



PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER – ADESA SITE
6464 33RD STREET
JURUPA VALLEY, CALIFORNIA

JUNE 21, 2024
PROJECT NO. T2875-22-02

PREPARED FOR:
RIVERSIDE COMMUNITY COLLEGE DISTRICT
RIVERSIDE, CALIFORNIA



Project No. T2875-22-02

June 21, 2024

Riverside Community College District
3801 Market Street, 3rd Floor
Riverside, California 92501

Attention: Ms. Myra Nava

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED INLAND EMPIRE TECHNICAL TRADE CENTER (IETTC)
ADESA SITE, 6464 33RD STREET
JURUPA VALLEY, CALIFORNIA

Dear Ms. Nava:

In accordance with your authorization of Geocon Proposal MU-24-0613-P-GE revised April 19, 2024, Geocon West, Inc. (Geocon) herein submits the results of our preliminary geotechnical investigation for the proposed Inland Empire Technical Trade Center (IETTC) located at the Adesa site at 6464 33rd Street in the City of Jurupa Valley, California. The accompanying report presents the results of our study, and our preliminary conclusions and recommendations pertaining to the geotechnical aspects of the proposed education center.

The site is considered suitable for development provided the recommendations of this report are followed.

The primary intent of this study was to address potential geologic hazards and geotechnical conditions that could impact the project. A design level geotechnical study will be required once a grading plan is available in order to provide updated geotechnical recommendations for design and construction.

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.

Ashleigh D. Love
PE 79019



Gerald A. Kasman
CEG 2251



Petrina Zen
GE 3217



ADL:PZ:GAK:hd

Distribution: Addressee (email)

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

We understand the Riverside Community College District (RCCD) has identified the Adesa Site as a potential location where the proposed Inland Empire Technical Trade Center (IETTC) may be constructed and has contracted with Gafcon to provide site assessment due diligence, entitlement, and support services in the evaluation and potential securement of the site. This report presents the results of our preliminary geotechnical investigation for the proposed IETTC at the Adesa site located at 6464 33rd Street in the City of Jurupa Valley, California (see *Vicinity Map*, Figure 1). The purpose of the investigation was to perform a subsurface exploration, laboratory testing, and geotechnical analyses and, based on the conditions encountered, provide preliminary recommendations pertaining to the geotechnical aspects of developing the property. A design level geotechnical study, in accordance with the requirements of California Geological Survey Note 48, will be required once a site plan is available in order to provide detailed geotechnical recommendations for design and construction.

The scope of our investigation included review of aerial photographs, geologic mapping, subsurface exploration, percolation testing, laboratory testing, engineering analyses, and the preparation of this report. The site was explored on May 16, 2024, by excavating eight 8-inch-diameter borings using a CME 75HT truck-mounted, hollow-stem auger drilling machine to approximate depths of 20 to 51½ feet below the existing ground surface. The approximate locations of the exploratory borings are depicted on the *Geologic Map* (Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A. A summary of the information reviewed for this study is presented in the *List of References*.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

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

2. SITE AND PROJECT DESCRIPTION

The site is comprised of two parcels located at 6464 33rd Street in Jurupa Valley, California: Assessor's Parcel Numbers (APNs) 177-110-016 and 177-110-018. The two parcels compose a 24-acre triangular site, which is bounded by 33rd Street and an undeveloped lot to the northeast, Florine Avenue to the northwest, Union Pacific train tracks to the south, a stormwater basin to the east, and a residential development at the site's northeast corner. A transient community exists at the southwestern end of the site close to the bridge for the California State Route 60. Jurupa Community Services District has a lift station located at the southeast corner of the site. Historically, the site has been used for industrial manufacturing and storage purposes. At the time of our investigation, the previous improvements had been demolished, except for asphalt concrete pavement and concrete foundations remaining. Various trash was scattered throughout the site. Utility lines remain below the subsurface, and the site is generally bounded by a chain link fence. The site slopes gently to the south with elevations ranging from approximately 894 feet above mean sea level (MSL) in the northern area to approximately 879 feet MSL in the southern area of the site. Drainage is likely by sheet flow to the south. The latitude and longitude of the site are approximately 34.0116 and -117.4244 degrees, respectively.

We understand the site will potentially be developed to include an education center consisting of the Riverside Community College District IETTC. We expect grading will entail cuts and fills up to approximately 5 feet, exclusive of remedial grading. Site BMPs will likely be constructed; however, the location and depth of the BMPs are not known at the time of this study. Due to the preliminary nature of the project, plans depicting the proposed development are not available for inclusion in this report.

Structural plans and loading information were not available for our review at the time of this investigation. We expect the proposed structures will be up to three stories and made of lightweight timber or steel frame construction supported by conventional foundation systems. For our preliminary analysis, we assume maximum column loads will be up to 300 kips and maximum wall loads will be up to 3 kips per linear foot. Preliminary geotechnical recommendations for design of the structure are based on these assumptions and provided herein.

3. GEOLOGIC SETTING

The subject site is situated within a natural geomorphic province in southwestern California known as the Peninsular Ranges, which is characterized by steep, elongated ranges and valleys that trend northwesterly. This geomorphic province encompasses an area that extends 125 miles, from the Transverse Ranges and the Los Angeles Basin on the north, south to the Mexican border, and beyond another 795 miles to the tip of Baja California. The site is located within the Perris block, a relatively tectonically stable structural block between the San Jacinto and Elsinore Fault zones. The site is located at the southern portion of a valley, toe of an old alluvial fan, that derives from the surrounding Jurupa Mountains. The Jurupa Mountains and Cretaceous to pre-Cretaceous in age and contain numerous quarries where various minerals, granitic rock and marble are mined.

4. GEOLOGIC MATERIALS

Based on our field investigation and published geologic maps, the soils underlying the site consist of undocumented artificial fill material over Pleistocene age alluvial fan deposits consisting of boulders, cobbles, gravel, sand and silt (CGS, 2010). Detailed stratigraphic profiles are provided in the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 2 feet below existing ground surface. The artificial fill generally consists of reddish to orange brown fine- to coarse-grained silty sand. The artificial fill is characterized as moist, and loose to medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Older Alluvium (Qof)

The artificial fill is underlain by Pleistocene age older alluvium. The older alluvial soils consist primarily of reddish to orange brown interbedded silty sand and sandy silt that is fine- to coarse-grained. The older alluvium can be characterized as moist and loose to very dense.

5. GROUNDWATER

Data from the California Department of Water Resources indicate the closest groundwater monitoring well to the site is the RCSD 28th St Well (Well 02S05W10P001S), located approximately 1.6 miles to the southeast of the site (CDWR, 2024). Groundwater level data for this well was available for the monitoring period between 2011 and 2023 and the groundwater elevation has ranged from approximately 773 to 778 feet above mean sea level (MSL), corresponding to a groundwater depth of 82 to 87 feet below the existing ground surface, respectively. Based on the available data, the most recent groundwater elevation measured in this well was approximately 776 feet MSL in May 2023. The lowest ground surface elevation at the site is approximately 879 feet MSL.

Groundwater was not encountered during our field excavation drilled to a maximum depth of approximately 51½ feet below ground surface. Based on the historic high groundwater levels in the site vicinity (CDWR, 2024), the lack of groundwater in our borings, and the depth of proposed construction, static groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. Using the available historical data, we estimate the groundwater to be at a depth of 100 feet or greater below the site. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the *Site Drainage and Moisture Protection* section of this report.

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include Holocene-active, pre-Holocene, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018). By definition, a Holocene-active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A pre-Holocene fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2024a; 2024b) for surface fault rupture hazards. No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the San Jacinto Fault located approximately 7.4 miles to the northeast (CGS, 2017). Other nearby active faults are the Cucamonga section of the Sierra Madre Fault Zone, the San Andreas Fault Zone, and the Chino Fault located approximately 10½ miles to the northwest, 14 miles to the north-northeast, and 14½ mile to the southwest of the site, respectively (USGS, 2006; Ziony and Jones, 1989).

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. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Long Beach	March 10, 1933	6.4	41	SW
Tehachapi	July 21, 1952	7.5	113	NW
San Fernando	February 9, 1971	6.6	62	WNW
Whittier Narrows	October 1, 1987	5.9	38	W
Sierra Madre	June 28, 1991	5.8	37	WNW
Landers	June 28, 1992	7.3	58	ENE
Big Bear	June 28, 1992	6.4	37	ENE
Northridge	January 17, 1994	6.7	65	WNW
Hector Mine	October 16, 1999	7.1	77	ENE
Ridgecrest	July 5, 2019	7.1	121	N

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be minimized if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Liquefaction Potential and Seismic Settlement

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Seismically induced settlement may occur whether the potential for liquefaction exists or not.

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* (SCEC, 1999) requires a 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 enough to induce liquefaction.

A review of the Riverside County General Plan (2012) indicates that the site is located in an area of moderate susceptibility for liquefaction. As discussed in the *Groundwater* section of this report, based on the available well data, we estimate the groundwater depth is 100 feet or greater below the site. Also, groundwater was not encountered during our field excavation drilled to a maximum depth of approximately 51½ feet below ground surface. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations to occur at the site is very low.

We performed a liquefaction analysis of the soils underlying the site using the 1996 NCEER method of analysis with the updates by Youd et al. (2001). This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance. An average conversion factor of 0.63 was used to derive SPT blow-count values from California Modified Sampler blow-count values.

The liquefaction potential evaluation was performed by utilizing a groundwater depth of 100 feet, a magnitude 8.1 earthquake, and the modified MCE_G peak ground acceleration (PGA_M) of 0.555g. Our *Empirical Estimation of Liquefaction Potential* is included as Figure 5. Due to the lack of shallow groundwater and dense to very dense soils encountered at the project site, liquefaction is not a design consideration.

Additionally, an evaluation of seismically induced “dry-sand” settlement was performed, with the resulting seismic “dry-sand” settlement estimated to be approximately $\frac{1}{8}$ inch. An analysis of seismically induced “dry-sand” settlement is included as Figure 6.

6.4 Expansive Soil

The on-site soil generally consists of silty sand and sandy silt. Laboratory testing result indicates a sample of the near surface soil exhibits a “very low” expansion potential (expansion index [EI] of 20 or less).

6.5 Hydrocompression

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

Soil samples tested for hydrocompression exhibit a collapse potential of up to 1 percent. Based on these results, the potential settlement due to hydrocompression is estimated to be up to approximately $\frac{3}{8}$ inch, with a differential settlement of approximately $\frac{1}{8}$ inch across 40 feet.

6.6 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes very gently to the south-southeast. There are no known landslides near the site, nor is the site in the path of any known or potential landslides (USGS, 20223a). Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the Riverside County General Plan (2012) the site is not located within a potential inundation area for an earthquake-induced dam failure. Therefore, the probability of earthquake-induced flooding is considered very low.

6.8 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, flooding resulting from a seismically induced seiche is considered unlikely.

The Riverside County General Plan (2012) indicates that the site is not within a 1% or 0.2% annual chance of flood.

6.9 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within an oil field and oil or gas wells are not documented within ½-mile of the site (CalGEM, 2024). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the CalGEM.

Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.10 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence (USGS, 2024b). No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

- 7.1.1 No soil or geologic conditions were encountered that would preclude the development of the property as assumed, provided the recommendations of this report are followed. This report should be considered “preliminary” and the geotechnical design parameters presented herein should be verified once the project progresses to a more finalized state.
- 7.1.2 Based on our investigation and available geologic information, active or potentially active faults are not present on or trending toward the site.
- 7.1.3 The artificial fill and the upper portion of the older alluvial deposits are considered unsuitable for the support of compacted fill or settlement-sensitive improvements. Remedial grading in the form of removal and compaction of these soils will be required in accordance with the *Grading* section of this report. The site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed.
- 7.1.4 An evaluation of seismically induced “dry-sand” settlement was performed, with the resulting seismic “dry-sand” settlement estimated to be approximately $\frac{3}{8}$ inch. Seismically induced differential settlement is expected to be on the order of $\frac{1}{8}$ inch across 40 feet.
- 7.1.5 Soil samples tested for hydrocompression exhibit a collapse potential of up to approximately 1 percent for the site. Remedial grading will reduce some of the settlement potential of the near-surface soils; however, the soil with the highest tested collapse potential is at a depth of 15 feet. The potential settlement due to hydrocompression is estimated to be up to $\frac{3}{8}$ inch, with a differential settlement of $\frac{1}{8}$ inch across 40 feet.
- 7.1.6 Moisture contents of the site soils are expected to vary based on the season and amount of precipitation. Special handling of the soil should be anticipated, particularly if grading occurs during the rainy season, as drying back of the existing materials should be anticipated prior to their use as fill.
- 7.1.7 Site infiltration evaluation was not in the scope of this investigation. However, due to the presence of potentially hydrocompressive soils, future infiltration systems, if any, should be located a minimum distance of 25 feet from proposed structures and a minimum distance of 15 feet from site improvements to reduce the potential for induced settlements to adversely impact the proposed structures and improvements.

7.1.8 Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

7.1.9 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.2 Excavation and Soil Characteristics

7.2.1 Excavation of the fill and upper alluvial deposits are expected to be possible with moderate effort using conventional heavy-duty equipment in proper functioning order.

7.2.2 The soil encountered in the field investigation is considered to be “non-expansive” (expansion index [EI] 20 or less) as defined by 2022 California Building Code (CBC) Section 1803.5.3. We expect a majority of the soil encountered to possess a “very low” expansion potential in accordance with ASTM D4829. The table below presents soil classifications based on the expansion index. Although unlikely, any medium to highly expansive soils encountered at the site should not be placed within 4 feet of the proposed foundations, flatwork or paving improvements.

SOIL CLASSIFICATION BASED ON EXPANSION INDEX

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

7.2.3 Additional testing for expansion potential should be performed during grading along with plasticity index testing on soils with expansion indices of more than 20.

