	62.6	Infin.	0.865	0.936	Non-Liq.
31.0	125.0	1	22.0	27.5	1	35	82	0.769	35.6	62.6	Infin.	0.861	0.934	Non-Liq.
32.0	125.0	1	22.0	27.5	1	35	82	0.762	35.3	62.6	Infin.	0.856	0.933	Non-Liq.
33.0	125.0	1	9.0	32.5	1	39	50	0.756	18.7	62.6	0.201	0.851	0.931	0.22
34.0	125.0	1	9.0	32.5	1	39	50	0.749	18.6	62.6	0.200	0.847	0.929	0.22
35.0	125.0	1	26.0	37.5	1	38	83	0.743	40.3	62.6	Infin.	0.842	0.927	Non-Liq.
36.0	125.0	1	26.0	37.5	1	38	83	0.737	40.1	62.6	Infin.	0.838	0.925	Non-Liq.
37.0	125.0	1	26.0	37.5	1	38	83	0.731	39.8	62.6	Infin.	0.833	0.922	Non-Liq.
38.0	125.0	1	26.0	37.5	1	38	83	0.725	39.5	62.6	Infin.	0.829	0.919	Non-Liq.
39.0	125.0	1	26.0	37.5	1	38	83	0.720	39.3	62.6	Infin.	0.824	0.917	Non-Liq.
40.0	125.0	1	22.0	40.0	1	40	75	0.714	34.1	62.6	Infin.	0.819	0.914	Non-Liq.
41.0	125.0	1	22.0	40.0	1	40	75	0.709	33.9	62.6	Infin.	0.815	0.911	Non-Liq.
42.0	125.0	1	32.0	42.5	1		89	0.703	38.8	62.6	Infin.	0.810	0.908	Non-Liq.
43.0	125.0	1	32.0	42.5	1		89	0.698	38.5	62.6	Infin.	0.806	0.904	Non-Liq.
44.0	125.0	1	32.0	42.5	1		89	0.693	38.3	62.6	Infin.	0.801	0.901	Non-Liq.
45.0	125.0	1	32.0	42.5	1		89	0.688	38.0	62.6	Infin.	0.797	0.897	Non-Liq.
46.0	125.0	1	32.0	42.5	1		89	0.683	37.7	62.6	Infin.	0.792	0.894	Non-Liq.
47.0	125.0	1	62.0	47.5	1		120	0.679	72.6	62.6	Infin.	0.787	0.890	Non-Liq.
48.0	125.0	1	62.0	47.5	1		120	0.674	72.1	62.6	Infin.	0.783	0.887	Non-Liq.
49.0	125.0	1	62.0	47.5	1		120	0.669	71.6	62.6	Infin.	0.778	0.883	Non-Liq.
50.0	125.0	1	62.0	47.5	1		120	0.665	71.1	62.6	Infin.	0.774	0.879	Non-Liq.

Figure 5

GEOCON

Client : Lake Resort Apartments
File No. : T2719-22-02
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.71
PGA_M (g):	0.877
Calculated Mag.Wtg.Factor:	1.078
Historic High Groundwater:	5.0
Groundwater @ Exploration:	26.5

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS	τ_{av}/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1.0	19	125	0.031	0.031	95	49	0.570	--	0.00	Grading
2.0	19	125	0.094	0.094	95	49	0.570	--	0.00	Grading
3.0	19	125	0.156	0.156	95	49	0.570	--	0.00	Grading
4.0	19	125	0.219	0.219	95	49	0.570	--	0.00	Grading
5.0	19	125	0.281	0.266	95	47	0.604	Non-Liq.	0.00	Grading
6.0	19	125	0.344	0.297	95	43	0.660	Non-Liq.	0.00	Grading
7.0	12	125	0.406	0.328	73	32	0.706	Non-Liq.	0.00	0.00
8.0	12	125	0.469	0.360	73	30	0.743	Non-Liq.	0.00	0.00
9.0	12	125	0.531	0.391	73	29	0.775	0.47	0.75	0.09
10.0	12	125	0.594	0.422	73	28	0.802	0.41	0.75	0.09
11.0	12	125	0.656	0.453	73	27	0.825	0.37	1.10	0.13
12.0	6	125	0.719	0.485	49	16	0.845	0.21	1.70	0.20
13.0	6	125	0.781	0.516	49	16	0.863	0.20	1.70	0.20
14.0	6	125	0.844	0.547	49	16	0.879	0.19	1.70	0.20
15.0	10	125	0.906	0.579	60	23	0.893	0.28	1.30	0.16
16.0	10	125	0.969	0.610	60	22	0.905	0.27	1.40	0.17
17.0	10	125	1.031	0.641	60	22	0.917	0.26	1.40	0.17
18.0	10	125	1.094	0.673	60	21	0.927	0.26	1.40	0.17
19.0	10	125	1.156	0.704	60	21	0.936	0.25	1.40	0.17
20.0	10	125	1.219	0.735	60	21	0.945	0.24	1.40	0.17
21.0	10	125	1.281	0.766	60	20	0.953	0.24	1.40	0.17
22.0	10	125	1.344	0.798	60	20	0.960	0.23	1.40	0.17
23.0	7	125	1.406	0.829	48	17	0.967	0.19	1.70	0.20
24.0	7	125	1.469	0.860	48	16	0.973	0.19	1.70	0.20
25.0	22	125	1.531	0.892	82	38	0.979	Non-Liq.	0.00	0.00
26.0	22	125	1.594	0.923	82	37	0.984	Non-Liq.	0.00	0.00
27.0	22	125	1.656	0.954	82	37	0.989	Non-Liq.	0.00	0.00
28.0	22	125	1.719	0.986	82	36	0.994	Non-Liq.	0.00	0.00
29.0	22	125	1.781	1.017	82	36	0.999	Non-Liq.	0.00	0.00
30.0	22	125	1.844	1.048	82	36	1.003	Non-Liq.	0.00	0.00
31.0	22	125	1.906	1.079	82	36	1.007	Non-Liq.	0.00	0.00
32.0	22	125	1.969	1.111	82	35	1.010	Non-Liq.	0.00	0.00
33.0	9	125	2.031	1.142	50	19	1.014	0.22	1.60	0.19
34.0	9	125	2.094	1.173	50	19	1.017	0.22	1.60	0.19
35.0	26	125	2.156	1.205	83	40	1.020	Non-Liq.	0.00	0.00
36.0	26	125	2.219	1.236	83	40	1.023	Non-Liq.	0.00	0.00
37.0	26	125	2.281	1.267	83	40	1.026	Non-Liq.	0.00	0.00
38.0	26	125	2.344	1.299	83	40	1.029	Non-Liq.	0.00	0.00
39.0	26	125	2.406	1.330	83	39	1.031	Non-Liq.	0.00	0.00
40.0	22	125	2.469	1.361	75	34	1.034	Non-Liq.	0.00	0.00
41.0	22	125	2.531	1.392	75	34	1.036	Non-Liq.	0.00	0.00
42.0	32	125	2.594	1.424	89	39	1.039	Non-Liq.	0.00	0.00
43.0	32	125	2.656	1.455	89	39	1.041	Non-Liq.	0.00	0.00
44.0	32	125	2.719	1.486	89	38	1.043	Non-Liq.	0.00	0.00
45.0	32	125	2.781	1.518	89	38	1.045	Non-Liq.	0.00	0.00
46.0	32	125	2.844	1.549	89	38	1.047	Non-Liq.	0.00	0.00
47.0	62	125	2.906	1.580	120	73	1.048	Non-Liq.	0.00	0.00
48.0	62	125	2.969	1.612	120	72	1.050	Non-Liq.	0.00	0.00
49.0	62	125	3.031	1.643	120	72	1.052	Non-Liq.	0.00	0.00
50.0	62	125	3.094	1.674	120	71	1.053	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 3.0 INCHES

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We drilled four percolation borings to depths of four feet on the site on February 2, 2022 and percolation testing was performed on February 3, 2022. Prior field investigations were performed on May 12 and August 30, 2016. The borings were excavated with an 8-inch hollow-stem auger drill to a maximum depth of 51.8 feet. Bulk samples of disturbed soils and ring samples of in-situ soils were transported to our laboratory for testing. Each boring was backfilled with the cuttings generated during excavation.

We obtained soil samples from the borings during our subsurface exploration using a California sampler. The sampler is composed of steel and is driven to obtain relatively undisturbed samples. The sampler was driven up to 18 inches. The sampler is connected to A rods and driven into the bottom of the boring using a 140-pound hammer. The California sampler has an inside diameter of 2.5 inches and an outside diameter of 3 inches. Rings are placed inside the sampler that are 2.4 inches in diameter and 1 inch in height. We obtained ring samples at appropriate intervals, placed them in moisture-tight containers, and transported them to the laboratory for testing. We also utilized a standard penetration sampler (SPT) alternately with the California sampler within one deep boring on the site to provide standard penetration resistance values necessary for liquefaction analysis. SPT samples were placed in plastic bags and transported to the laboratory. Disturbed bulk samples were also collected and transported back to the laboratory for testing.

Blow counts were recorded for every 6 inches the sampler was driven. The penetration resistances shown on the boring logs are shown in terms of blows per foot. These values are not to be taken as N-values as adjustments have not been applied.

The soil conditions encountered in the excavations were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the bore holes are presented on Figures A-1 through A-4 and previous geotechnical borings within the proposed development area of the site (Borings 1 through 6, and 10). The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the boreholes are indicated the *Geotechnical Map*, Figure 2. We estimated elevations shown on the boring logs either from a topographic map or by using a benchmark.

BORING P-1

ELEV. (MSL.) **1276** DATE COMPLETED **2/2/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **L. BATTIATO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 5				SM	**ALLUVIAL FAN DEPOSITS (Qyf)** Silty SAND, loose, moist, brown			
5 – 6.5	P-1@5'			SM	Silty SAND, loose, dry, brown; some pea gravel	19		

Total Depth = 6' 6" feet
Perc set at 5'
Presaturated with 5 gal H2O
Perc performed on 02/03/2022
Pipe removed and backfilled with native soil

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

T2719-22-02 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.