- 7.2.4 We performed laboratory tests on a near surface sample of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. Laboratory tests performed on samples of the site materials indicate that the on-site materials possess a “S0” sulfate exposure to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-19 Chapter 19. The following table presents a summary of concrete requirements set forth by 2022 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

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

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

- 7.2.5 Laboratory test results indicate a resistivity of 3,100 ohm-cm, pH of 8.3, chloride content of 63 ppm, and sulfate content of 0 ppm. Resistivity serves as an indication parameter for the presence of soluble salts only and does not specifically indicate a corrosive situation exists. As shown in the table below, the site would **not** be classified as “corrosive” to buried improvements based on resistivity in accordance with the Caltrans Corrosion Guidelines (Caltrans, 2021).

CALTRANS CORROSION GUIDELINES

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

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

7.3 Grading

- 7.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix C and the City of Jurupa Valley Grading Ordinance.
- 7.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the city inspector, owner or developer, grading contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.
- 7.3.3 Site preparation should begin with the removal of deleterious material, debris and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.
- 7.3.4 The upper 4 feet of artificial fill and the upper alluvium in structural areas should be removed to expose dense, non-porous alluvium that has an in-place relative compaction of at least 85 percent based on ASTM D1557. Each building pad should have a uniform compacted subgrade, and a uniform remedial grading depth. At a minimum, 3 feet of newly placed engineered fill should be placed below the proposed building foundations. The removals should be extended laterally a minimum of 5 feet from the building footprint, or for a distance equal to the depth of fill below the footings, whichever is deeper. The actual depth of remedial grading should be evaluated by the Geotechnical Engineer (a representative of Geocon) during grading operations. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned, and compacted to a minimum of 90 percent of the maximum dry density (ASTM D1557), prior to fill placement.
- 7.3.5 In areas where small ancillary structures are proposed, such as trash enclosures and equipment pads, remedial removals should be extended to provide at least 12 inches of newly placed engineered fill below the bottom of proposed footings.
- 7.3.6 Remedial removals should be extended to depths of at least 3 feet below planned subgrade elevations in non-structural areas where pavement and flatwork are proposed.

- 7.3.7 The site should be brought to finish grade elevations with fill compacted in layers. Layers of fill should be no thicker than will allow for adequate bonding and compaction, approximately 6 to 8 inches in loose lift thickness. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content as determined by ASTM D1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.
- 7.3.8 The fill placed within 4 feet of proposed finish grade should possess a "very low" expansion potential (EI of 20 or less), where practical.
- 7.3.9 Import fill (if necessary) should consist of granular materials with a "very low" expansion potential (EI of 20 or less), generally free of deleterious material and rock fragments larger than 6 inches and should be compacted as recommended herein. Geocon should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material. Laboratory testing typically takes up to four working days to perform. The grading contractor should plan for the laboratory testing in their schedule to provide sufficient time to allow for completion of testing prior to importing materials.
- 7.3.10 Percolation testing to calculate infiltration rates were not in the scope of this investigation. However, if site infiltration rates are practical, then infiltration trenches or ponds, if planned, should be excavated into native soil without compaction effort applied to the basin bottom. Basin maintenance should include the removal of silt from the basin bottom after each significant rain event. Proposed infiltration systems, if planned, should be located a minimum distance of 25 feet from proposed structures and a minimum distance of 15 feet from site improvements to reduce the potential for induced settlements to adversely impact the proposed structures and improvements.
- 7.3.11 Foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing fill, steel, gravel or concrete.

7.4 Earthwork Grading Factors

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

7.5 Utility Trench Backfill

- 7.5.1 Utility trenches should be properly backfilled in accordance with the requirements of City of Jurupa Valley and the latest edition of the *Standard Specifications for Public Works Construction* (Greenbook). The pipes should be bedded with well graded crushed rock or clean sands (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe. The use of well-graded crushed rock is only acceptable if used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of 2-sack slurry and controlled low strength material (CLSM) are also acceptable. However, consideration should be given to the possibility of differential settlement where the slurry ends and earthen backfill begins. These transitions should be minimized, and additional stabilization should be considered at these transitions.
- 7.5.2 Utility excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, gravel, concrete, or geogrid.
- 7.5.3 During the rainy season, localized perched water conditions may develop that may require special consideration during earthwork operations. The contractor should be prepared to mitigate seepage and perched water conditions. Groundwater, seepage, and perched water are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result.

7.6 Seismic Design Criteria

- 7.6.1 The following table summarizes the site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *U.S. Seismic Design Maps*, provided by the Structural Engineers Association of California (SEAOC).
- 7.6.2 The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.5g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.6g	Figure 1613.2.1(3)
Site Coefficient, F_A	1	Table 1613.2.3(1)
Site Coefficient, F_V	1.7	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.5g	Section 1613.2.3 (Eqn 16-20)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.02g*	Section 1613.2.3 (Eqn 16-21)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1g	Section 1613.2.4 (Eqn 16-22)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.68g*	Section 1613.2.4 (Eqn 16-23)
*Per Supplement 3 of ASCE 7-16, a ground motion hazard analysis (GMHA) shall be performed for projects on Site Class "D" sites with 1-second spectral acceleration (S_1) greater than or equal to 0.2g, which is true for this site. However, Supplement 3 of ASCE 7-16 provides an exception stating that the GMHA may be waived provided that the parameter S_{M1} is increased by 50% for all applications of S_{M1} . The values for parameters S_{M1} and S_{D1} presented above have not been increased in accordance with Supplement 3 of ASCE 7-16.		

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.505g	Figure 22-9
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.555g	Section 11.8.3 (Eqn 11.8-1)

- 7.6.4 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 an 8.1 magnitude event occurring at a hypocentral distance of 13.46 kilometers from the site.
- 7.6.5 Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.7 Foundation Design

- 7.7.1 Subsequent to the recommended grading, a conventional shallow spread foundation system may be utilized for support of the anticipated structures (three-stories or less), provided foundations derive support on at least 3 feet of newly placed engineered fill as described in the *Grading* section of this report.
- 7.7.2 Conventional shallow spread foundations for the proposed structure should consist of continuous strip footings and/or isolated spread footings and should be designed using the parameters in the following table.

SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width	18 Inches
Minimum Isolated Foundation Width	24 Inches
Minimum Foundation Depth	18 Inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	4 No. 4 Bars, 2 Top and 2 Bottom
Allowable Bearing Pressure – Continuous Foundation	2,500 psf
Allowable Bearing Pressure – Isolated Foundation	3,000 psf
Bearing Pressure Increase	250 psf per Foot of Width
	500 psf per Foot of Depth
Maximum Allowable Bearing Pressure	4,000 psf
Estimated Combined Seismic and Static Settlement	¾ Inch
Estimated Differential Settlement	½ Inch over 40 Feet

- 7.7.3 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.
- 7.7.4 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.
- 7.7.5 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.
- 7.7.6 No special subgrade presaturation is required prior to placement of concrete. However, the moisture in the foundation subgrade should be sprinkled as necessary to maintain a moist condition at the time of concrete placement.
- 7.7.7 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 excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.
- 7.7.8 This office should be provided with a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.8 Miscellaneous Foundations

- 7.8.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 24 inches of newly placed engineered fill which extends laterally at least 24 inches beyond the foundation area. It is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundations supported therein.
- 7.8.2 If the soils exposed in the excavation bottom are soft or loose, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.
- 7.8.3 Miscellaneous foundations may be designed for a bearing value of 1,500 pounds per square and should be a minimum of 12 inches in width, 18 inches in depth below lowest adjacent grade, and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.
- 7.8.4 Foundation excavations should be observed and approved in writing by a representative of Geocon, prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.9 Exterior Concrete Flatwork

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

MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Reinforcing Steel* Options	Minimum Thickness
EI ≤ 20	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 18 inches on center, Both Directions	

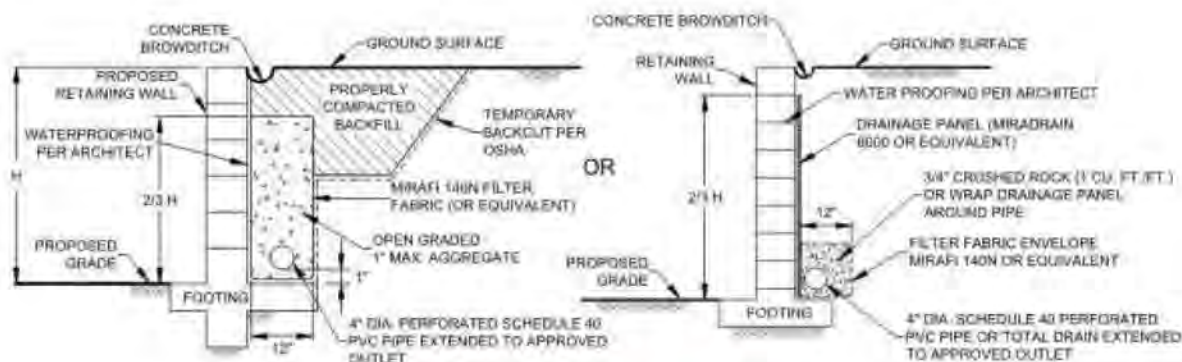
*In excess of 8 feet square.

- 7.9.2 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 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content in accordance with ASTM D1557.
- 7.9.3 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to potentially expansive soil beneath grade. The reinforcing steel should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.
- 7.9.4 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.
- 7.9.5 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.
- 7.9.6 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.

7.10 Retaining Wall Design

- 7.10.1 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.
- 7.10.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* section of this report.
- 7.10.3 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 36 pcf.
- 7.10.4 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 63 pcf.
- 7.10.5 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.
- 7.10.6 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, restrained walls is 95 pcf for the full height of the wall. The value includes hydrostatic pressures plus buoyant lateral earth pressures. If a partially drained wall is proposed, Geocon should be contacted to provide additional recommendations.
- 7.10.7 Additional 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.

- 7.10.8 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfilling, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.
- 7.10.9 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140N (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. Alternatively, a drainage panel, such as a Miradrain 6000 or equivalent, can be placed along the back of the wall, as shown in the figure below. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall.



Typical Retaining Wall Drainage Detail

- 7.10.10 The recommendations herein assume a properly compacted backfill (EI of 20 or less) with no hydrostatic forces or imposed surcharge load. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations.

7.11 Elevator Pit Design

- 7.11.1 If used, the elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit foundation and walls may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report.

- 7.11.2 Additional pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent foundations and should be designed for each condition as the project progresses.
- 7.11.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Design* section of this report.
- 7.11.4 We recommend that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the Geotechnical Engineer.

7.12 Elevator Piston

- 7.12.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation, or the drilled excavation could compromise the existing foundation support, especially if the drilling is performed subsequent to the foundation construction.
- 7.12.2 Caving is expected, and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer should be performed.
- 7.12.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

7.13 Lateral Design

- 7.13.1 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 0.4 may be used with the dead load forces in the newly placed engineered fill or undisturbed alluvial soils.
- 7.13.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill or undisturbed alluvial soils may be computed as an equivalent fluid having a density of 260 pounds per cubic foot (pcf) with a maximum earth pressure of 2,600 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. A one-third increase in the passive value may be used for wind or seismic loads.

7.14 Preliminary Pavement Recommendations

- 7.14.1 The final pavement design should be based on R-value testing of soils at subgrade. Streets should be designed in accordance with the City of Jurupa Valley standards and specifications when final Traffic Indices and R-Value test results of subgrade soil are completed. For preliminary design purposes, we used an estimated R-value of 30 based on the soil classifications. Pavements should meet the minimum requirement for asphalt thickness in City of Jurupa Valley *Roadway Structural Requirements* (Standard No. 120). Preliminary flexible pavement sections are presented in the table below. Geocon should be contacted if other roadway classifications and traffic indices are appropriate for the project.

PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Road Classification	Traffic Index	Asphalt Concrete (inches)	Aggregate Base (inches)
Local Road	5.5	3.0	7.0
Local Collector	7.0	4.0	9.5
Industrial Collector/Secondary Major	9.0	5.5	13.0

- 7.14.2 Prior to construction of new pavement sections, remedial grading should be performed in accordance with the *Grading* section in this report.
- 7.14.3 The upper 12 inches of the subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content beneath pavement sections.
- 7.14.4 Aggregate base materials should conform to the requirements for Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB) in Sections 200-2.2 and 200-2.4, respectively, of the latest edition of the *Standard Specifications for Public Works Construction* (Greenbook). Class 2 aggregate base materials should conform to Section 26-1.02A of the latest edition of the Caltrans *Standard Specifications*. Base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at or slightly above optimum moisture content in accordance with ASTM D1557. Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density in accordance with ASTM D1561.

- 7.14.5 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330-21 *Commercial Concrete Parking Lots and Site Paving Design and Construction – Guide*. The following table provides the traffic categories and design parameters used for the calculations for 20-year design life.

TRAFFIC CATEGORIES

Traffic Category	Description	Reliability (%)	Slabs Cracked at End of Design Life (%)
A	Car Parking Areas and Access Lanes	60	15
B	Entrance and Truck Service Lanes	60	15
C	School or City Buses (Excluding Large Articulated Buses)	75	15
D	Heavy Duty Trucks (Gross Weight of 80 Kips)	75	15
E	Garbage or Fire Truck Lane	75	15

- 7.14.6 We used the parameters presented in the *Traffic Categories* table above to calculate the pavement design sections. We should be contacted to provide updated design sections, if necessary.

RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of Subgrade Reaction, k	100 pci
Modulus of Rupture for Concrete, M_R	550 psi
Concrete Compressive Strength	3,500 psi
Concrete Modulus of Elasticity, E	3,712,500 psi

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

RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Traffic Category	Trucks Per Day	Portland Cement Concrete, T (Inches)
A = Car Parking Areas and Access Lanes	10	4½
B = Entrance and Truck Service Lanes	10	5¼
	25	5½
	50	5¾
C = School or City Buses	10	8
	50	8½
D = Heavy Duty Trucks	50	6¼
	100	6¾
E = Garbage or Fire Truck Lanes	5	6¼
	10	6½

- 7.14.8 The PCC vehicular pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content. The garbage truck pad should be large enough such that all wheels are on the concrete pad during the loading operations.
- 7.14.9 Adequate joint spacing should be incorporated into the design and construction of the rigid pavement in accordance with the following table.

MAXIMUM JOINT SPACING

Pavement Thickness, T (Inches)	Maximum Joint Spacing (Feet)
4<T<5	10
5≤T<6	12½
6≤T	15

- 7.14.10 The rigid pavement should also be designed and constructed incorporating the parameters presented in the following table.

ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

Subject	Value
Thickened Edge	1.2 Times Slab Thickness Adjacent to Structures
	1.5 Times Slab Thickness Adjacent to Soil
	Minimum Increase of 2 Inches
	4 Feet Wide
Crack Control Joint Depth	Early Entry Sawn = $T/6$ to $T/5$, 1.25 Inch Minimum
	Conventional (Tooled or Conventional Sawing) = $T/4$ to $T/3$
Crack Control Joint Width	$\frac{1}{4}$ -Inch for Sealed Joints and Per Sealer Manufacturer's Recommendations
	$\frac{1}{16}$ - to $\frac{1}{4}$ -Inch is Common for Unsealed Joints

- 7.14.11 Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.
- 7.14.12 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be in accordance with the referenced ACI guide.
- 7.14.13 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab.
- 7.14.14 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Cross-gutters that receive vehicular should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials should not be placed below the curb/gutter, or cross-gutters so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb/gutter, the concrete flatwork should be structurally connected to the curbs to help reduce the potential for offsets between the curbs and the flatwork.

7.15 Temporary Excavations

- 7.15.1 Excavations of up to 10 feet in height may be required during earthwork operations. The excavations are expected to expose artificial fill and alluvial deposits, which may contain material that could be subject to caving. The contractor should be prepared for caving soils in open excavations, and formwork may be required in foundation excavations. Vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent foundations or traffic, or where caving soils are not present.
- 7.15.2 Excavations of up to 10 feet in height are expected for construction of the proposed utility improvements, and we expect that the proposed utilities will be installed with conventional cut-and-cover methods.
- 7.15.3 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping and/or shoring measures in order to provide a stable excavation. The contractor's competent person should evaluate the appropriate slope and/or shoring based on soil type and prevailing site conditions per Cal-OSHA regulations.
- 7.15.4 Where there is insufficient space for sloped excavations, shoring or trench shields should be used to support excavations. Shoring may also be necessary where sloped excavation could remove vertical or lateral support of existing improvements, including existing utilities and adjacent structures. The contractor's competent person should evaluate the appropriate shoring system to provide per Cal-OSHA regulations.
- 7.15.5 Where temporary construction 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 competent person should inspect the soils exposed in the cut slopes during excavation in accordance with Cal-OSHA regulations so that modifications of the slopes can be made if variations in the soil conditions occur.
- 7.15.6 We expect that braced shoring, such as conventionally braced shields, cross-braced hydraulic shoring, or driven sheet piles may be utilized. The selection of the shoring system is the responsibility of the contractor. Shoring systems should be designed by a California licensed civil or structural engineer with experience in designing shoring systems.

- 7.15.7 We recommend that an equivalent fluid pressure based on the following table be utilized for design of temporary shoring. These pressures are based on the assumption that the shoring is supporting a level backfill and there are no hydrostatic pressures above the bottom of the excavation.

RECOMMENDED SHORING PRESSURES

HEIGHT OF SHORED EXCAVATION (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (Active Pressure)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
≤ 10	27	17H

- 7.15.8 Active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure or where braced shoring will be utilized, the at-rest pressure should be considered for design purposes.
- 7.15.9 Additional pressure should be added for a surcharge condition due to sloping ground, construction equipment, vehicular traffic, or adjacent structures and should be designed for each condition as the project progresses.
- 7.15.10 In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to roadways or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. Higher surcharge loads may be required to account for construction equipment.
- 7.15.11 It is difficult to accurately predict the amount of deflection of a shored embankment, but some deflection will occur. We recommend that the deflection be minimized to prevent damage to existing structures and adjacent improvements. Where a public right-of-way is present or adjacent offsite structures do not surcharge the shoring excavation, the shoring deflection should be limited to less than 1 inch at the top of the shored embankment. Where offsite structures are within the shoring surcharge area it is recommended that the beam deflection be limited to less than ½ inch at the elevation of the adjacent offsite foundation, and no deflection at all if deflections will damage existing structures. The allowable deflection is dependent on many factors, such as the presence of structures and utilities near the top of the embankment and will be assessed and designed by the project shoring engineer.

7.16 Site Drainage and Moisture Protection

- 7.16.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion, soil collapse potential, and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2022 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.
- 7.16.2 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.
- 7.16.3 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall 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.
- 7.16.4 If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to infiltration areas. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeology study at the site. Down-gradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration. We recommend that infiltration systems be kept at least 25 feet away from existing or proposed buildings and at least 15 feet away from existing or proposed improvements.

7.17 Plan Review

- 7.17.1 Grading and structural/foundation plans should be reviewed by Geocon prior to finalization 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 the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous materials was not part of the scope of services provided by Geocon.

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.

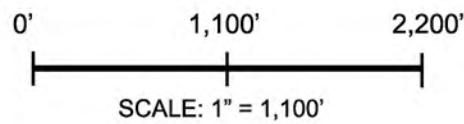
The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are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

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SOURCE: Google Earth 2024

VICINITY MAP

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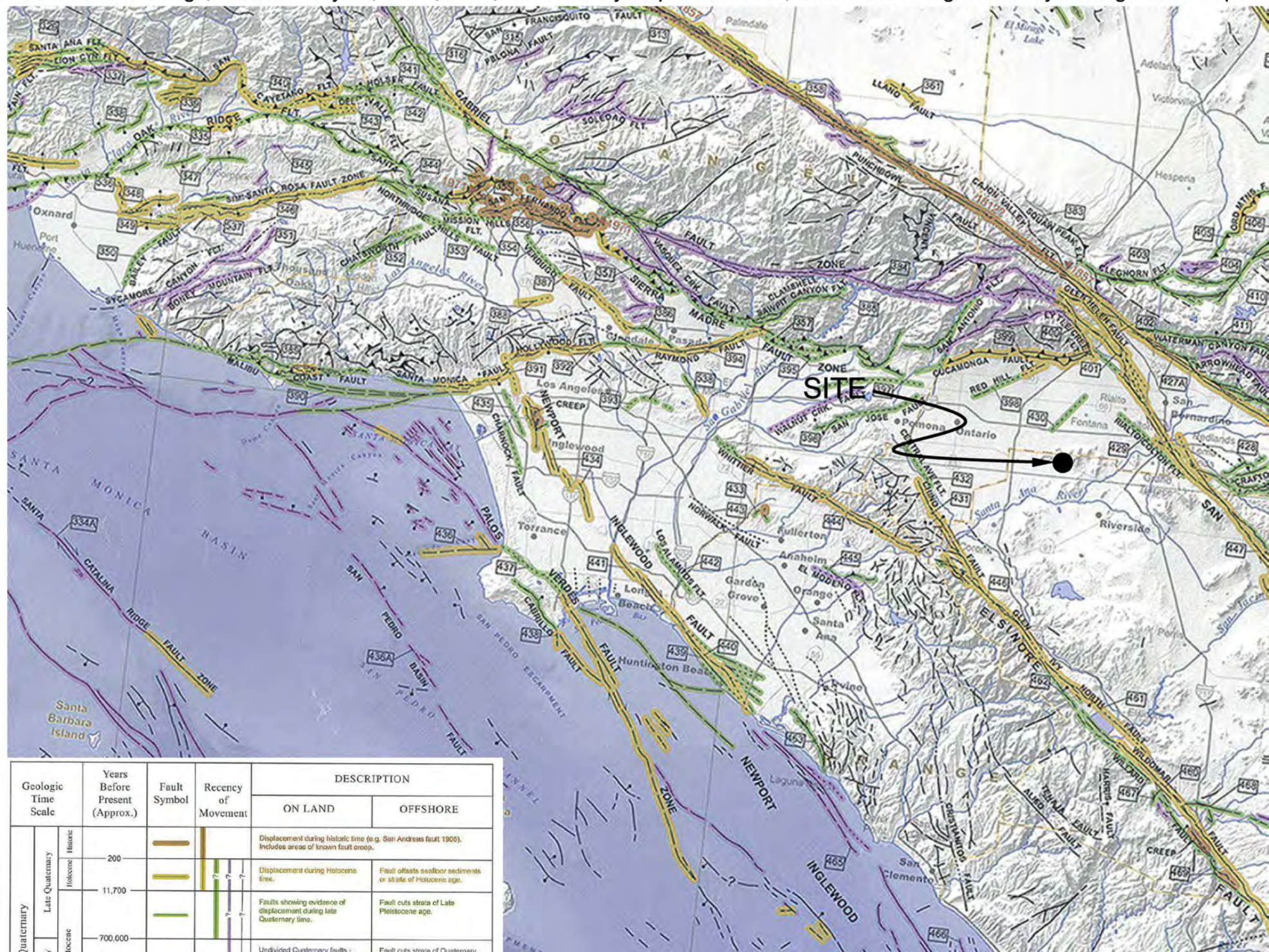
JUNE 2024

PROJECT NO. T2875-22-02

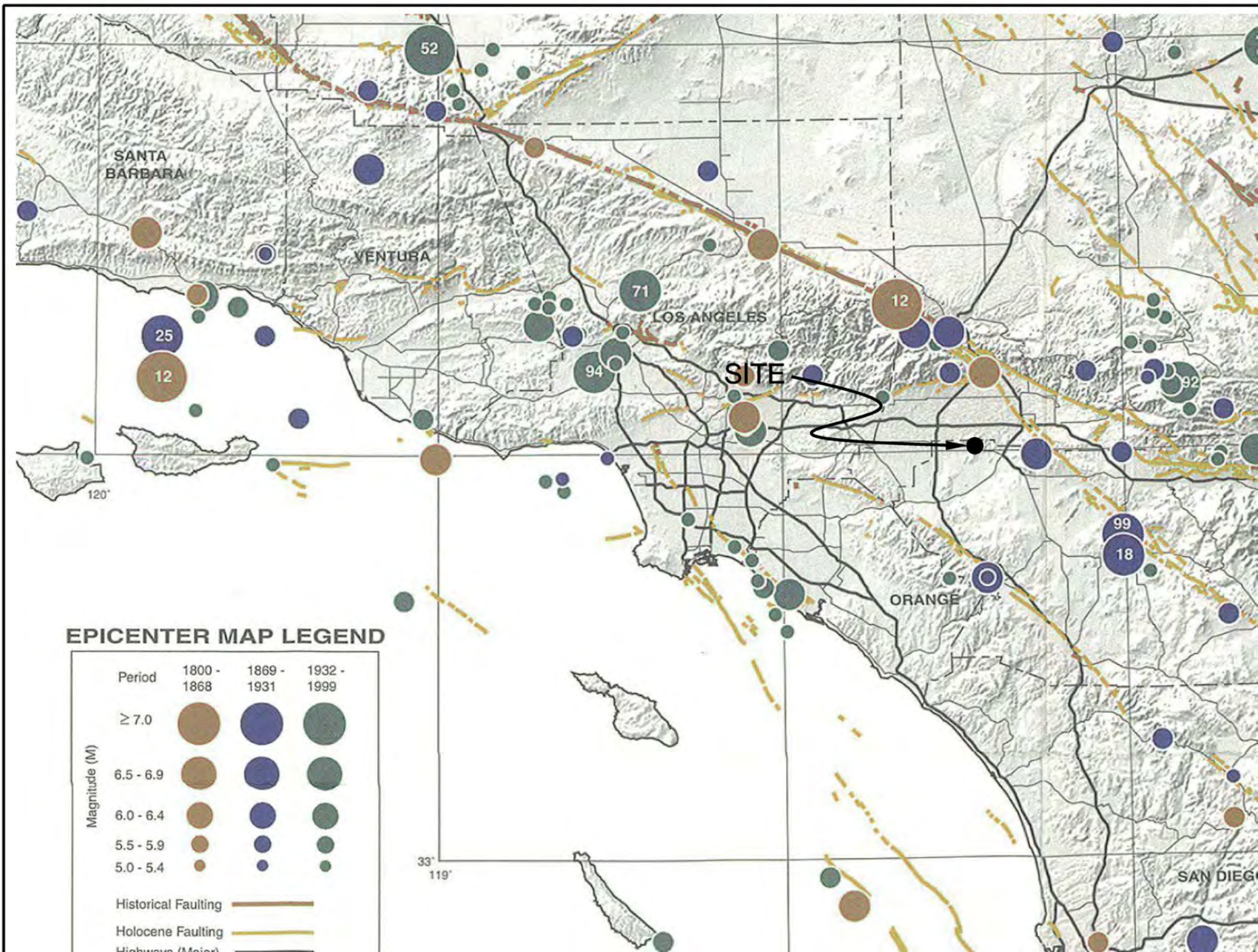
FIG. 1



Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No.