BORING P-2

ELEV. (MSL.) 1271 DATE COMPLETED 2/2/2022

EQUIPMENT HOLLOW STEM AUGER BY: L. BATTIATO

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	**UNDOCUMENTED FILL (afu)** Silty SAND, loose, moist, very dark brown			
2								
4					-Becomes medium yellow brown			
6	P-2@5'			SM	Silty SAND, dry, yellow brown; trace pea gravel	19		

Total Depth = 6' 6" feet
Perc set at 5'
Presaturated with 5 gal H2O
Perc performed on 02/03/2022
Pipe removed and backfilled with native soil

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

T2719-22-02 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 P-3

ELEV. (MSL.) **1266** DATE COMPLETED **2/2/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **L. BATTIATO**

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	**UNDOCUMENTED FILL (afu)** Silty SAND, loose, moist, very dark brown; organic odor			
2								
4					-Becomes medium brown at 4'			
6	P-3@5'			SM	Silty SAND, dry, yellow brown; some pea gravel	21		

Total Depth = 6' 6" feet
Perc set at 5'
Presaturated with 5 gal H2O
Perc performed on 02/03/2022
Pipe removed and backfilled with native soil

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

T2719-22-02 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. T2719-22-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-4 ELEV. (MSL.) 1271 DATE COMPLETED 2/2/2022 EQUIPMENT HOLLOW STEM AUGER BY: L. BATTIATO	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				ML	**UNDOCUMENTED FILL (afu)** SILT, loose, moist, dark brown; medium sand			
2								
4								
6	P-4@5'			ML	SILT, firm, damp, olive brown; porous, carbonate stringers	11		
					Total Depth = 6' 6" feet Perc set at 5' Presaturated with 5 gal H2O Perc performed on 02/03/2022 Pipe removed and backfilled with native soil			

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

T2719-22-02 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:	Builder's Max Rome Hill	**Project No.:**	T2719-22-02
Test Hole No.:	P-1	**Date Excavated:**	2/2/2022
Length of Test Pipe:	48.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	2/2/2022
Depth of Test Hole:	48.0 inches	**Perc Test Date:**	2/3/2022
Check for Sandy Soil Criteria Tested by:	LW	**Percolation Tested by:**	LW

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 (in)	Percolation Rate (min/inch)
1	8:00 AM 8:25 AM	25	25	-16.2	-4.7	11.5	2.2
2	8:25 AM 8:50 AM	25	50	-34.4	-21.0	13.4	1.9

Soil Criteria: Sandy

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	8:50 PM 9:00 PM	10	10	21.0	17.5	3.5	2.9
2	9:00 PM 9:10 PM	10	20	17.5	14.5	3.0	3.3
3	9:10 PM 9:20 PM	10	30	14.5	11.9	2.6	3.8
4	9:20 PM 9:30 PM	10	40	11.9	7.1	4.8	2.1
5	9:30 PM 9:40 PM	10	50	6.8	2.4	4.4	2.3
6	9:40 PM 9:50 PM	10	60	33.0	28.8	4.2	2.4

Infiltration Rate (in/hr):	1.5
Radius of test hole (in):	4
Average Head (in):	30.9

Figure A-5

PERCOLATION TEST REPORT

Project Name:	Builder's max Rome hill	**Project No.:**	T2962-22-01
Test Hole No.:	P-2	**Date Excavated:**	2/1/2022
Length of Test Pipe:	48.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	2/3/2022
Depth of Test Hole:	48.0 inches	**Perc Test Date:**	2/4/2022
Check for Sandy Soil Criteria Tested by:	LW	**Percolation Tested by:**	LW

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 (in)	Percolation Rate (min/inch)
1	12:00 AM 8:27 AM	25	25	19.6	16.7	2.9	8.7
2	8:27 AM 8:52 AM	25	50	16.7	15.0	1.7	14.9

Soil Criteria: Normal

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	8:52 AM 8:52 AM	0.5	30	15.0	12.8	2.2	0.2
2	9:22 AM 9:22 AM	0.5	60	12.8	7.8	5.0	0.1
3	9:52 AM 9:52 AM	0.5	90	28.2	24.7	3.5	0.1
4	10:22 AM 10:22 AM	0.5	120	24.7	22.0	2.8	0.2
5	10:52 AM 10:52 AM	0.5	150	22.0	19.9	2.0	0.2
6	11:22 AM 11:22 AM	0.5	180	19.9	19.1	0.8	0.6
7	11:52 AM 11:52 AM	0.5	210	19.1	14.9	4.2	0.1
8	12:22 PM 12:22 PM	0.5	240	14.9	12.0	2.9	0.0
9	12:52 PM 12:52 PM	0.5	270	12.0	10.4	1.6	0.0
10	1:22 PM 1:22 PM	0.5	300	28.7	26.4	2.3	0.2
11	1:52 PM 1:52 PM	0.5	330	26.4	24.0	2.4	0.0
12	2:22 PM 2:22 PM	0.5	360	24.0	21.8	2.2	0.2

Infiltration Rate (in/hr):	21.4	
Radius of test hole (in):	4	**Figure A-6**
Average Head (in):	22.9	

PERCOLATION TEST REPORT

Project Name:	Builder's Max Rome Hill	**Project No.:**	T2719-22-02
Test Hole No.:	P-3	**Date Excavated:**	2/2/2022
Length of Test Pipe:	48.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	2/2/2022
Depth of Test Hole:	48.0 inches	**Perc Test Date:**	2/3/2022
Check for Sandy Soil Criteria Tested by:	LW	**Percolation Tested by:**	LW

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 (in)	Percolation Rate (min/inch)
1	8:04 AM 8:29 AM	25	25	-21.1	-11.6	9.5	2.6
2	8:29 AM 8:54 AM	25	50	-31.8	-21.0	10.8	2.3

Soil Criteria: Sandy

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	8:54 PM 9:04 PM	10	10	21.0	17.9	3.1	3.2
2	9:04 PM 9:14 PM	10	20	17.9	15.1	2.8	3.6
3	9:14 PM 9:24 PM	10	30	15.1	12.1	3.0	3.3
4	9:24 PM 9:34 PM	10	40	12.1	10.2	1.9	5.2
5	9:34 PM 9:44 PM	10	50	10.2	8.4	1.8	5.6
6	9:44 PM 9:54 PM	10	60	27.0	23.2	3.8	2.6

Infiltration Rate (in/hr):	1.7
Radius of test hole (in):	4
Average Head (in):	25.1

Figure A-7

PERCOLATION TEST REPORT

Project Name:	Builder's max Rome hill	**Project No.:**	T2962-22-01
Test Hole No.:	P-4	**Date Excavated:**	2/1/2022
Length of Test Pipe:	48.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	2/3/2022
Depth of Test Hole:	48.0 inches	**Perc Test Date:**	2/4/2022
Check for Sandy Soil Criteria Tested by:	LW	**Percolation Tested by:**	LW

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 (in)	Percolation Rate (min/inch)
1	12:00 AM 8:31 AM	25	25	16.7	11.6	5.0	5.0
2	8:31 AM 8:56 AM	25	50	36.0	25.8	10.2	2.5

Soil Criteria: Normal

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	8:56 AM 8:56 AM	0.5	30	25.8	22.6	3.2	0.2
2	9:26 AM 9:26 AM	0.5	60	22.6	19.6	3.0	0.2
3	9:56 AM 9:56 AM	0.5	90	19.6	17.6	1.9	0.3
4	10:26 AM 10:26 AM	0.5	120	17.6	15.7	1.9	0.3
5	10:56 AM 10:56 AM	0.5	150	16.3	14.2	2.2	0.2
6	11:26 AM 11:26 AM	0.5	180	14.2	12.7	1.4	0.3
7	11:56 AM 11:56 AM	0.5	210	12.7	11.2	1.6	0.3
8	12:26 PM 12:26 PM	0.5	240	11.2	10.4	0.7	0.0
9	12:56 PM 12:56 PM	0.5	270	25.9	24.7	1.2	0.0
10	1:26 PM 1:26 PM	0.5	300	24.7	23.8	1.0	0.5
11	1:56 PM 1:56 PM	0.5	330	23.8	22.2	1.6	0.0
12	2:26 PM 2:26 PM	0.5	360	22.2	20.9	1.3	0.4

Infiltration Rate (in/hr):	13.5	
Radius of test hole (in):	4	**Figure A-8**
Average Head (in):	21.5	

PROJECT NO. T2719-22-01

BORING B-1

ELEV. (MSL.) **1262** DATE COMPLETED **05/12/2016**

EQUIPMENT **HOLLOW STEM AUGER** BY: **P. Theriault**

DEPTH IN FEET	SAMPLE NO.	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, loose, slightly moist, blackish dark brown; fine to medium sand; some coarse sand; micaceous; trace porosity; root hairs; some weeds			
2	B-1@2.5'			15	101.1	7.0
4	B-1@5.0'		-Becomes medium dense; trace light brown modling	35	122.4	9.3
6						
8	B-1@7.5'	SM	**ALLUVIAL FAN DEPOSITS (Qyf)** Silty SAND, medium dense, moist, strong brown; fine to coarse sand; micaceous; trace fine gravel; non-porous	22		
10	B-1@10.0'			21		
12	B-1@12.5'		-Becomes loose	6		
14	B-1@15.0'		-Becomes medium dense, brown; trace clay	26		
16						
18	B-1@17.5'			10		
20	B-1@20.0'		-Becomes loose, wet; increase in clay	18	109.6	20.7
22	B-1@22.5'		-Becomes grayish brown	7		
24	B-1@25.0'		-Becomes medium dense, saturated, brown; fine to coarse sand; some gravel	38	121.3	14.4
26			-Becomes medium dense			
28	B-1@27.5'			22		

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

T2719-22-01 BORING LOGS.GPJ

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

GEOCON

PROJECT NO. T2719-22-01

BORING B-1

ELEV. (MSL.) **1262** DATE COMPLETED **05/12/2016**

EQUIPMENT **HOLLOW STEM AUGER** BY: **P. Theriault**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B-1@30.0'			SM	**ALLUVIAL FAN DEPOSITS (Qyf)** Silty SAND, medium dense, moist, brown; fine to coarse sand; increase in clay	20		
32	B-1@32.5'					9		
34				SM	**PAUBA FORMATION FANGLOMERATE (Qpfs)** Silty SAND, dense, moist, mottled orange and grayish brown; fine to medium sand; some coarse sand; micaceous; moderately cemented			
36	B-1@35'					62		
38	B-1@37.5'					26		
40	B-1@40'				-Becomes dense, brown; trace clay; micaceous	39		
42	B-1@42.5'				-Becomes reddish brown; no clay	32		
44								
46	B-1@45'				-Becomes wet	50-5"		
48	B-1@47.5'					62		
50	B-1@50'				-Becomes strong brown	50-3"		