Geologic Time Scale	Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
				ON LAND	OFFSHORE
Quaternary	Late Quaternary	Holocene	200	Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
				Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
	Pleistocene	11,700	7	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
				Undivided Quaternary faults -	Fault cuts strata of Quaternary



EPICENTER MAP LEGEND

Period	1800 - 1868	1869 - 1931	1932 - 1999
≥ 7.0			
6.5 - 6.9			
6.0 - 6.4			
5.5 - 5.9			
5.0 - 5.4			

Magnitude (M)

Historical Faulting

Holocene Faulting

Highways (Major)

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	8.10
Peak Horiz. Acceleration PGA_M (g):	0.555
Magnitude Scaling Factor:	0.821
Historic High Groundwater:	100.0
Groundwater Depth During Explore:	100.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len. Corr. (CR) (0-no or 1-yes):	1
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0-no or 1-yes):	1

Boring ID: B-1

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)60cs	Eff. Unit Wt. (pcf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe Fact.
1.0	132.5	0	11	5.0	1		71	1.700	20.5	132.5	0.222	1.000	0.361	--
2.0	132.5	0	11	5.0	1		71	1.700	20.5	132.5	0.222	0.998	0.360	--
3.0	132.5	0	11	5.0	1		71	1.700	20.5	132.5	0.222	0.996	0.359	--
4.0	132.5	0	11	5.0	1		71	1.700	20.5	132.5	0.222	0.994	0.359	--
5.0	132.5	0	11	5.0	1	44	71	1.700	29.6	132.5	0.441	0.991	0.358	--
6.0	132.5	0	11	5.0	1	44	71	1.692	29.5	132.5	0.434	0.989	0.357	--
7.0	132.5	0	11	5.0	1	44	71	1.557	27.5	132.5	0.353	0.987	0.356	--
8.0	129.8	0	13	7.5	1	44	73	1.450	30.9	129.8	Infin.	0.985	0.355	--
9.0	129.8	0	13	7.5	1	44	73	1.364	29.4	129.8	0.428	0.982	0.354	--
10.0	129.8	0	13	7.5	1	44	73	1.291	28.1	129.8	0.372	0.980	0.354	--
11.0	129.7	0	16	10.0	1	44	76	1.229	31.1	129.7	Infin.	0.978	0.353	--
12.0	129.7	0	16	10.0	1	44	76	1.175	30.0	129.7	0.467	0.976	0.352	--
13.0	129.7	0	20	12.5	1	44	81	1.128	34.7	129.7	Infin.	0.974	0.351	--
14.0	129.7	0	20	12.5	1	44	81	1.086	33.6	129.7	Infin.	0.972	0.351	--
15.0	129.7	0	20	12.5	1	44	81	1.048	32.6	129.7	Infin.	0.970	0.350	--
16.0	129.7	0	17	15.0	1	30	73	1.014	28.8	129.7	0.400	0.967	0.349	--
17.0	129.7	0	17	15.0	1	30	73	0.983	28.0	129.7	0.371	0.965	0.348	--
18.0	124.7	0	17	15.0	1	30	73	0.955	27.4	124.7	0.350	0.963	0.347	--
19.0	124.7	0	17	15.0	1		73	0.930	19.1	124.7	0.205	0.961	0.347	--
20.0	124.7	0	17	15.0	1		73	0.907	18.7	124.7	0.199	0.958	0.346	--
21.0	124.7	0	11	20.0	1		54	0.886	13.1	124.7	0.141	0.956	0.345	--
22.0	124.7	0	11	20.0	1		54	0.866	12.8	124.7	0.138	0.953	0.344	--
23.0	140.1	0	50	25.0	1		108	0.846	60.6	140.1	Infin.	0.950	0.343	--
24.0	140.1	0	50	25.0	1		108	0.826	59.2	140.1	Infin.	0.947	0.342	--
25.0	140.1	0	50	25.0	1		108	0.808	57.9	140.1	Infin.	0.944	0.340	--
26.0	140.1	0	50	25.0	1		108	0.791	56.7	140.1	Infin.	0.940	0.339	--
27.0	140.1	0	50	25.0	1		108	0.775	55.5	140.1	Infin.	0.936	0.338	--
28.0	140.1	0	50	25.0	1		108	0.760	54.4	140.1	Infin.	0.932	0.336	--
29.0	140.1	0	50	25.0	1		108	0.745	53.4	140.1	Infin.	0.928	0.335	--
30.0	140.1	0	50	25.0	1		108	0.732	52.4	140.1	Infin.	0.923	0.333	--
31.0	130.9	0	50	25.0	1		108	0.719	51.5	130.9	Infin.	0.918	0.331	--
32.0	130.9	0	50	25.0	1		108	0.708	50.7	130.9	Infin.	0.912	0.329	--
33.0	130.9	0	50	25.0	1		108	0.697	49.9	130.9	Infin.	0.907	0.327	--
34.0	130.9	0	50	25.0	1		108	0.687	49.2	130.9	Infin.	0.900	0.325	--
35.0	130.9	0	50	25.0	1		108	0.677	48.5	130.9	Infin.	0.894	0.322	--
36.0	130.9	0	34	35.0	1		79	0.667	34.0	130.9	Infin.	0.887	0.320	--
37.0	130.9	0	34	35.0	1		79	0.658	33.6	130.9	Infin.	0.880	0.317	--
38.0	130.9	0	34	35.0	1		79	0.649	33.1	130.9	Infin.	0.872	0.314	--
39.0	130.9	0	34	35.0	1		79	0.641	32.7	130.9	Infin.	0.864	0.312	--
40.0	130.9	0	34	35.0	1		79	0.633	32.3	130.9	Infin.	0.855	0.309	--
41.0	130.9	0	59	40.0	1	31	99	0.625	69.1	130.9	Infin.	0.846	0.305	--
42.0	130.9	0	59	40.0	1	31	99	0.618	68.3	130.9	Infin.	0.837	0.302	--
43.0	130.9	0	59	40.0	1	31	99	0.610	67.6	130.9	Infin.	0.828	0.299	--
44.0	130.9	0	59	40.0	1	31	99	0.603	66.8	130.9	Infin.	0.818	0.295	--
45.0	130.9	0	59	40.0	1	31	99	0.597	66.2	130.9	Infin.	0.808	0.292	--
46.0	130.9	0	19	45.0	1	31	54	0.590	24.3	130.9	0.279	0.798	0.288	--
47.0	130.9	0	19	45.0	1	31	54	0.584	24.1	130.9	0.275	0.788	0.284	--
48.0	130.9	0	19	45.0	1	31	54	0.578	23.9	130.9	0.272	0.778	0.281	--
49.0	130.9	0	19	45.0	1	31	54	0.572	23.7	130.9	0.268	0.768	0.277	--
50.0	130.9	0	19	45.0	1	31	54	0.566	23.5	130.9	0.265	0.757	0.273	--
51.0	130.9	0	29	50.0	1	31	64	0.560	33.1	130.9	Infin.	0.747	0.270	--

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LIQUEFACTION POTENTIAL - MAXIMUM CONSIDERED EARTHQUAKE

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER - ADESA SITE - 6464 33RD STREET
JURUPA VALLEY, CALIFORNIA

DRAFTED BY: ADL

CHECKED BY: PZ

JUNE 2024

PROJECT NO. T3072-22-02

FIG. 5

MCE EARTHQUAKE INFORMATION:

Earthquake Magnitude:	8.10
Peak Horiz. Acceleration (g):	0.555

Boring ID: B-1

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress (TsV)	Field SPT (N)	Correction Factor (Car)	Relative Density (Dr) (%)	Correction Factor (Cn)	Corrected (N)160	r _d Factor	Maximum Shear Mod. (Gmax) (tsf)	[γ _{eff}]/[G _{max}] (Gmax)	γ _{eff} Shear Strain	[γ _{eff}]*100%	Volume Strain MT.5 (V _{VS}) (%)	Number of Strain Cycles (Nc)	Corrected Vol. Strains (E _c)	Estimated Settlement (S) (inches)
1.0	1.0	0.5	132.5	0.03	0.02	0.012	10.71	1.25	70.6	1.7	20.5	1.0	182.2	6.48E-05	1.00E-04	0.010	9.72E-03	21.4	1.14E-02	0.00
2.0	1.0	1.5	132.5	0.10	0.07	0.036	10.71	1.25	70.6	1.7	20.5	1.0	315.6	1.10E-04	2.30E-04	0.023	2.24E-02	21.4	2.62E-02	0.01
3.0	1.0	2.5	132.5	0.17	0.11	0.060	10.71	1.25	70.6	1.7	20.5	1.0	407.5	1.40E-04	1.70E-04	0.017	1.65E-02	21.4	1.94E-02	0.00
4.0	1.0	3.5	132.5	0.23	0.16	0.084	10.71	1.25	70.6	1.7	20.5	1.0	482.1	1.62E-04	1.70E-04	0.017	1.65E-02	21.4	1.94E-02	0.00
5.0	1.0	4.5	132.5	0.30	0.20	0.107	10.71	1.25	70.6	1.7	29.6	1.0	617.9	1.56E-04	1.70E-04	0.017	1.08E-02	21.4	1.25E-02	0.00
6.0	1.0	5.5	132.5	0.36	0.24	0.131	10.71	1.25	70.6	1.7	29.5	1.0	682.3	1.73E-04	1.50E-04	0.015	9.42E-03	21.4	1.10E-02	0.00
7.0	1.0	6.5	132.5	0.43	0.29	0.156	10.71	1.25	70.6	1.6	27.5	1.0	724.9	1.88E-04	1.50E-04	0.015	1.02E-02	21.4	1.20E-02	0.00
8.0	1.0	7.5	129.8	0.50	0.33	0.178	13.23	1.25	73.1	1.5	30.9	1.0	808.9	1.92E-04	1.50E-04	0.015	8.00E-03	21.4	1.04E-02	0.00
9.0	1.0	8.5	129.8	0.56	0.38	0.201	13.23	1.25	73.1	1.4	29.4	1.0	845.6	2.03E-04	4.50E-04	0.045	2.84E-02	21.4	3.33E-02	0.01
10.0	1.0	9.5	129.8	0.63	0.42	0.224	13.23	1.25	73.1	1.3	28.1	1.0	879.8	2.14E-04	4.50E-04	0.045	3.00E-02	21.4	3.51E-02	0.01
11.0	1.0	10.5	129.7	0.69	0.46	0.247	15.75	1.25	76.5	1.2	31.1	1.0	956.6	2.13E-04	4.50E-04	0.045	2.65E-02	21.4	3.10E-02	0.01
12.0	1.0	11.5	129.7	0.76	0.51	0.269	15.75	1.25	76.5	1.2	30.0	1.0	968.3	2.22E-04	3.70E-04	0.037	2.28E-02	21.4	2.67E-02	0.01
13.0	1.0	12.5	129.7	0.82	0.55	0.291	19.53	1.25	80.5	1.1	34.7	1.0	1081.5	2.16E-04	3.70E-04	0.037	1.91E-02	21.4	2.24E-02	0.01
14.0	1.0	13.5	129.7	0.89	0.59	0.314	19.53	1.25	80.5	1.1	33.6	1.0	1111.3	2.23E-04	2.70E-04	0.027	1.96E-02	21.4	2.33E-02	0.01
15.0	1.0	14.5	129.7	0.95	0.64	0.336	19.53	1.25	80.5	1.0	32.6	1.0	1139.8	2.29E-04	2.70E-04	0.027	2.06E-02	21.4	2.41E-02	0.01
16.0	1.0	15.5	129.7	1.02	0.66	0.356	17	1.25	72.6	1.0	28.8	1.0	1129.6	2.43E-04	3.70E-04	0.037	2.38E-02	21.4	2.80E-02	0.01
17.0	1.0	16.5	129.7	1.08	0.72	0.379	17	1.25	72.6	1.0	28.0	1.0	1155.3	2.48E-04	3.70E-04	0.037	2.47E-02	21.4	2.89E-02	0.01
18.0	1.0	17.5	124.7	1.14	0.77	0.400	17	1.25	72.6	1.0	27.4	1.0	1179.5	2.53E-04	3.70E-04	0.037	2.54E-02	21.4	2.96E-02	0.01
19.0	1.0	18.5	124.7	1.21	0.81	0.421	17	1.25	72.6	0.9	19.1	1.0	1074.7	2.88E-04	3.70E-04	0.037	3.90E-02	21.4	4.57E-02	0.01
20.0	1.0	19.5	124.7	1.27	0.86	0.441	17	1.25	72.6	0.8	18.7	1.0	1092.9	2.93E-04	3.70E-04	0.037	4.02E-02	21.4	4.72E-02	0.01
21.0	1.0	20.5	124.7	1.33	0.89	0.461	11	1.25	54.1	0.9	13.1	1.0	994.5	3.32E-04	7.10E-04	0.071	1.18E-01	21.4	1.39E-01	0.03
22.0	1.0	21.5	124.7	1.39	0.93	0.480	11	1.25	54.1	0.9	12.6	1.0	1009.8	3.37E-04	7.10E-04	0.071	1.21E-01	21.4	1.42E-01	0.03
23.0	1.0	22.5	140.1	1.46	0.98	0.501	50	1.25	107.8	0.8	60.6	0.9	1736.1	2.02E-04	3.70E-04	0.037	9.78E-03	21.4	1.15E-02	0.00
24.0	1.0	23.5	140.1	1.53	1.02	0.523	50	1.25	107.8	0.8	59.2	0.9	1763.5	2.05E-04	3.00E-04	0.030	8.16E-03	21.4	9.57E-03	0.00
25.0	1.0	24.5	140.1	1.60	1.07	0.544	50	1.25	107.8	0.8	57.9	0.9	1790.0	2.08E-04	3.00E-04	0.030	6.38E-03	21.4	6.83E-03	0.00
26.0	1.0	25.5	140.1	1.67	1.12	0.566	50	1.25	107.8	0.8	56.7	0.9	1815.8	2.11E-04	3.00E-04	0.030	8.60E-03	21.4	1.01E-02	0.00
27.0	1.0	26.5	140.1	1.74	1.17	0.586	50	1.25	107.8	0.8	55.5	0.9	1840.8	2.13E-04	3.00E-04	0.030	8.81E-03	21.4	1.03E-02	0.00
28.0	1.0	27.5	140.1	1.81	1.21	0.607	50	1.25	107.8	0.8	54.4	0.9	1865.2	2.15E-04	3.00E-04	0.030	9.03E-03	21.4	1.06E-02	0.00
29.0	1.0	28.5	140.1	1.88	1.26	0.627	50	1.25	107.8	0.7	53.4	0.9	1889.0	2.17E-04	3.00E-04	0.030	9.23E-03	21.4	1.08E-02	0.00
30.0	1.0	29.5	140.1	1.95	1.31	0.647	50	1.25	107.8	0.7	52.4	0.9	1912.2	2.19E-04	3.00E-04	0.030	9.44E-03	21.4	1.11E-02	0.00
31.0	1.0	30.5	130.9	2.02	1.35	0.669	50	1.25	107.8	0.7	51.5	0.9	1934.1	2.21E-04	3.00E-04	0.030	9.63E-03	21.4	1.13E-02	0.00
32.0	1.0	31.5	130.9	2.08	1.40	0.684	50	1.25	107.8	0.7	50.7	0.9	1954.6	2.22E-04	3.00E-04	0.030	9.82E-03	21.4	1.15E-02	0.00
33.0	1.0	32.5	130.9	2.15	1.44	0.701	50	1.25	107.8	0.7	49.9	0.9	1975.0	2.24E-04	3.00E-04	0.030	1.00E-02	21.4	1.17E-02	0.00
34.0	1.0	33.5	130.9	2.21	1.48	0.718	50	1.25	107.8	0.7	49.2	0.9	1994.9	2.25E-04	3.00E-04	0.030	1.02E-02	21.4	1.19E-02	0.00
35.0	1.0	34.5	130.9	2.28	1.53	0.735	50	1.25	107.8	0.7	48.5	0.9	2014.3	2.26E-04	3.00E-04	0.030	1.04E-02	21.4	1.22E-02	0.00
36.0	1.0	35.5	130.9	2.34	1.57	0.751	34	1.25	79.1	0.7	34.0	0.9	1815.6	2.54E-04	3.00E-04	0.030	1.59E-02	21.4	1.86E-02	0.00
37.0	1.0	36.5	130.9	2.41	1.61	0.767	34	1.25	79.1	0.7	33.6	0.9	1832.3	2.55E-04	3.00E-04	0.030	1.61E-02	21.4	1.89E-02	0.00
38.0	1.0	37.5	130.9	2.49	1.66	0.783	34	1.25	79.1	0.6	33.1	0.9	1848.7	2.56E-04	3.00E-04	0.030	1.64E-02	21.4	1.92E-02	0.00
39.0	1.0	38.5	130.9	2.54	1.70	0.798	34	1.25	79.1	0.6	32.7	0.9	1864.9	2.56E-04	3.00E-04	0.030	1.66E-02	21.4	1.95E-02	0.00
40.0	1.0	39.5	130.9	2.61	1.75	0.813	34	1.25	79.1	0.6	32.3	0.9	1880.8	2.57E-04	3.00E-04	0.030	1.69E-02	21.4	1.98E-02	0.00
41.0	1.0	40.5	130.9	2.67	1.79	0.826	59	1.25	99.4	0.8	69.1	0.8	2454.0	1.99E-04	1.30E-04	0.013	2.94E-03	21.4	3.44E-03	0.00
42.0	1.0	41.5	130.9	2.74	1.83	0.842	59	1.25	99.4	0.8	68.3	0.8	2474.6	1.99E-04	1.30E-04	0.013	2.98E-03	21.4	3.49E-03	0.00
43.0	1.0	42.5	130.9	2.80	1.88	0.856	59	1.25	99.4	0.6	67.6	0.8	2494.8	2.00E-04	1.30E-04	0.013	3.02E-03	21.4	3.54E-03	0.00
44.0	1.0	43.5	130.9	2.87	1.92	0.870	59	1.25	99.4	0.6	66.8	0.8	2514.8	2.00E-04	1.30E-04	0.013	3.06E-03	21.4	3.58E-03	0.00
45.0	1.0	44.5	130.9	2.93	1.97	0.883	59	1.25	99.4	0.6	66.2	0.8	2534.4	2.00E-04	1.30E-04	0.013	3.09E-03	21.4	3.63E-03	0.00
46.0	1.0	45.5	130.9	3.00	2.01	0.896	19	1.25	54.0	0.6	24.3	0.8	1835.6	2.76E-04	3.00E-04	0.030	2.37E-02	21.4	2.78E-02	0.01
47.0	1.0	46.5	130.9	3.06	2.06	0.909	19	1.25	54.0	0.6	24.1	0.8	1850.3	2.76E-04	3.00E-04	0.030	2.39E-02	21.4	2.80E-02	0.01
48.0	1.0	47.5	130.9	3.13	2.10	0.921	19	1.25	54.0	0.6	23.9	0.8	1864.7	2.76E-04	3.00E-04	0.030	2.41E-02	21.4	2.82E-02	0.01
49.0	1.0	48.5	130.9	3.20	2.14	0.933	19	1.25	54.0	0.6	23.7	0.8	1878.9	2.76E-04	3.00E-04	0.030	2.43E-02	21.4	2.84E-02	0.01
50.0	1.0	49.5	130.9	3.26	2.18	0.945	19	1.25	54.0	0.6	23.5	0.8	1892.9	2.76E-04	3.00E-04	0.030	2.45E-02	21.4	2.86E-02	0.01

TOTAL SETTLEMENT (inches) = 0.34

REFERENCE: TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9

GEOCON
WEST, INC.ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE - SUITE 101 - MURRIETA, CA 92562
PHONE: 909-894-2175EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS
IN DRY SANDY SOILS

MAXIMUM CONSIDERED EARTHQUAKE

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER - ADESA SITE - 6464 33RD STREET
JURUPA VALLEY, CALIFORNIA

DRAFTED BY: ADL

CHECKED BY: PZ

JUNE 2024

PROJECT NO. T3072-22-02

FIG. 6

APPENDIX

A

APPENDIX A

FIELD INVESTIGATION

Geocon performed the field investigation on May 16, 2024, which included the drilling of eight geotechnical borings to depths of approximately 20 to 51½ feet below existing grade.

The geotechnical borings were drilled with a CME truck-mounted drill rig equipped with an 8-inch-diameter hollow-stem auger. The geotechnical borings were drilled to observe the subsurface geological conditions at the site, collect relatively undisturbed in-situ and disturbed bulk samples for laboratory testing, and evaluate the depth to groundwater, if any. We collected bulk and relatively undisturbed samples from the borings by driving a 3-inch O. D., California Modified Sampler or Standard Penetration Test Sampler (SPT) into the “undisturbed” soil mass with blows from a 140-pound hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch inside diameter brass sampler rings to facilitate removal and testing. Relatively undisturbed samples and bulk samples of disturbed soils were transported to our laboratory for testing. The soil conditions encountered were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the geotechnical borings are presented on Figures A-1 through A-8.

The approximate locations of the soil borings are depicted on the *Geologic Map*, Figure 2.

T2875-22-02 BORING LOGS.GPJ

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.

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-1 ELEV. (MSL.) <u>883</u> DATE COMPLETED <u>5/16/2024</u> EQUIPMENT <u>CME 75</u>
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**Figure A-1,
Log of Boring B-1, Page 2 of 2**

T2875-22-02 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.

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-2 ELEV. (MSL.)882 DATE COMPLETED 5/16/2024 EQUIPMENTCME 75 BY: ADL	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
				SM	PAVEMENT SECTION			
	B-2@1-5'				4" AC			
2				SM	Hand auger upper 5'			
					ARTIFICIAL FILL			
					Silty SAND, loose, moist, reddish brown; fine to coarse sand			
4					OLDER ALLUVIUM (Qof)			
	B-2@5'				Silty SAND, medium dense, reddish brown; fine to medium sand	25	126.1	10.8
6								
8	B-2@7.5'				-Becomes brown	25	118.9	7.0
10	B-2@10'					30	121.5	5.9
12								
14								
16	B-2@15'				-Becomes strong brown; fine sand	28	108.9	8.4
18								
20	B-2@20'				-Sand seams	34	122.1	8.2
					Total Depth = 21 1/2 feet			
					Groundwater not encountered			
					Penetration resistance for 140-lb hammer falling 30 inches by auto hammer			
					Backfilled with cuttings 5/16/2024			

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

T2875-22-02 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.

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-3 ELEV. (MSL.)879 DATE COMPLETED 5/16/2024 EQUIPMENTCME 75 BY: ADL	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
					PAVEMENT SECTION			
				SM	4" AC			
2	B-3@1-4'			SM	ARTIFICIAL FILL			
					Silty SAND, loose, moist, orangish brown; fine to coarse sand			
	B-3@2.5'				OLDER ALLUVIUM (Qof)	10	117.7	10.9
4					Silty SAND, loose, moist, reddish brown; fine to coarse sand			
	B-3@5'					11	112.3	9.9
6								
8	B-3@7.5'				-Becomes fine to coarse sand; less silt	12	111.3	4.6
10	B-3@10'				-Becomes medium dense, orangish brown	22	111.4	3.2
12								
14								
16	B-3@15'			ML	Sandy SILT, stiff, moist, strong brown; fine sand	37	112.7	7.9
18								
20	B-3@20'			SM	Silty SAND, medium dense, moist, orangish brown, fine to medium sand	37	122.4	8.3
					Total Depth = 21 1/2 feet Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 5/16/2024			

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

T2875-22-02 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.

T2875-22-02 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

GEOCON

T2875-22-02 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

GEOCON

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-6 ELEV. (MSL.) <u>882</u> DATE COMPLETED <u>5/16/2024</u> EQUIPMENT <u>CME 75</u> BY: <u>ADL</u>	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
				SM	PAVEMENT SECTION 6" AC			
2	B-6@2-4' B-6@2.5'			SM	ARTIFICIAL FILL Silty SAND, loose, moist, reddish brown; fine to coarse sand	7	116.4	8.6
4					OLDER ALLUVIUM (Qof) Silty SAND, medium dense, moist, reddish brown; fine to medium sand			
6								
8	B-6@7.5'				-Becomes strong brown; fine sand	18	113.1	9.7
10								
12	B-6@12.5'					33	123.0	7.5
14								
16								
18	B-6@18.5'					47	124.0	8.4
20					Total Depth = 20 feet Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 5/16/2024			

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

T2875-22-02 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.

T2875-22-02 BORING LOGS.GPJ

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.

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-8		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) <u>893</u>	DATE COMPLETED <u>5/16/2024</u>			
					EQUIPMENT <u>CME 75</u> BY: <u>ADL</u>				
					MATERIAL DESCRIPTION				
0					PAVEMENT SECTION				
	B-8@1-5'			SM	6" AC				
2				SM	ARTIFICIAL FILL				
					Silty SAND, loose, moist, reddish brown; fine to coarse sand				
4					OLDER ALLUVIUM (Qof)				
					Silty SAND, medium dense, reddish brown; fine to medium sand				
6	B-8@5'						27	120.9	11.0
8					-Becomes orangish brown; fine sand				
10	B-8@10'						23	121.1	8.6
12									
14									
16	B-8@15'						59	122.0	10.5
18									
20	B-8@20'				- Fine to medium sand		45	124.0	4.4
					Total Depth = 21 1/2 feet Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 5/16/2024				

**Figure A-8,
Log of Boring B-8, Page 1 of 1**

T2875-22-02 BORING LOGS.GPJ

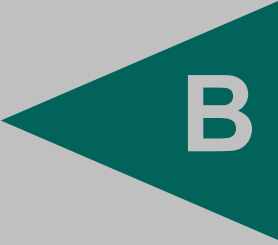
SAMPLE SYMBOLS

 ... 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

APPENDIX

**B**

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with current, generally accepted test methods of ASTM International (ASTM) or other suggested procedures. We analyzed selected soil samples for expansion potential, corrosion, grain size distribution, consolidation, collapse potential, direct shear, and maximum dry density and optimum moisture content. The results of the laboratory tests are presented on Figures B-1 through B-16. The in-place moisture content of the samples tested are presented on the boring logs in Appendix A

B-1@2-4

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	627.3	650.8
Wt. of Mold	(gm)	202.2	202.2
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	473.2	650.8
Dry Wt. of Soil + Cont.	(gm)	451.5	394.3
Wt. of Container	(gm)	173.2	202.2
Moisture Content	(%)	7.8	13.8
Wet Density	(pcf)	128.2	135.1
Dry Density	(pcf)	119.0	118.8
Void Ratio		0.4	0.4
Total Porosity		0.3	0.3
Pore Volume	(cc)	60.9	62.0
Degree of Saturation	(%) [S_{meas}]	50.9	87.5

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
5/21/2024	10:00	1.0	0	0.3943
5/21/2024	10:10	1.0	10	0.3944
Add Distilled Water to the Specimen				
5/22/2024	10:00	1.0	1430	0.3994
5/22/2024	11:00	1.0	1490	0.3994

Expansion Index (EI meas) =	5
Expansion Index (Report) =	5

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2022 California Building Code, Section 1803.5.3

** Reference: 1997 Uniform Building Code, Table 18-I-B.

 GEOCON	EXPANSION INDEX TEST RESULTS ASTM D-4829	Project No.:	T2875-22-02
		PROPOSED INLAND EMPIRE TECHNICAL TRADE CENTER - ADESSA SITE 6464 33RD STREET, JURUPA VALLEY, CALIFORNIA	
	Checked by: ADL	June 2024	Figure B-1

SUMMARY OF LABORATORY
POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
AASHTO T289 ASTM D4972 and AASHTO T288 ASTM G187