Total depth 51 feet, 9 inches
Groundwater encountered at 36 feet, 11 inches. Stabilized at 26 feet, 6 inches
No caving
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 5/12/2016

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

T2719-22-01 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-2

ELEV. (MSL.) **1263** DATE COMPLETED **05/12/2016**

EQUIPMENT **HOLLOW STEM AUGER** BY: **P. Theriault**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL (afu)** Silty SAND, medium dense, slightly moist, dark gray; fine to coarse sand; micaceous; root hairs			
2	B-2@2.5'				-Becomes moist	13	111.1	6.1
4				SM	**ALLUVIAL FAN DEPOSITS (Qyf)** Silty SAND, medium dense, moist, dark gray; fine to coarse sand; micaceous			
	B-2@5'					41		
6					-Poorly developed carbonate stringers			
8	B-2@7.5'				-Becomes brown; decrease in coarse sand	18		
10	B-2@10'				-Becomes reddish brown; no carbonates	22		
12								
14								
	B-2@15'				-Becomes brownish gray; trace gravel	30		
16								

Total depth 16.5 feet
No groundwater encountered
No caving
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 5/12/2016

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

T2719-22-01 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. T2719-22-01

BORING B-3

ELEV. (MSL.) 1270 DATE COMPLETED 05/12/2016

EQUIPMENT HOLLOW STEM AUGER BY: P. Theriault

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-3@0-5'			SM	**ALLUVIAL FAN DEPOSITS (Qyf)** Silty SAND, medium dense, slightly moist, blackish dark brown; fine to coarse sand; micaceous; some weeds			
2	B-3@2.5'				-Becomes brown; root hairs; trace gravel	20	118.2	4.7
4								
6	B-3@5'				-Becomes moist; trace carbonate stringers (poorly developed)	26	116.3	2.5
8	B-3@7.5'				-Becomes reddish brown; carbonate stringers (poorly developed)	43	123.4	5.1
10	B-3@10'					28		
12					-Gravel layer; no carbonates			
	B-3@12.5'					36	112.1	5.3
14								
16	B-3@15'					30		
18								
20	B-3@20'					30		
					-Becomes wet; some clay			
22								
				SM	**PAUBA FORMATION FANGLOMERATE (Qpfs)** Silty SAND, very dense, moist, mottled reddish brown and gray; fine to coarse sand; micaceous; trace fine gravel			
24								
26	B-3@25'					90-11"		

Total depth 26.5 feet
No groundwater encountered
No caving
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 5/12/2016

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

T2719-22-01 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. T2719-22-01

BORING B-4

ELEV. (MSL.) 1268 DATE COMPLETED 05/12/2016

EQUIPMENT HOLLOW STEM AUGER BY: P. Theriault

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ALLUVIAL FAN DEPOSITS (Qyf)** Silty SAND, medium dense, slightly moist, blackish dark brown; fine to coarse sand; micaceous; some weeds			
2					-Becomes brown; weak carbonate stringers; decrease in coarse sand			
	B-4@2.5'					26		
4								
	B-4@5'					41		
6								
					-Becomes moist, reddish brown; carbonate stringers			
8	B-4@7.5'					31		
10	B-4@10'					22		
12					-Becomes strong brown; no carbonates			
	B-4@12.5'					20		
14								
	B-4@15'				-Increase in coarse sand; trace fine gravel (in shoe)	26		
16								
18								
20				SM	**PAUBA FORMATION FANGLOMERATE (Qpfs)** Silty SAND, medium dense, moist, mottled reddish brown and brown; fine to coarse sand; some gravel; micaceous			
	B-4@20'					40		
22								
24								
	B-4@25'				-No recovery	35		
26								

Total depth 26.5 feet
No groundwater encountered
No caving
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 5/12/2016

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

T2719-22-01 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. T2719-22-01

BORING B-5

ELEV. (MSL.) **1261** DATE COMPLETED **05/12/2016**

EQUIPMENT **HOLLOW STEM AUGER** BY: **P. Theriault**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL (afu)** Silty SAND, medium dense, slightly moist, brown; fine to medium sand; chunks of AC			
2	B-5@2.5'					18	104.1	4.3
4				ML	**LACUSTRINE DEPOSITS (Ql)** Sandy SILT, firm, moist, olive; fine sand; micaceous			
6	B-5@5'					8		
8	B-5@7.5'				-Trace weakly cemented carbonate	24	94.1	23.8
10	B-5@10'			SM	**ALLUVIAL FAN DEPOSITS (Qyf)** Silty SAND, loose, moist, brown with light brown mottling; fine to medium sand; some coarse sand; micaceous	11		
12	B-5@12.5'				-Trace poorly developed carbonate	15		
14								
16	B-5@15'				-Becomes olive; some clay	22		

Total depth 16.5 feet
No groundwater encountered
No caving
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 5/12/2016

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

T2719-22-01 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-6

ELEV. (MSL.) 1262 DATE COMPLETED 05/12/2016

EQUIPMENT HOLLOW STEM AUGER BY: P. Theriault

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-6@0-5'			SM	**UNDOCUMENTED FILL (afu)** Silty SAND, medium dense, moist, gray; fine sand; micaceous; some orange staining; debris (AC chunks and rocks) -No debris			
	B-6@2.5'					16	96.8	6.6
	B-6@5'					13	97.6	15.8
	B-6@7.5'			ML	**LACUSTRINE DEPOSITS (Ql)** Sandy SILT, stiff, moist, grayish brown; fine sand; micaceous	14		
10	B-6@10'				-Becomes brown; some clay	17		
	B-6@12.5'			SM	**ALLUVIAL FAN DEPOSITS (Qyf)** Silty SAND, medium dense, moist, reddish brown; fine sand	17	112.0	18.2
	B-6@15'				-Becomes fine to coarse sand	15		
20	B-6@20'					15		
24	B-6@25'			SC	Clayey SAND, stiff, moist, grayish brown; fine to coarse sand; trace fine gravel; micaceous	23		

Total depth 26.5 feet
No groundwater encountered
No caving
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 5/12/2016

T2719-22-01 BORING LOGS.GPJ

Figure A-6,
Log of Boring B-6, 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. T2719-22-01

BORING B-10

ELEV. (MSL.) 1268 DATE COMPLETED 08/30/2016

EQUIPMENT HOLLOW STEM AUGER BY: A. Orton

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-10@0-5'			SM	**UNDOCUMENTED FILL (afu)**			
					Silty SAND, medium dense, dry, brown; fine to coarse sand; weeds at surface			
2				ML	SILT, stiff, slightly moist, olive brown; trace fine sand; trace mica			
	B-10@2.5'					21	95.2	13.8
4								
				SM	Silty SAND, medium dense, slightly moist, olive brown; fine to medium sand, trace coarse sand			
	B-10@5'					47	85.9	22.3
6								
8	B-10@7.5'				-Becomes brown; fine to coarse sand; trace mica	22	109.3	11.5
10	B-10@10'				-Increase in medium sand	23	113.1	10.1

Total depth 11 feet, 6 inches
No groundwater encountered
No caving
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 8/30/2016

Figure A-10,
Log of Boring B-10, Page 1 of 1

T2719-22-01 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.

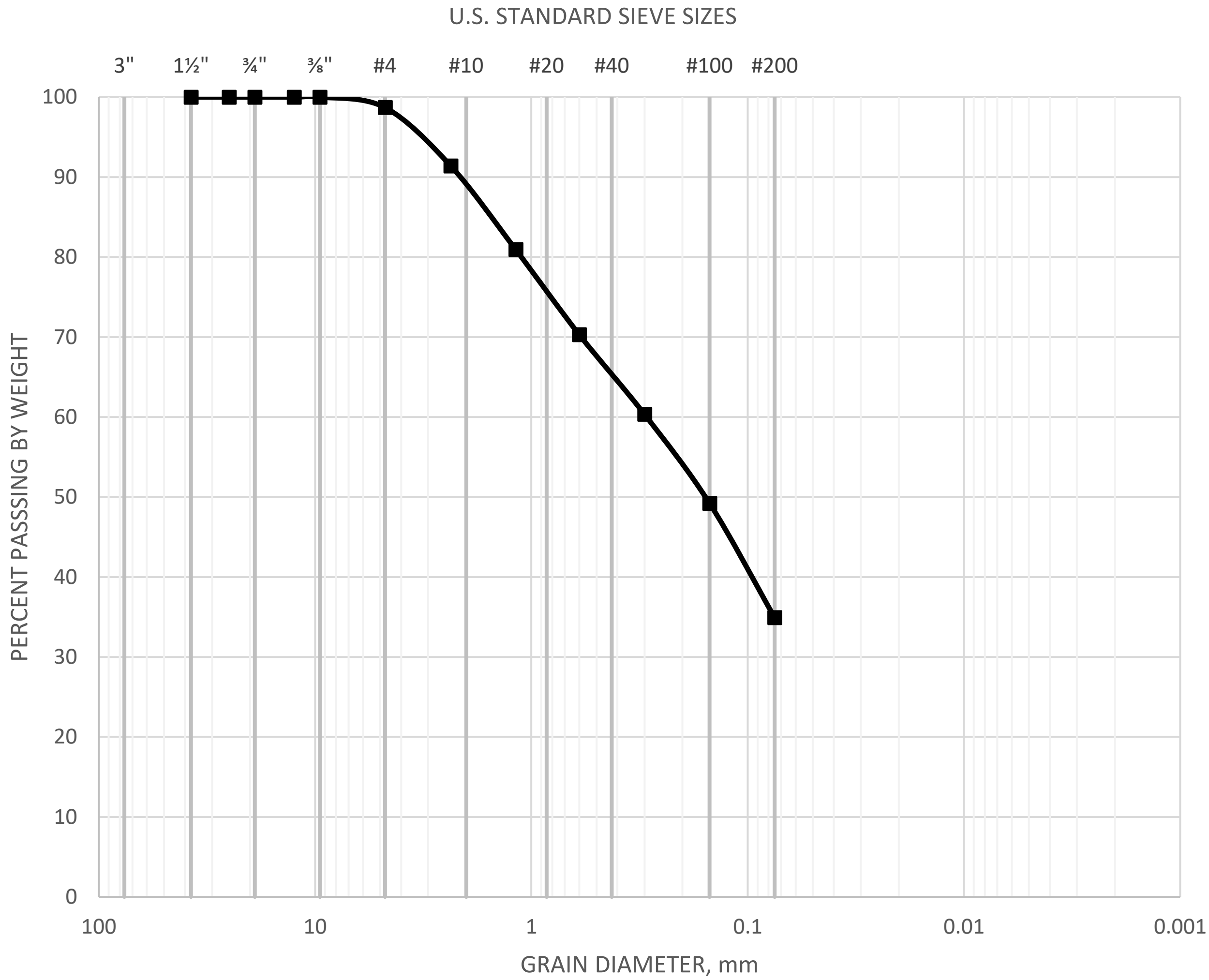
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for maximum dry density and optimum moisture content, corrosivity, Atterberg limits, consolidation, expansion characteristics, grain size analysis, direct shear strength, and in-place dry density and moisture content. The results of the laboratory tests are summarized herein. The in-place dry density and moisture content of the samples tested are presented on the boring logs in Appendix A.