Sample No.	pH	Resistivity (ohm centimeters)
B-3@1-4	8.3	3100

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

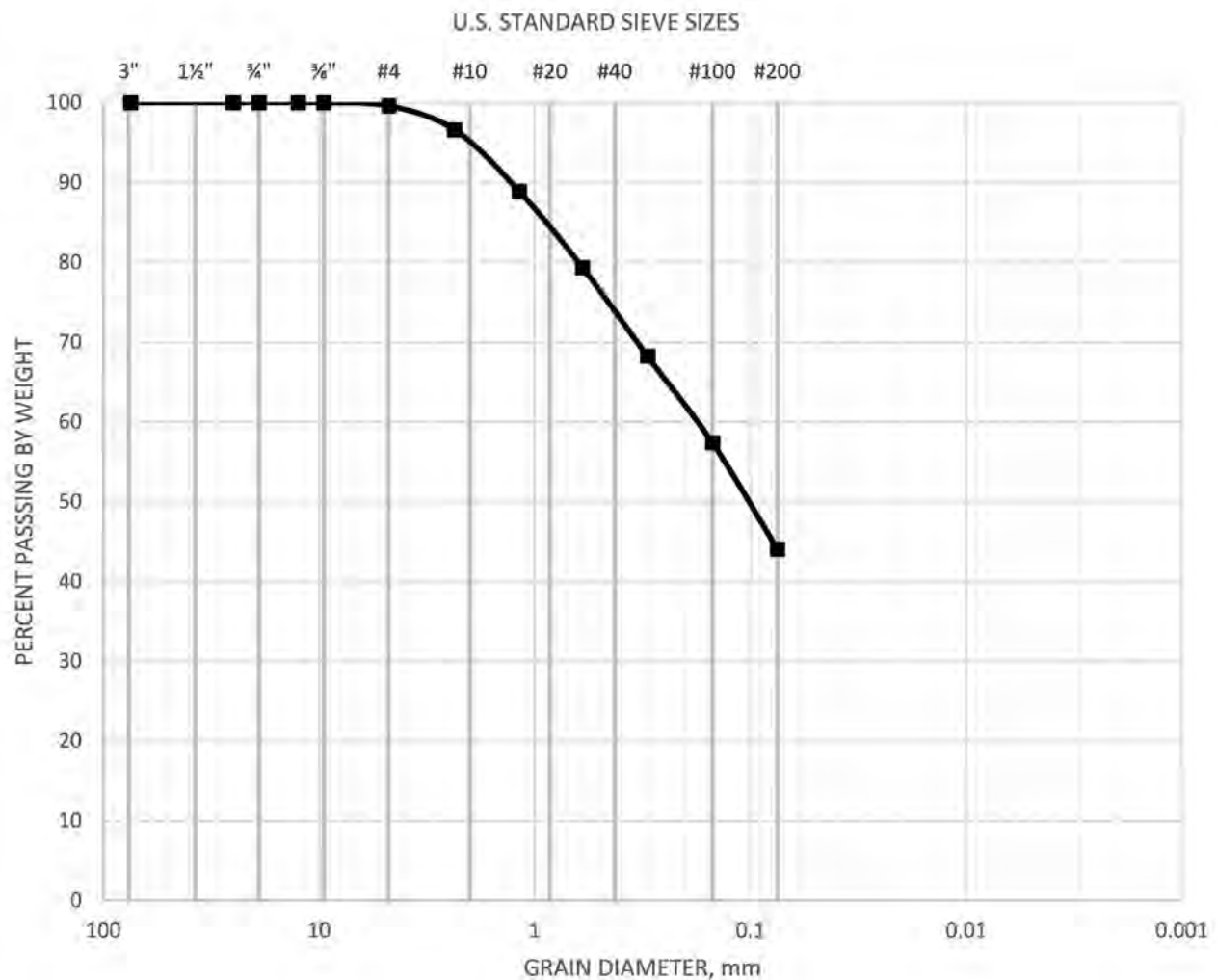
Sample No.	Chloride Ion Content (%)
B-3@1-4	0.063

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
AASHTO T290 ASTM C1580

Sample No.	Water Soluble Sulfate (% SO ₄)	Sulfate Exposure
B-3@1-4	0.000	S0

 GEOCON	CORROSIVITY TEST RESULTS		Project No.:	T2875-22-02
			PROPOSED INLAND EMPIRE TECHNICAL TRADE CENTER - ADESSA SITE 6464 33RD STREET, JURUPA VALLEY, CALIFORNIA	
	Checked by:	ADL	June 2024	Figure B-2

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
B-1@5	Silty SAND (SM), orangish brown	0.18	0.075	0.075



GEOCON

GRAIN SIZE DISTRIBUTION

CT 202

Checked by: ADL

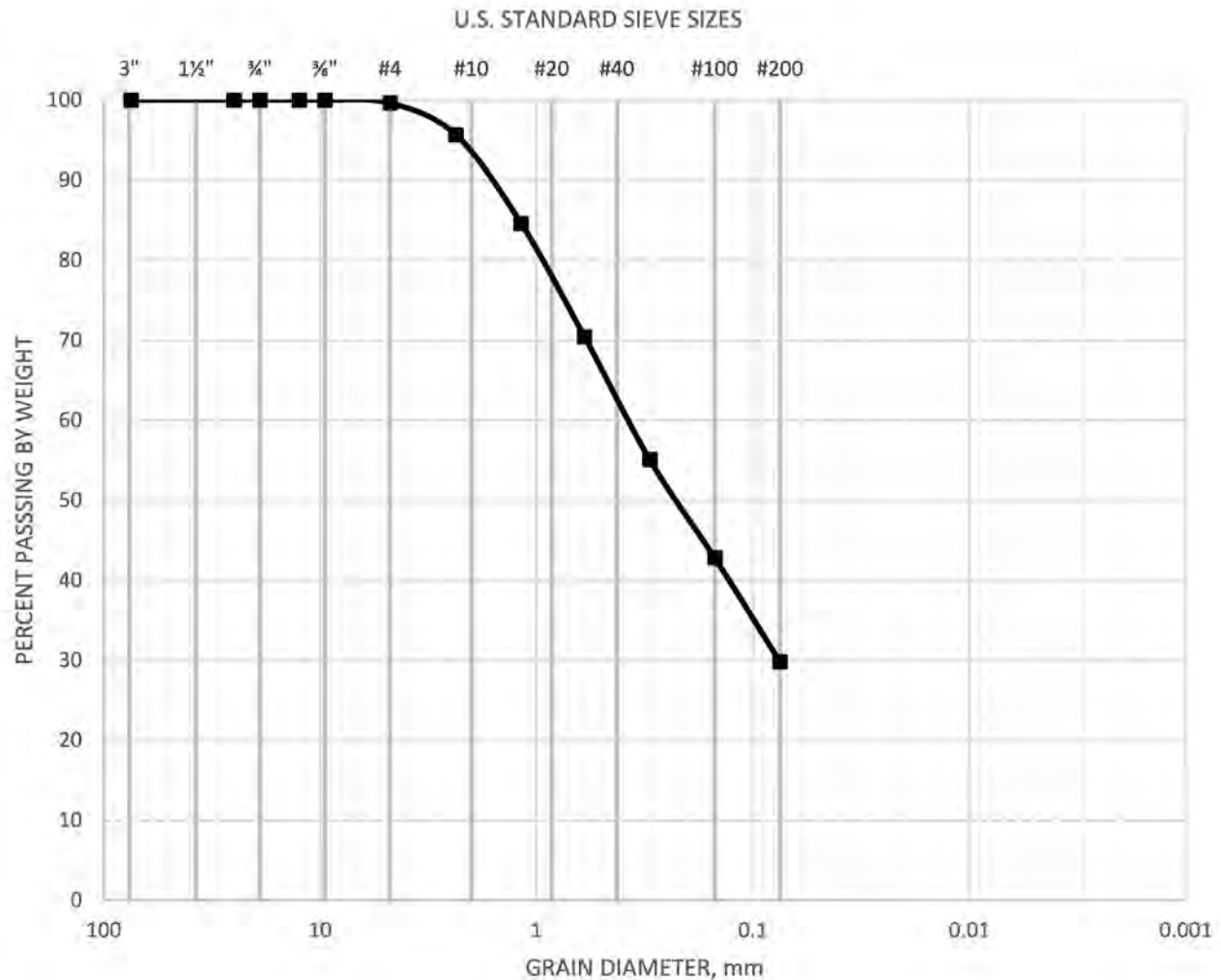
Project No.: T2875-22-02

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER - ADESSA SITE
6464 33RD STREET, JURUPA VALLEY, CALIFORNIA

June 2024

Figure B-3

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
B-1@15	Silty SAND (SM), strong brown	0.39	0.075	0.075



GEOCON

GRAIN SIZE DISTRIBUTION

CT 202

Checked by: ADL

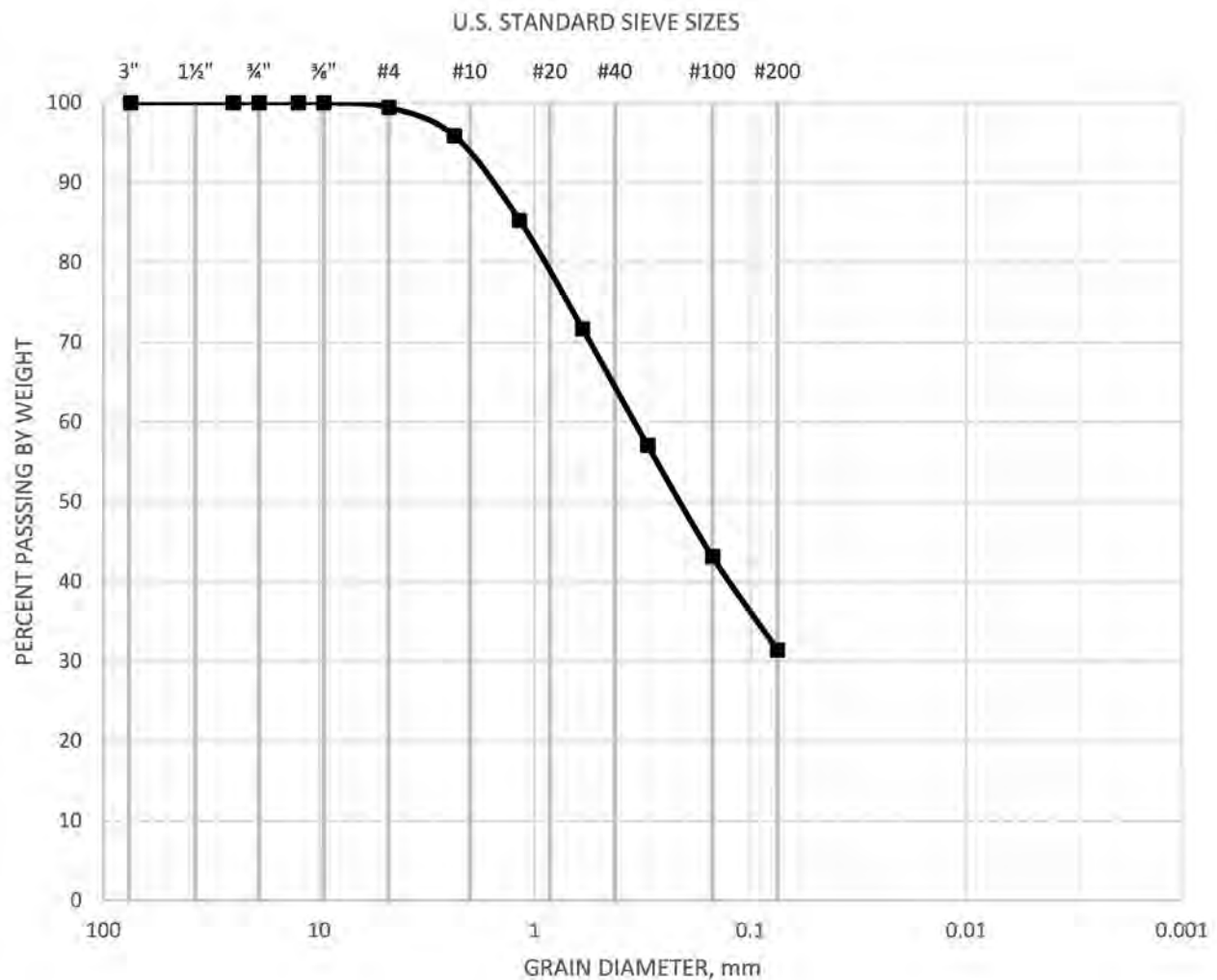
Project No.: T2875-22-02

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER - ADESSA SITE
6464 33RD STREET, JURUPA VALLEY, CALIFORNIA

June 2024

Figure B-4

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
B-1@40	Silty SAND (SM), strong brown	0.35	0.075	0.075



GEOCON

GRAIN SIZE DISTRIBUTION

CT 202

Checked by: ADL

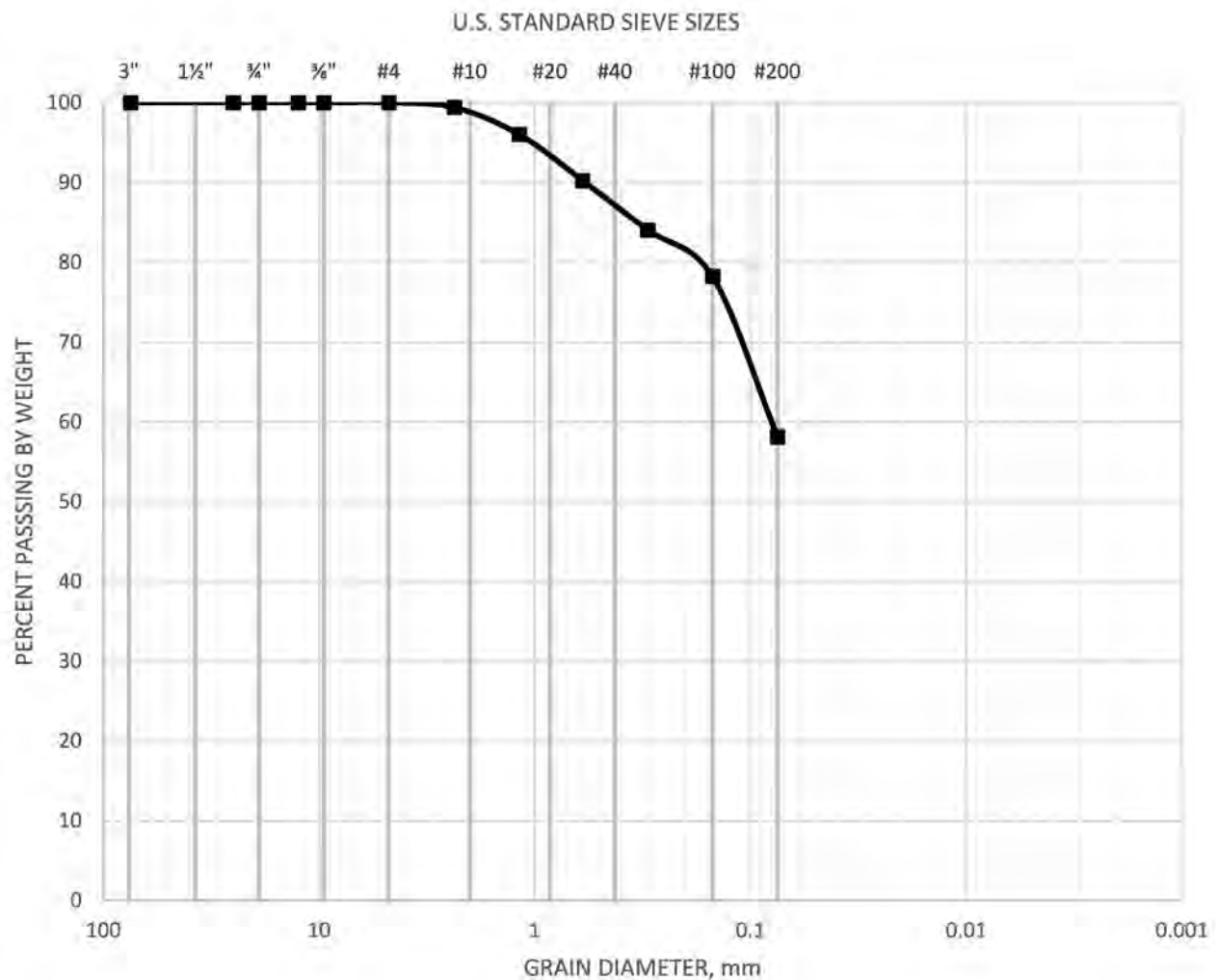
Project No.: T2875-22-02

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER - ADESSA SITE
6464 33RD STREET, JURUPA VALLEY, CALIFORNIA

June 2024

Figure B-5

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
B-3@15	Sandy SILT (ML), strong brown	0.08	0.075	0.075



GRAIN SIZE DISTRIBUTION

CT 202

Checked by: ADL

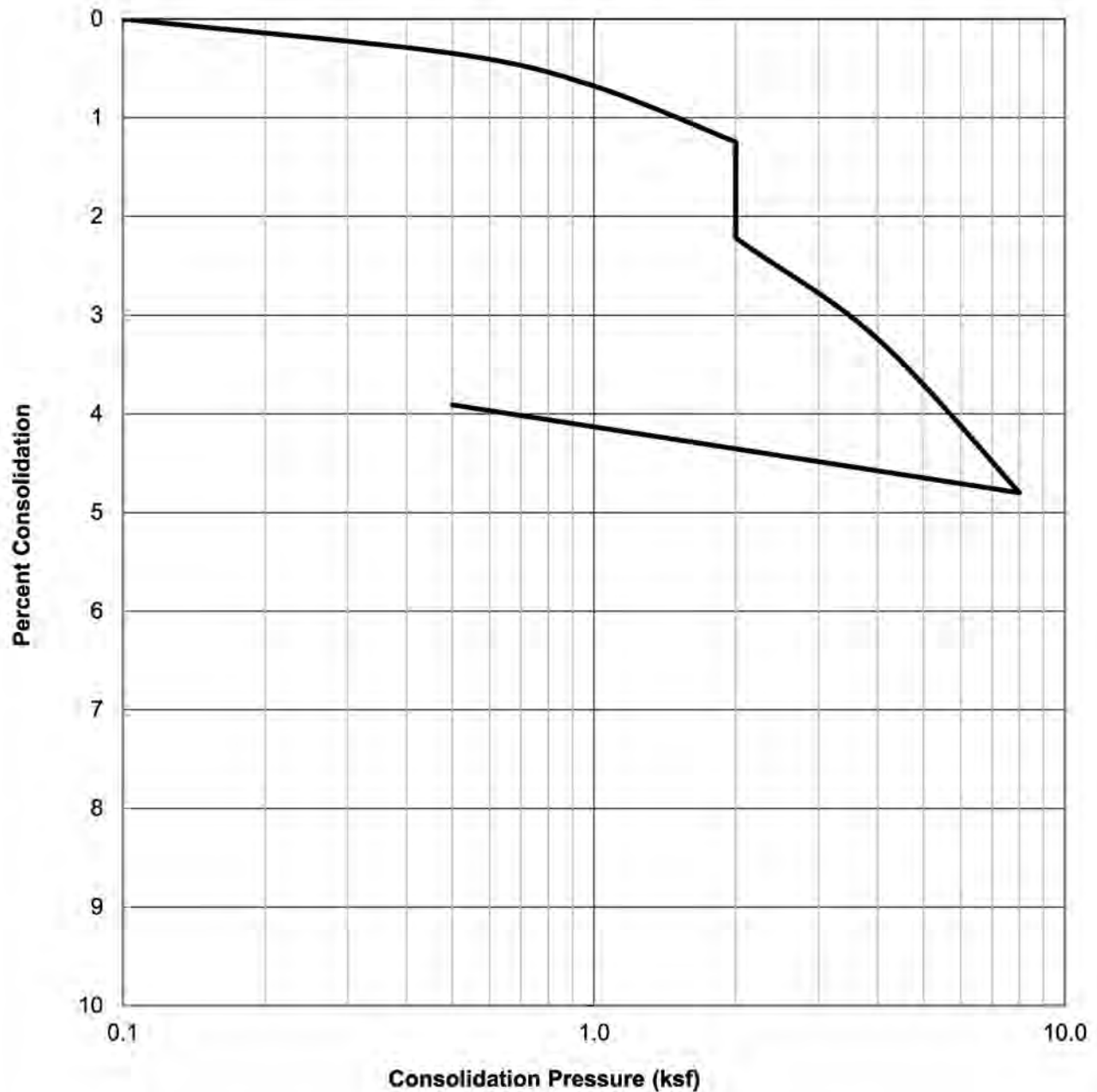
Project No.: T2875-22-02

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER - ADESSA SITE
6464 33RD STREET, JURUPA VALLEY, CALIFORNIA

June 2024

Figure B-6

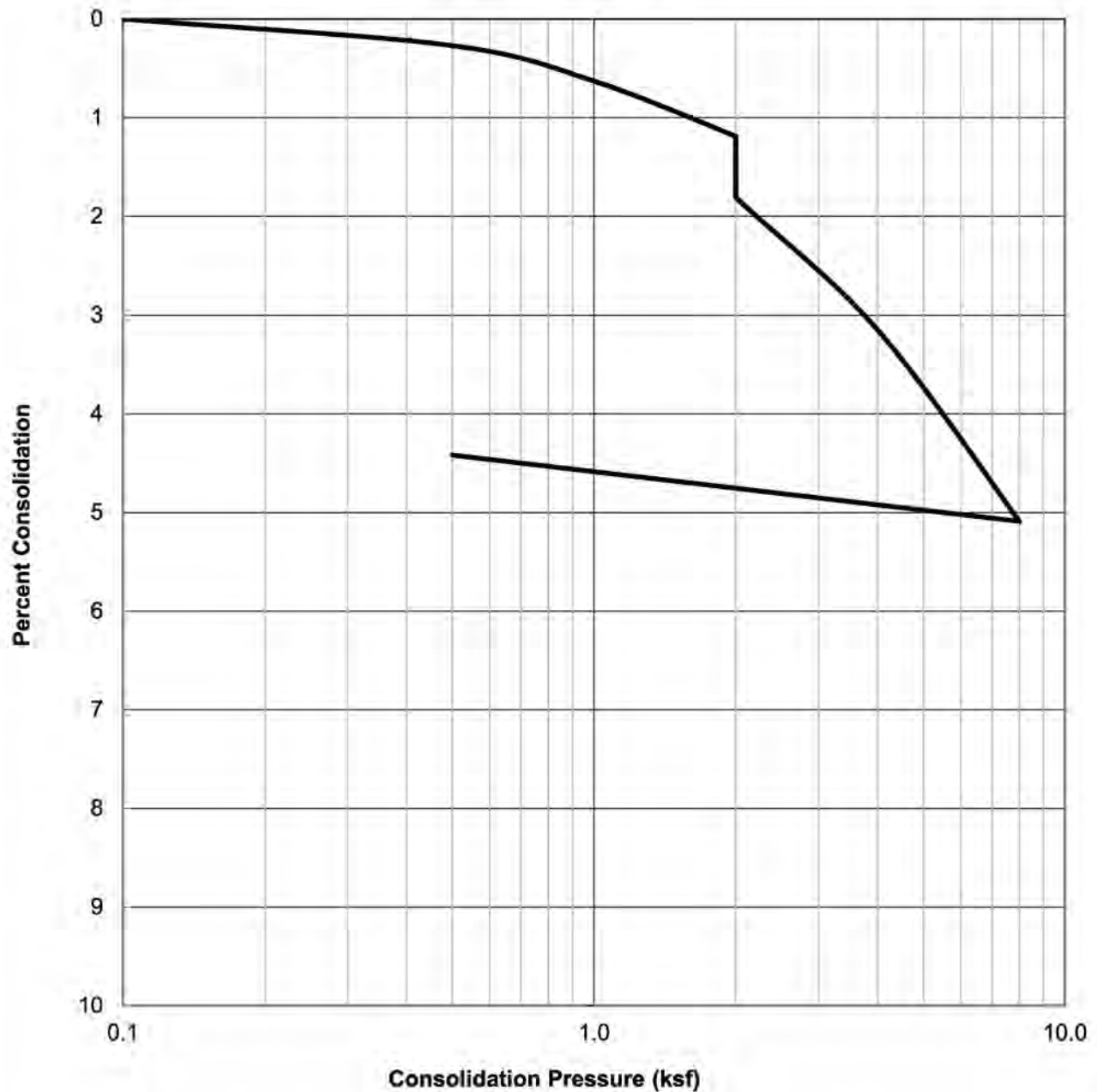
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-2@15	Silty SAND (SM), strong brown	105.1	8.4	20.2

 GEOCON	CONSOLIDATION TEST RESULTS		Project No.: T2875-22-02
	ASTM D-2435		PROPOSED INLAND EMPIRE TECHNICAL TRADE CENTER - ADESSA SITE 6464 33RD STREET, JURUPA VALLEY, CALIFORNIA
	Checked by: ADL		June 2024 Figure B-7

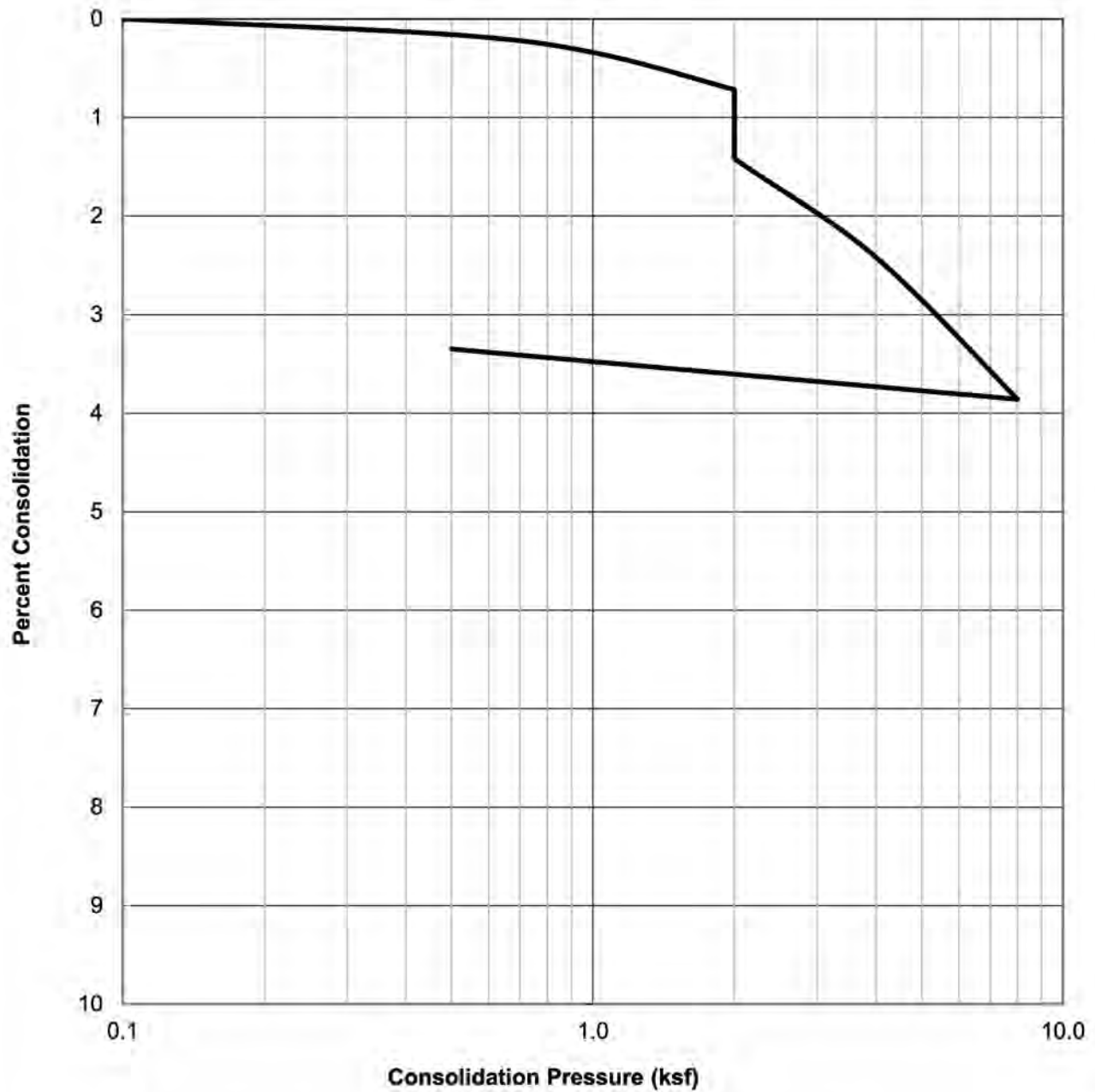
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-3@5	Silty SAND (SM), reddish brown	117.8	10.0	13.7