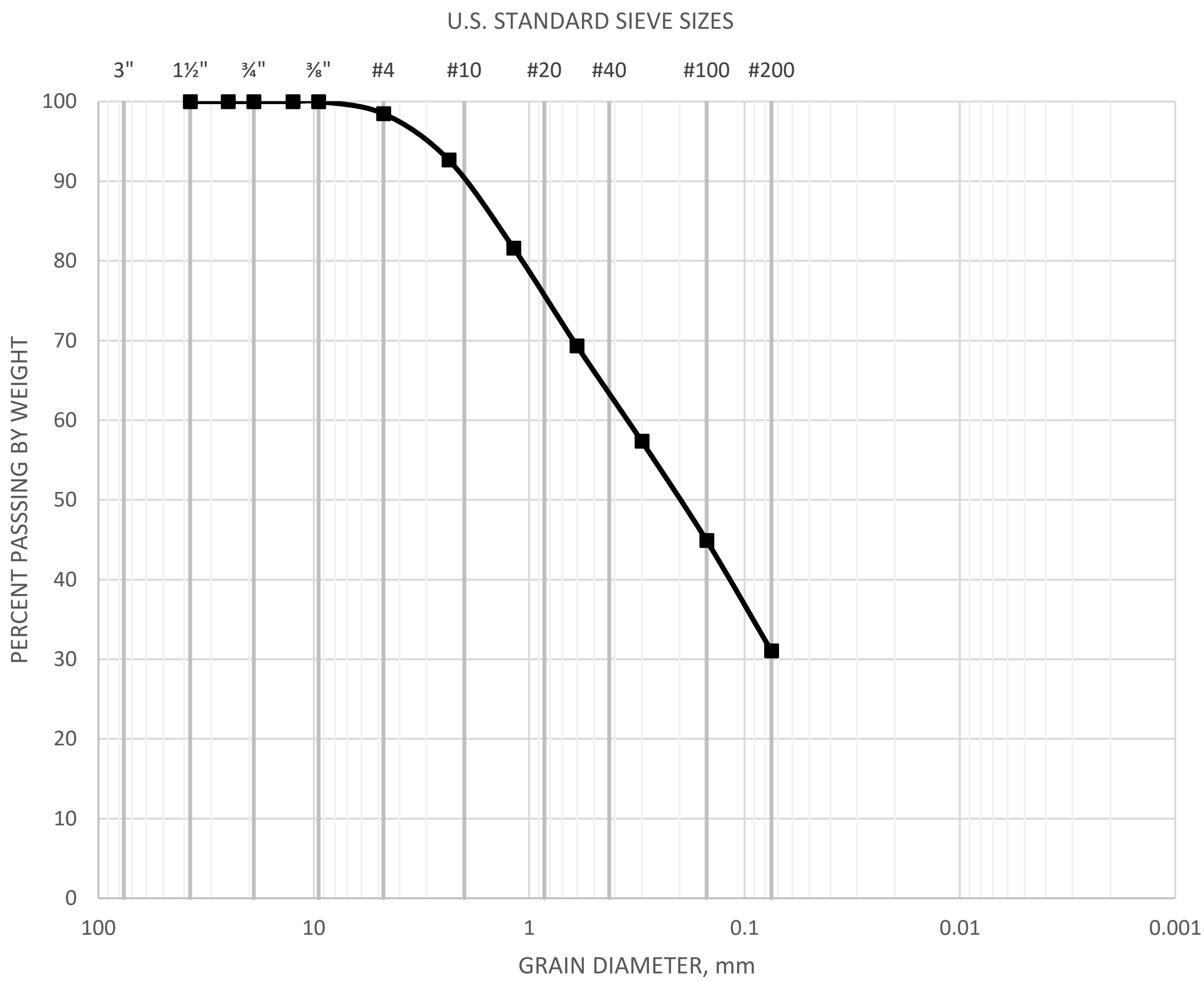
GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
P-1@5'	Silty SAND (SM), Grayish Brown	0.2	0.073	0.073

GEOCON	**GRAIN SIZE DISTRIBUTION** ASTM D-422	Project No.: T2719-22-02 APN 371-150-001 & -002 Grand Ave. & Kathryn Way Lake Elsinore, CA
	Checked by:	Feb 22 Figure B1

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
P-2@5'	Silty SAND (SM), Grayish Brown	0.33	0.073	0.073

GEOCON

GRAIN SIZE DISTRIBUTION

ASTM D-422

Checked by:

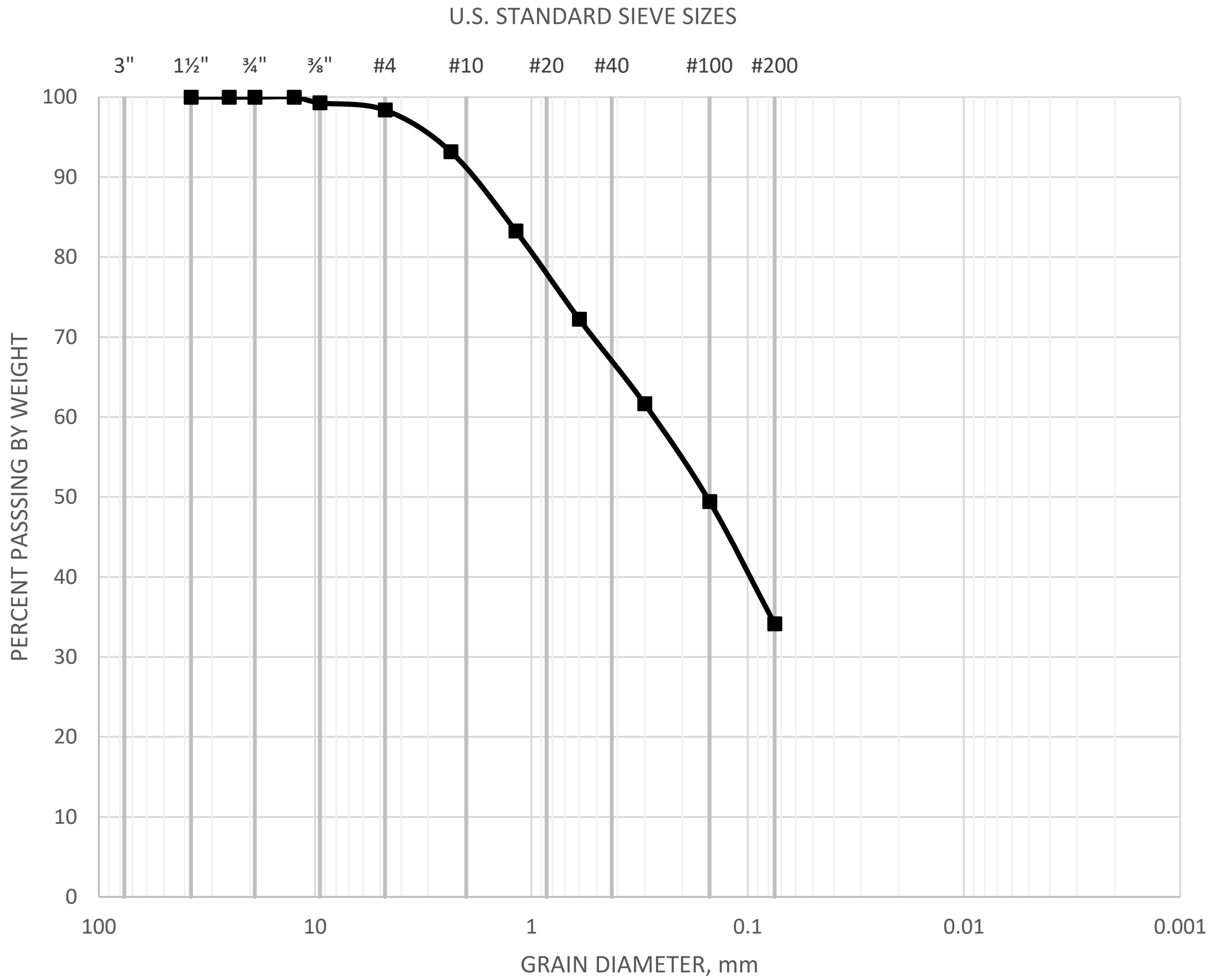
Project No.: T2719-22-02

APN 371-150-001 & -002
Grand Ave. & Kathryn Way
Lake Elsinore, CA

Feb 22

Figure B2

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
P-3@5'	Silty SAND (SM), Grayish Brown	0.28	0.073	0.073

GEOCON

GRAIN SIZE DISTRIBUTION

ASTM D-422

Checked by:

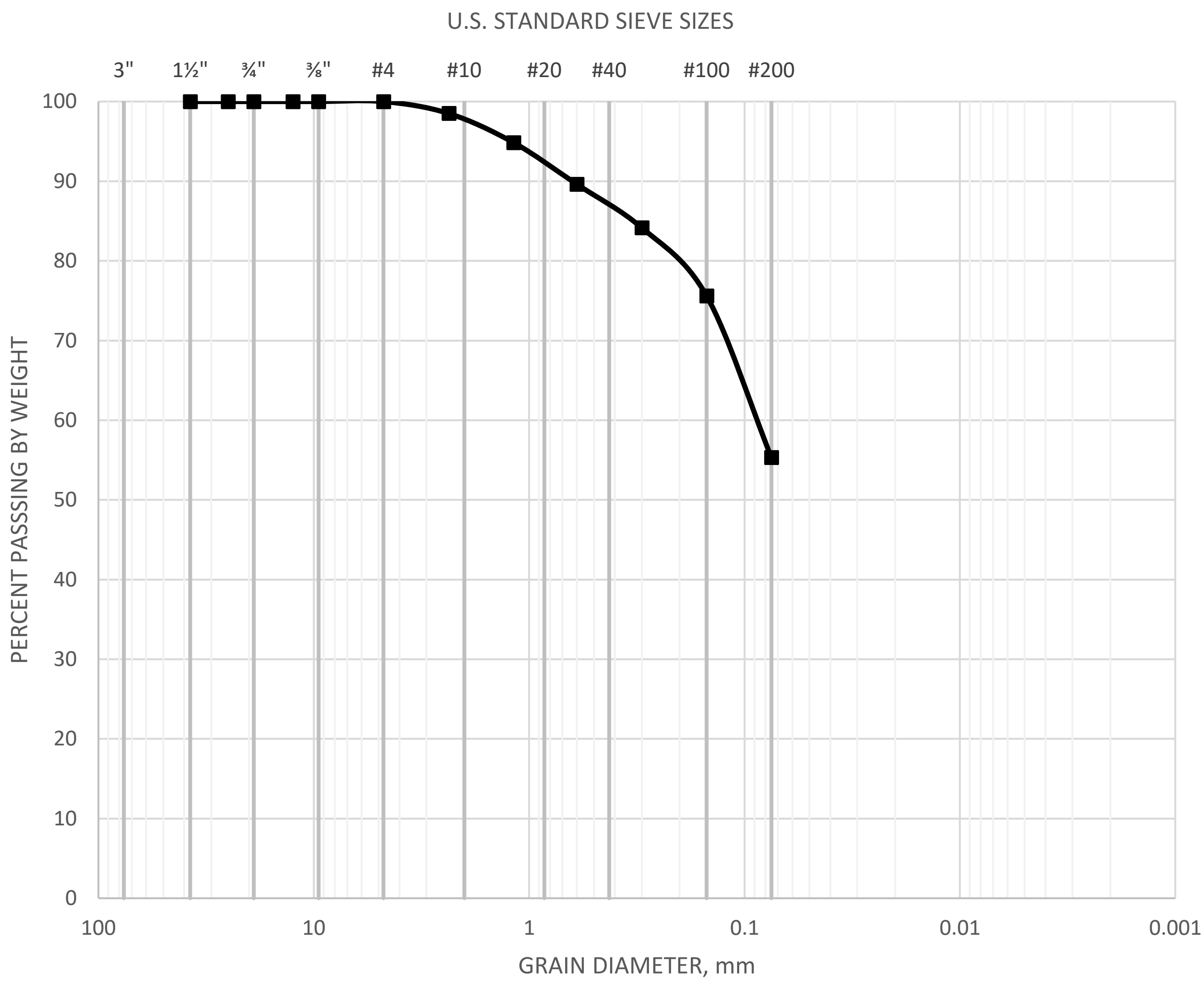
Project No.: T2719-22-02

APN 371-150-001 & -002
Grand Ave. & Kathryn Way
Lake Elsinore, CA

Feb 22

Figure B3

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
P-4@5'	Silty SAND (SM), Grayish Brown	0.089	0.073	0.073

GEOCON

GRAIN SIZE DISTRIBUTION

ASTM D-422

Checked by:

Project No.: T2719-22-02

APN 371-150-001 & -002
Grand Ave. & Kathryn Way
Lake Elsinore, CA

Feb 22

Figure B4

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-3 @ 0-5’	Silty SAND (SM), blackish dark brown	131.0	7.5
B-6 @ 0-5’	Silty SAND (SM), gray	120.0	11.0
T-5 @ 3-5’	Silty SAND to Sandy SILT (SM/ML), dark grayish brown to olive	128.0	10.0

SUMMARY OF CORROSIVITY TEST RESULTS

Sample No.	Chloride Content (ppm)	Sulfate Content (%)	pH	Resistivity (ohm-centimeter)
B-3 @ 0-5’	35	0.001	7.5	7,000

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 ATTERBERG LIMIT TEST RESULTS ASTM D4318

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	USCS
B-1 @ 12.5’	21	20	1	ML
B-1 @ 17.5’	20	17	3	ML
B-1 @ 30’	26	19	7	CL-ML
B-1 @ 32.5	23	18	5	ML
B-1 @ 40’	20	20	0	ML

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

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

Sample No.	Geologic Origin	In-situ Dry Density (pcf)	Moisture Content Before Test (%)	Final Moisture Content (%)	Axial Load with Water Added (psf)	Percent Collapse
B-1 @ 2.5’	afu	101.1	7.0	29.0	2,000	2.3
B-1 @ 20	Qyf	109.6	20.7	100.0	2,000	-0.1
B-1 @ 25’	Qyf	121.3	14.4	100.0	2,000	0.0
B-2 @ 2.5’	afu	118.2	4.7	31.2	2,000	1.5
B-3 @ 7.5’	Qyf	123.4	5.1	39.3	2,000	1.3
B-3 @ 12.5’	Qyf	112.1	5.3	29.3	2,000	1.3
B-5 @ 7.5’	Ql	94.1	23.8	82.9	2,000	-0.1
B-6 @ 12.5’	Qyf	112.0	18.2	100.0	2,000	0.0
B-8 @ 2.5’	afu	93.0	15.1	24.9	2,000	1.1
B-8 @ 5’	afu	100.3	10.8	18.7	2,000	1.5
B-8 @ 7.5’	afu	98.9	8.5	16.6	2,000	2.6
B-8 @ 10’	Ql	104.0	13.5	17.0	2,000	0.6
B-8 @ 15’	Ql	107.2	18.3	17.7	2,000	0.1
B-8 @ 20’	Ql	117.4	13.8	13.6	3,000	0.4
B-8 @ 25’	Ql	105.8	19.3	17.6	3,000	0.1
B-11 @ 5’	afu	98.5	8.9	20.2	2,000	-2.5
B-11 @ 15’	Qyf	107.0	9.9	16.5	2,000	-2.0
B-12 @ 10’	afu	112.2	17.5	17.2	2,000	0
B-12 @ 15’	Ql	111.7	22.8	24.1	2,000	-0.1
B-12 @ 20’	Ql	110.4	19.8	18.4	2,000	0
T-5 @ 3’	afu	114.2	3.4	13.5	2,000	1.5
T-5 @ 14’	Ql	89.6	10.0	19.0	2,000	11.8
T-6 @ 16’	Ql	112.4	7.7	17.4	2,000	0.7

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-3 @ 0-5’	8.4	13.3	115.8	3
B-6 @ 0-5’	10.8	20.8	104.7	3
B-11 @ 0-5’	10.5	20.2	105.9	15

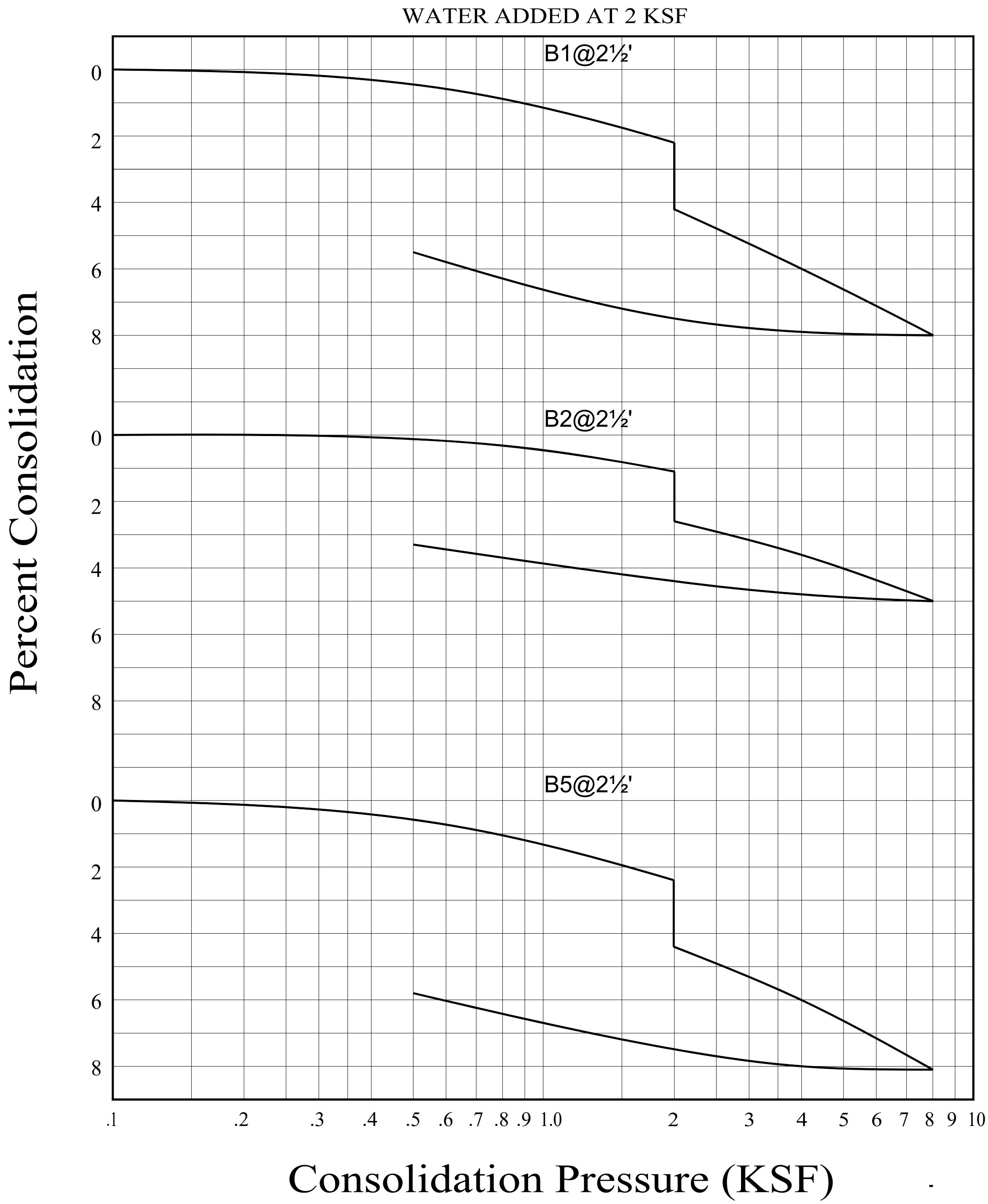
GEOCON
WEST, INC.
GEOTECHNICAL CONSULTANTS
41571 CORNING PLACE SUITE 101 MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

AMO

LABORATORY TEST RESULTS

GRAND AVE. AT KATHRYN WAY
LAKE ELSINORE, CALIFORNIA

FEB. 2022 | PROJECT NO. T2719-22-02 | FIG B-7



GEOCON
W E S T, I N C.

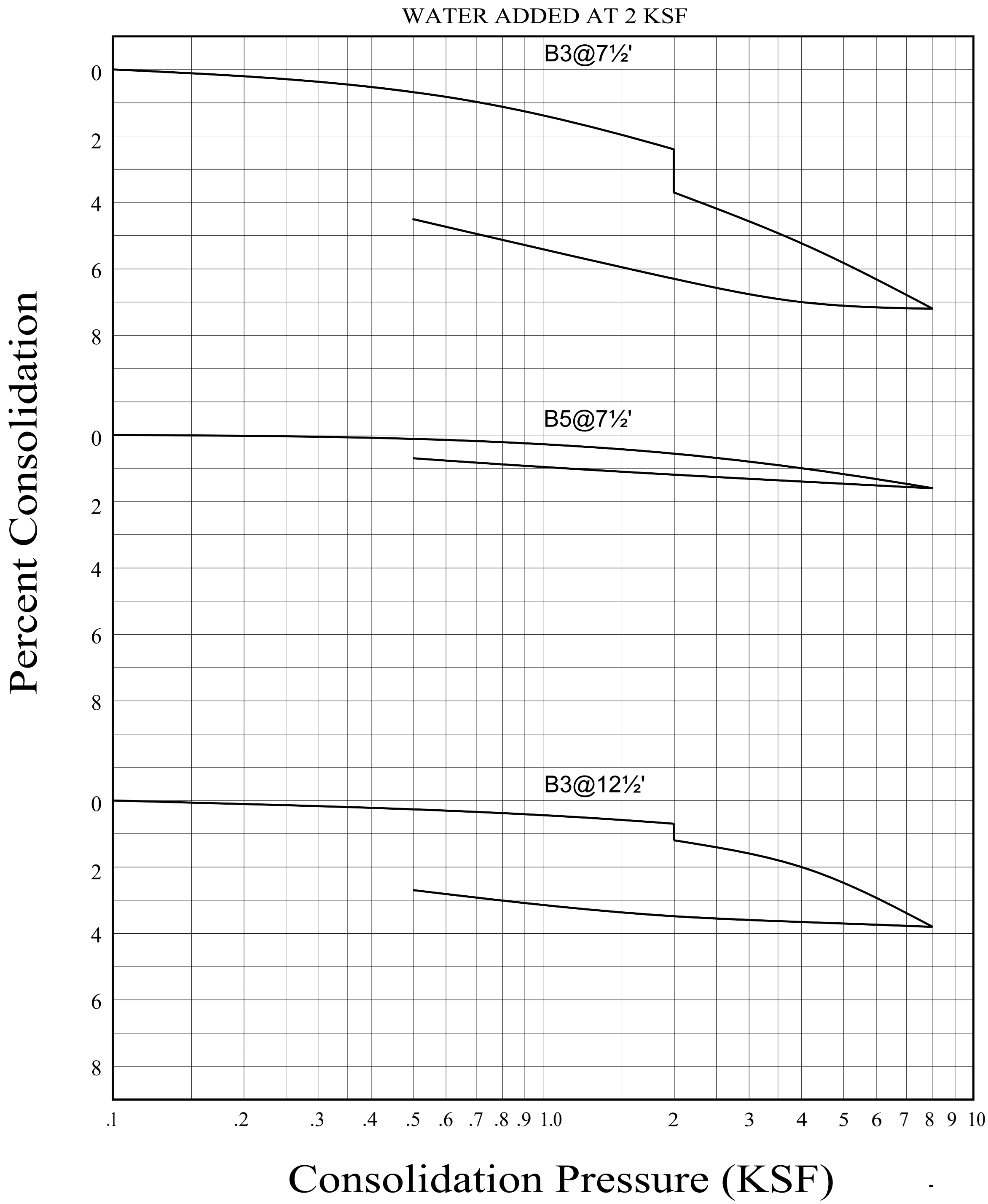
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE (951) 304-2300 - FAX (951) 304-2392

Drafted by: RDG	Checked by: HHD

CONSOLIDATION TEST RESULTS

GRAND AVE. AT KATHRYN WAY
LAKE ELSINORE, CALIFORNIA

FEB. 2022	PROJECT NO. T2719-22-02	FIG. B-9



GEOCON WEST, INC.

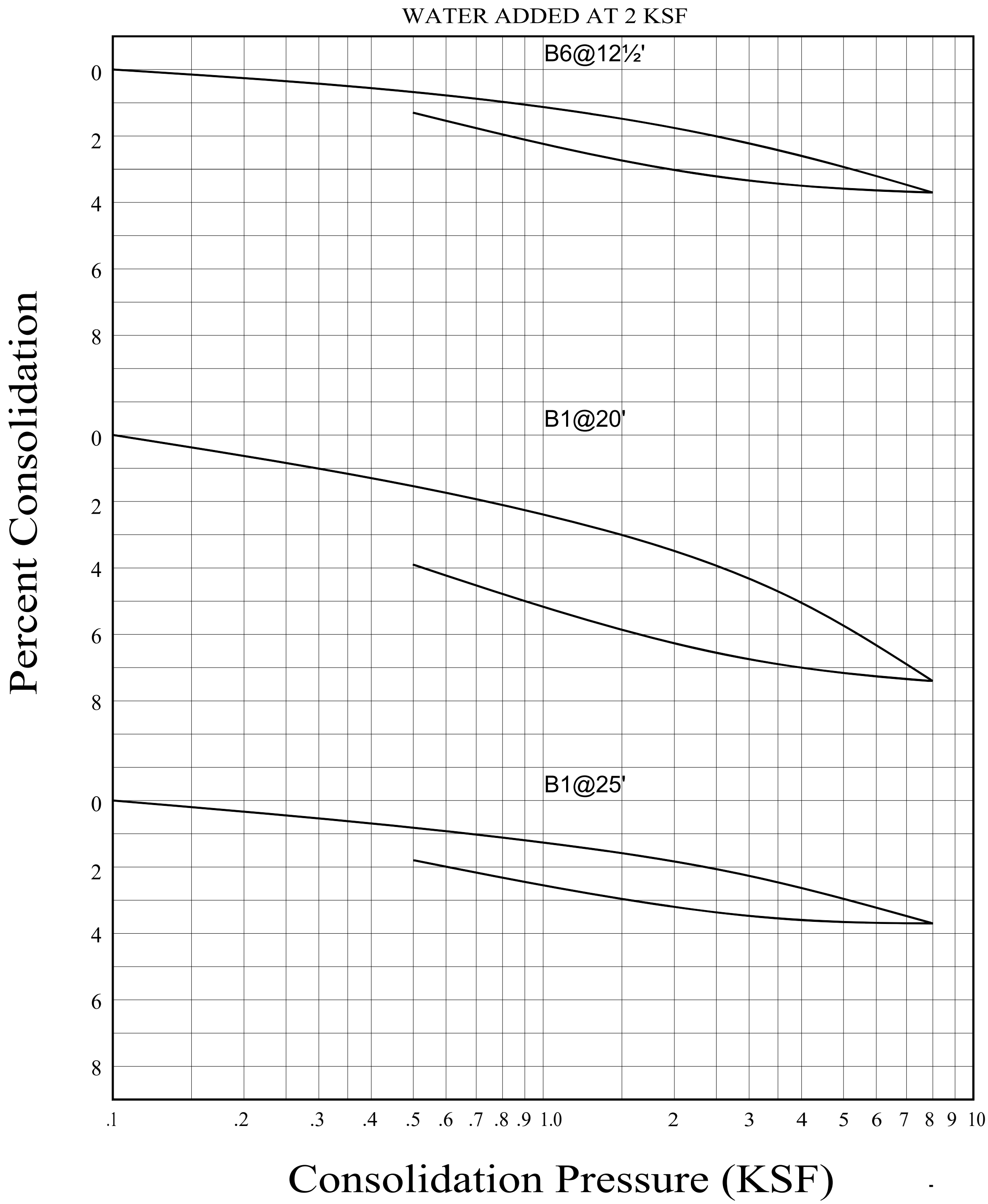
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE (951) 304-2300 - FAX (951) 304-2392

Drafted by: RDG | Checked by: HHD

CONSOLIDATION TEST RESULTS

GRAND AVE. AT KATHRYN WAY
LAKE ELSINORE, CALIFORNIA

FEB. 2022 | PROJECT NO. T2719-22-02 | FIG. B-10

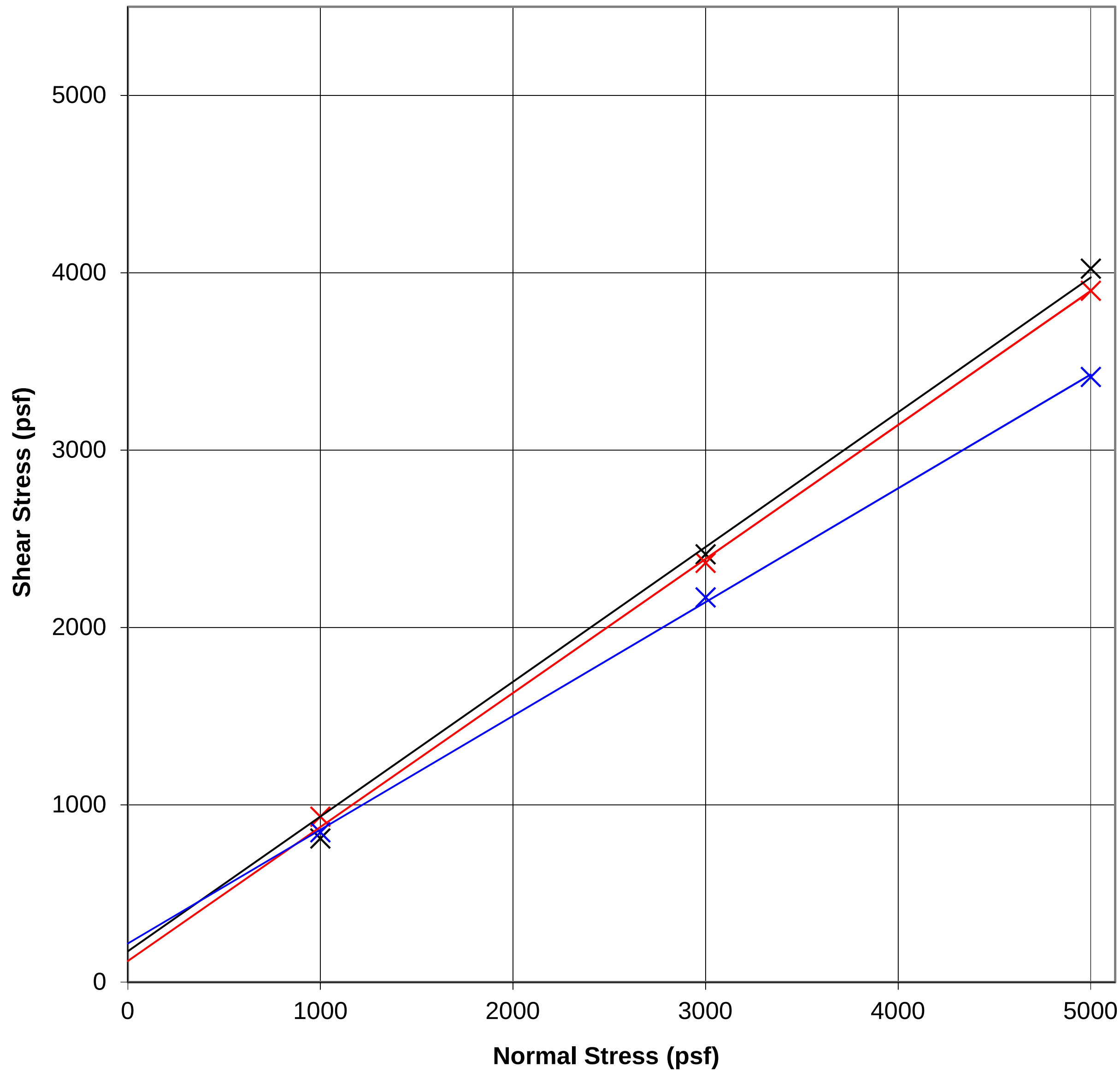


GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE (951) 304-2300 - FAX (951) 304-2392

CONSOLIDATION TEST RESULTS

GRAND AVE. AT KATHRYN WAY
LAKE ELSINORE, CALIFORNIA

Drafted by: RDG	Checked by: HHD	FEB. 2022	PROJECT NO. T2719-22-02	FIG. B-11



SAMPLE ID	SOIL TYPE	INITIAL DRY DENSITY (pcf)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)	C (psf)	ϕ (deg)
B-2 @ 2.5'	SM	111.1	6.1	15.3	140	35
*B-3 @ 0-5'	SM	114.7	9.6	15.1	170	37
*B-6 @ 0-5'	SM	104.2	13.9	21.4	220	33

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

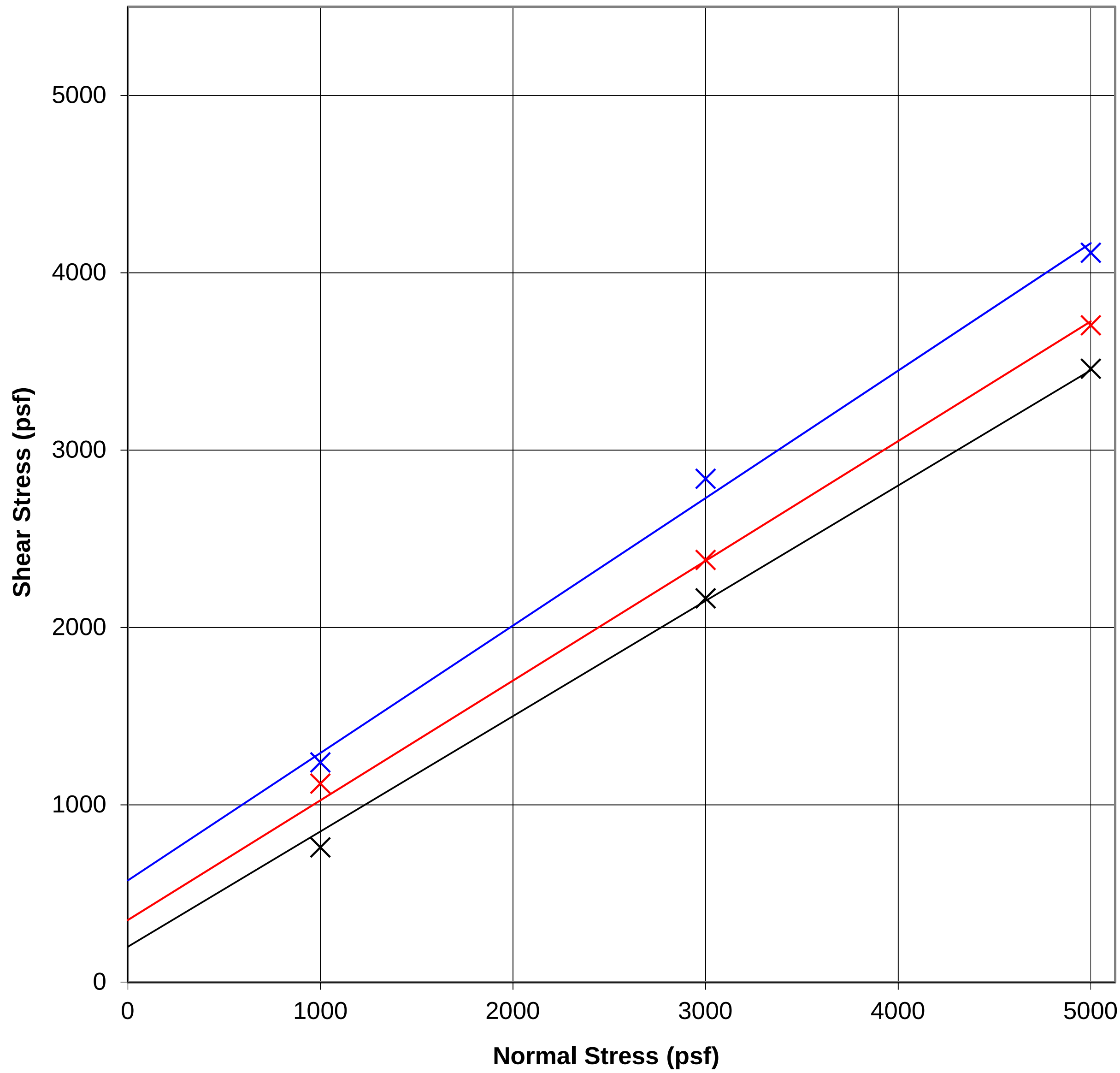
GEOCON WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

AMO

DIRECT SHEAR TEST RESULTS

GRAND AVE. AT KATHRYN WAY
ELSINORE, CALIFORNIA

FEB. 2022 | PROJECT NO. T2719-22-02 | FIG B-16



SAMPLE ID	SOIL TYPE	INITIAL DRY DENSITY (pcf)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)	C (psf)	ϕ (deg)
B-6 @ 2.5'	SM	96.5	6.9	24.8	220	31
B-11 @ 10'	SM	109.5	19.0	19.8	460	33
B-12 @ 5'	SM	107.6	14.9	22.2	570	36

*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

AMO

DIRECT SHEAR TEST RESULTS

GRAND AVE. AT KATHRYN WAY
ELSINORE, CALIFORNIA

FEB. 2022	PROJECT NO. T2719-22-02	FIG B-17

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

BUILDER'S MAX
APN 371-150-001 & 371-150-002
GRAND AVENUE AT KATHRYN WAY
LAKE ELSINORE, CALIFORNIA

PROJECT NO. T2719-22-02

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

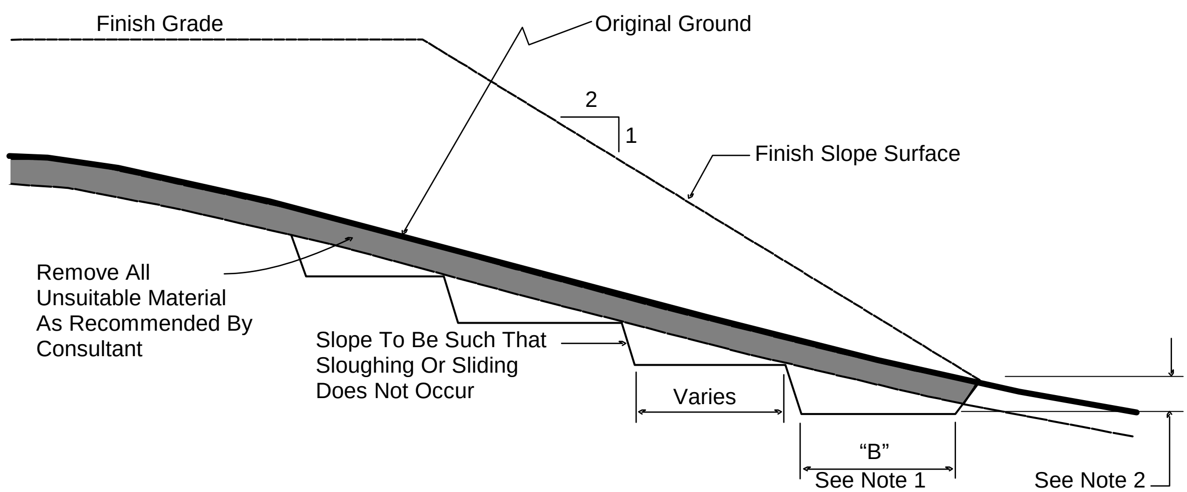
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



No Scale

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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

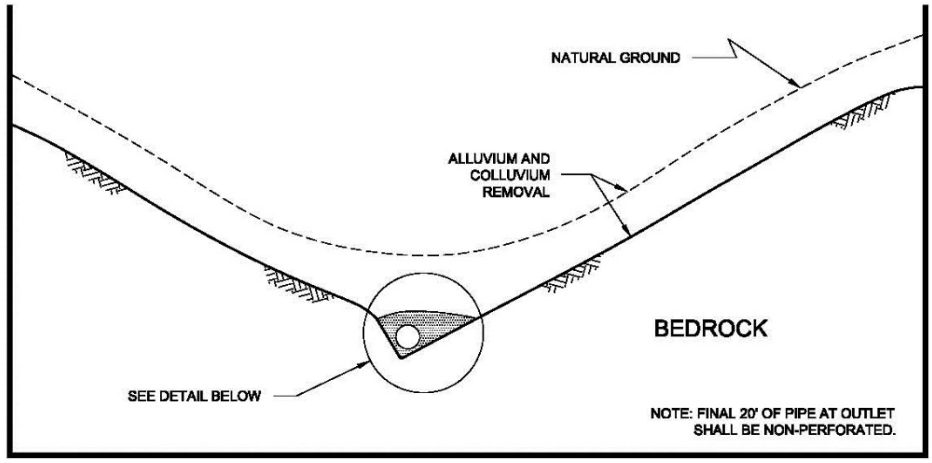
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

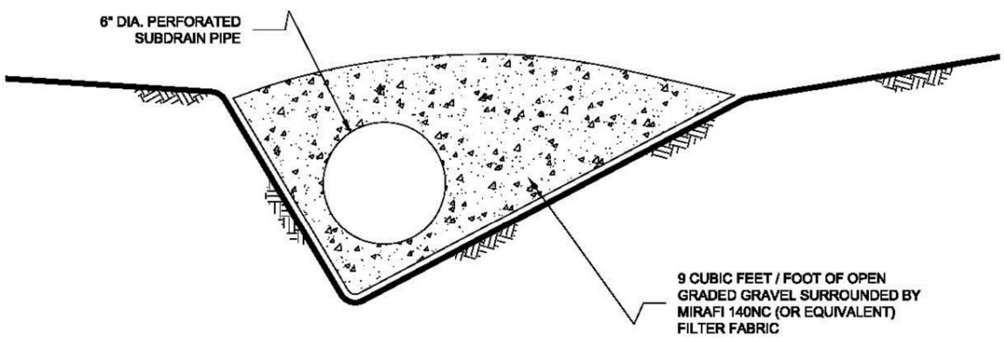
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL





NOTES:

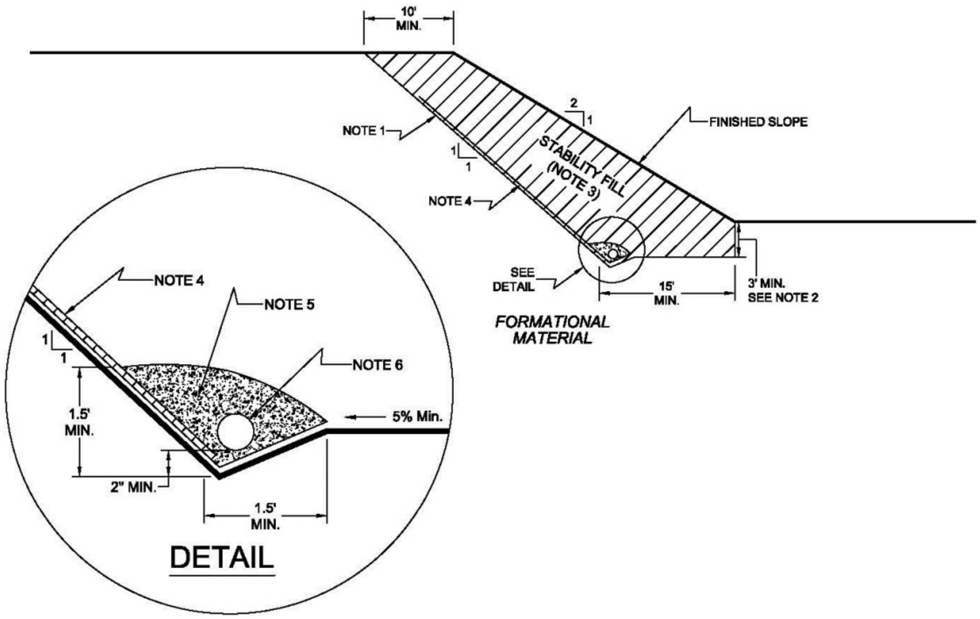
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

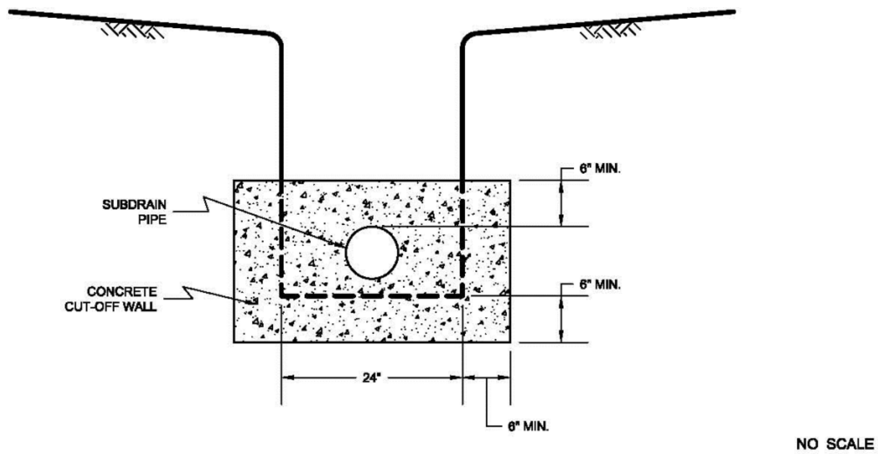
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

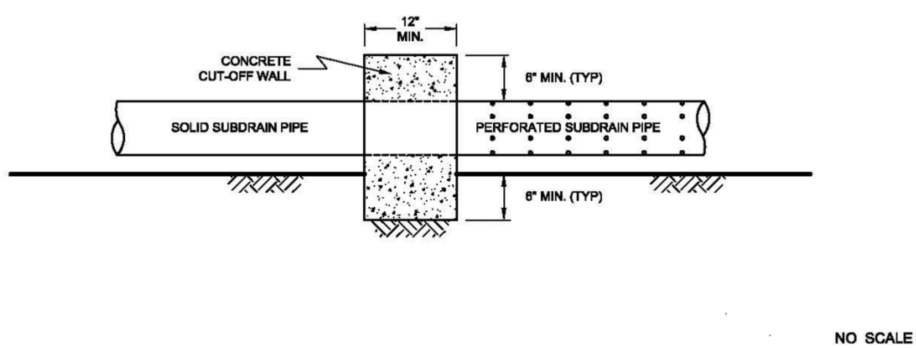
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

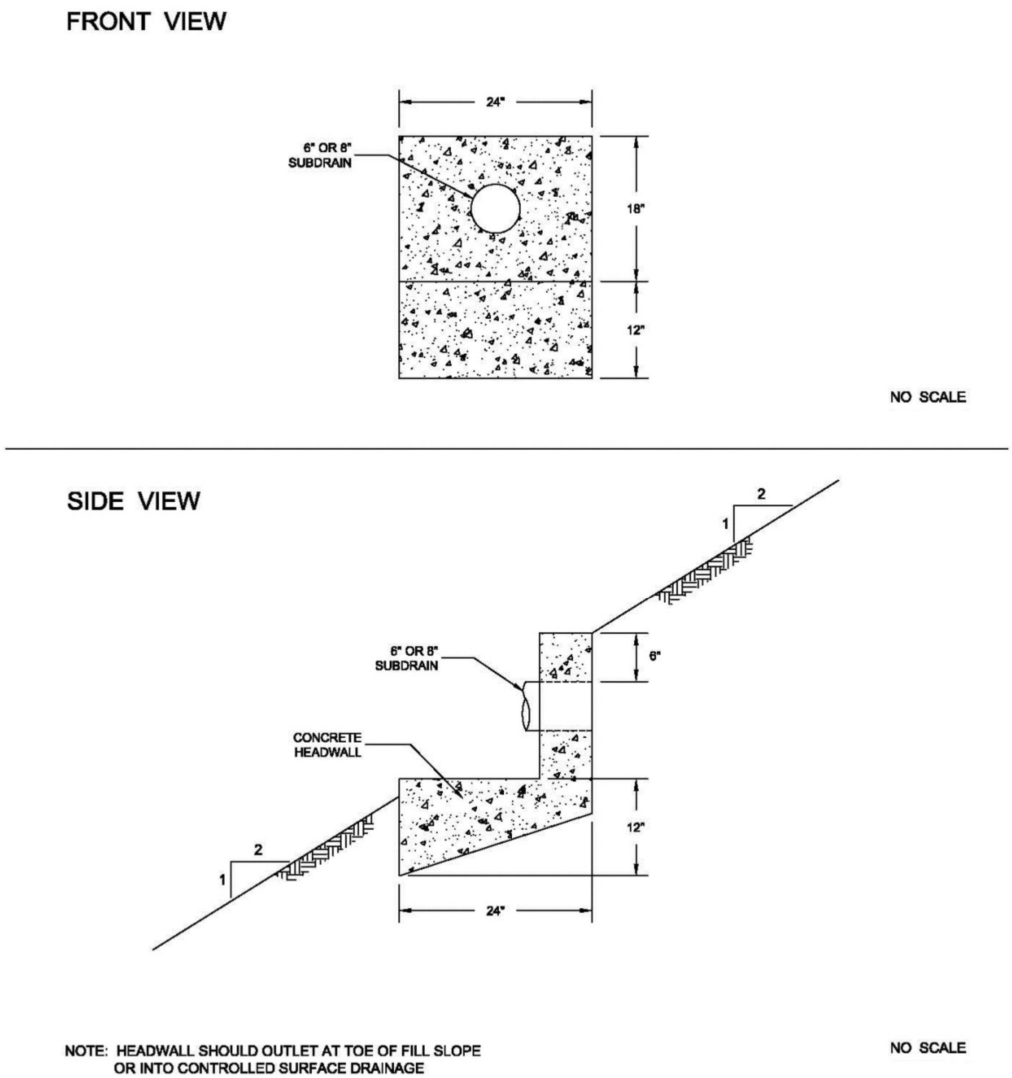


SIDE VIEW



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



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.