 GEOCON	CONSOLIDATION TEST RESULTS		Project No.: T2875-22-02
	ASTM D-2435		PROPOSED INLAND EMPIRE TECHNICAL TRADE CENTER - ADESSA SITE 6464 33RD STREET, JURUPA VALLEY, CALIFORNIA
	Checked by: ADL	June 2024	Figure B-8

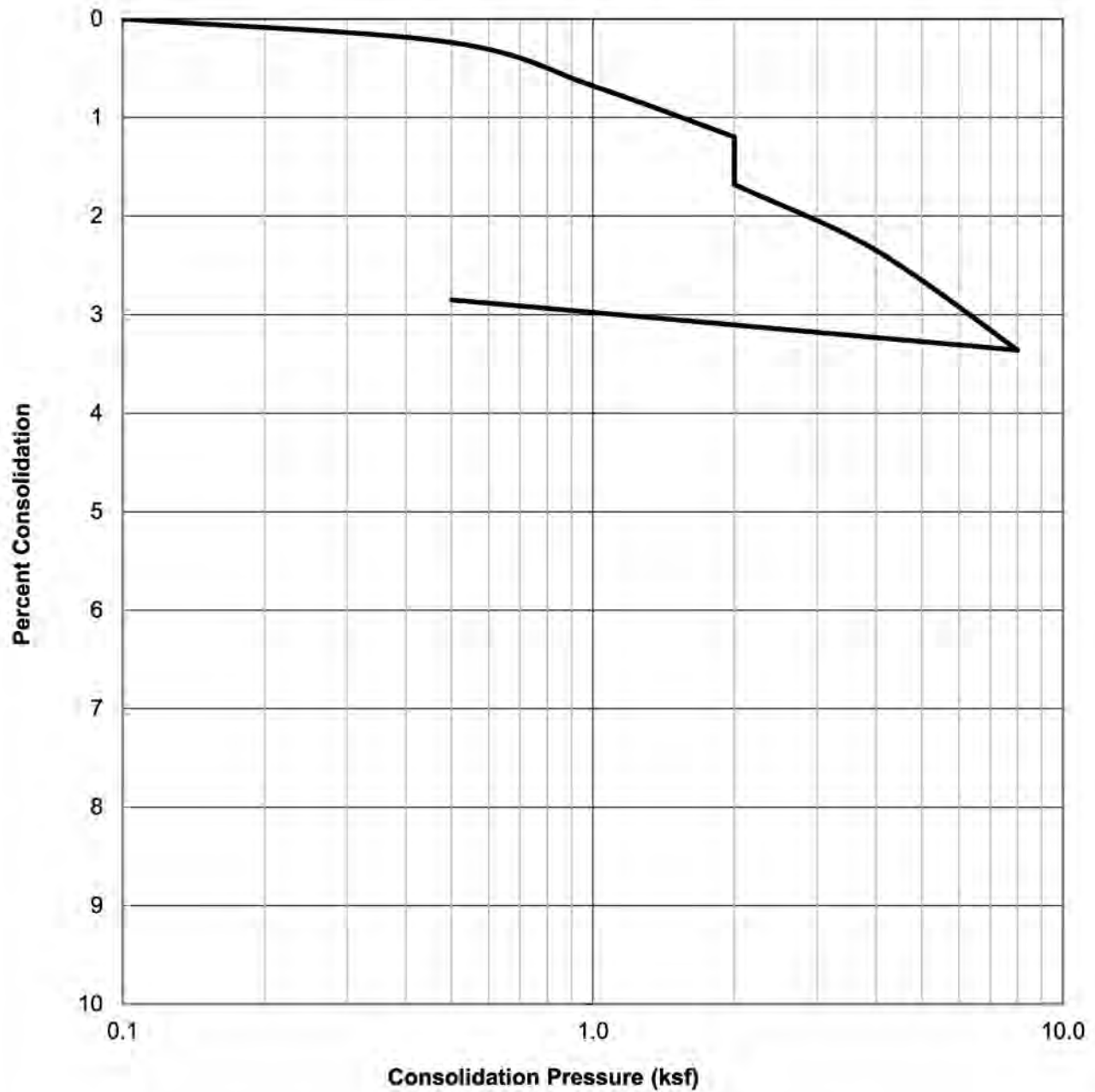
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-3@7.5	Silty SAND (SM), reddish brown	109.3	4.6	14.8

 GEOCON	CONSOLIDATION TEST RESULTS		Project No.: T2875-22-02	
			PROPOSED INLAND EMPIRE TECHNICAL TRADE CENTER - ADESSA SITE 6464 33RD STREET, JURUPA VALLEY, CALIFORNIA	
	ASTM D-2435		June 2024	
	Checked by: ADL	Figure B-9		

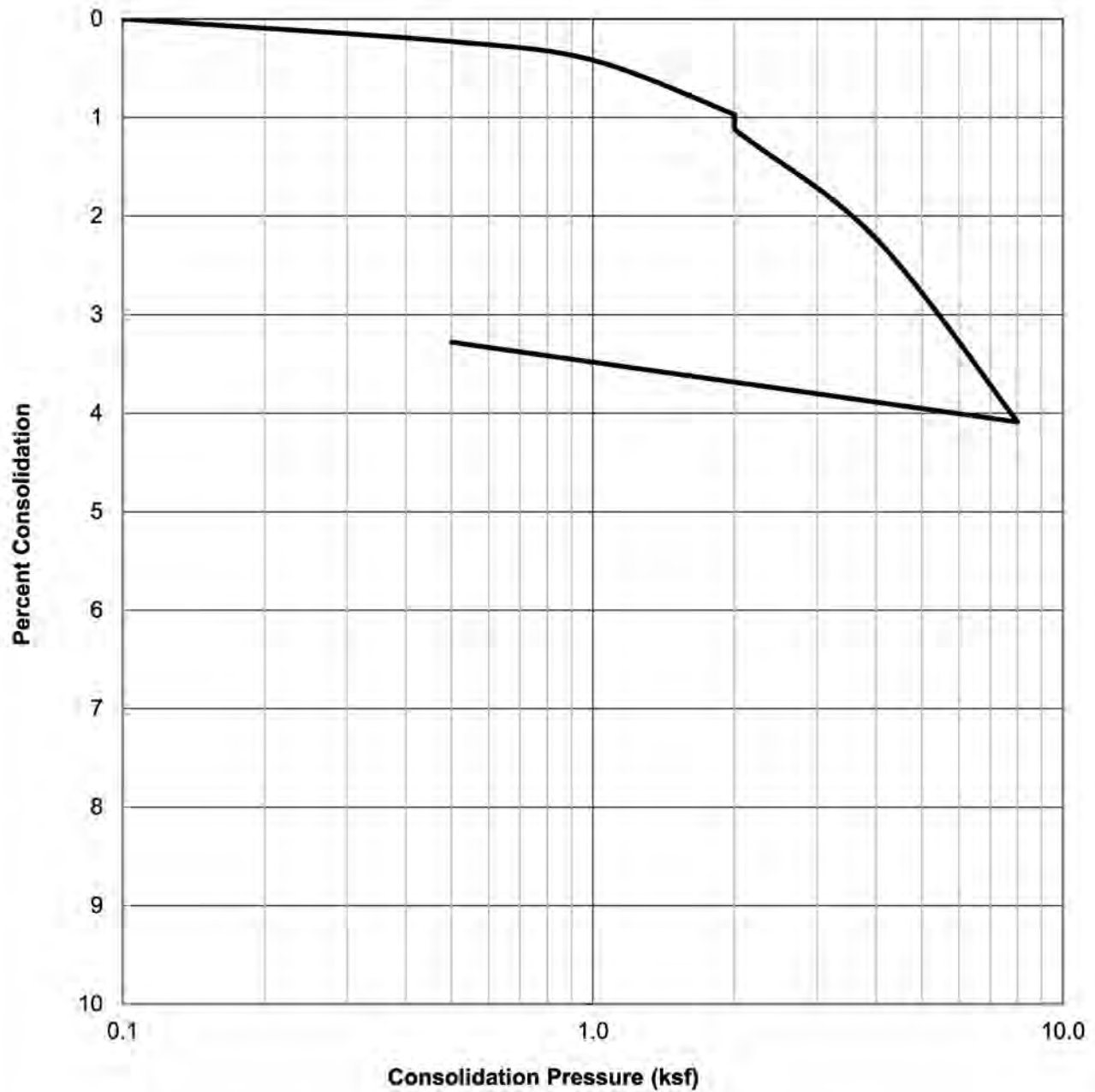
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-3@10	Silty SAND (SM), orangish brown	111.7	3.2	15.6

 GEOCON	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: T2875-22-02	
		PROPOSED INLAND EMPIRE TECHNICAL TRADE CENTER - ADESSA SITE 6464 33RD STREET, JURUPA VALLEY, CALIFORNIA	
	Checked by: ADL	June 2024	Figure B-10

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-4@2.5	Silty SAND (SM), reddish brown	117.9	10.4	13.6



CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: ADL

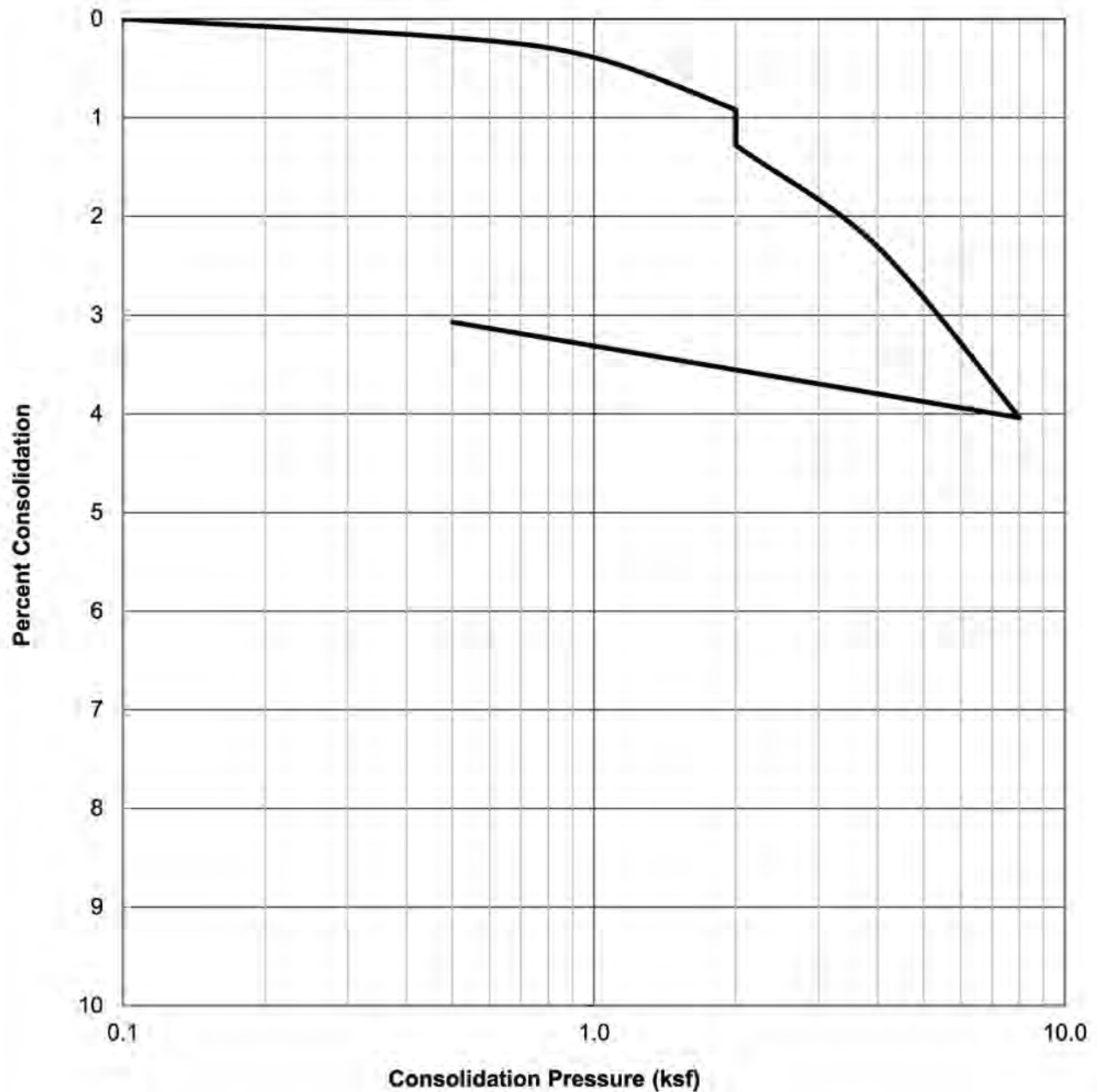
Project No.: T2875-22-02

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER - ADESSA SITE
6464 33RD STREET, JURUPA VALLEY, CALIFORNIA

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Figure B-11

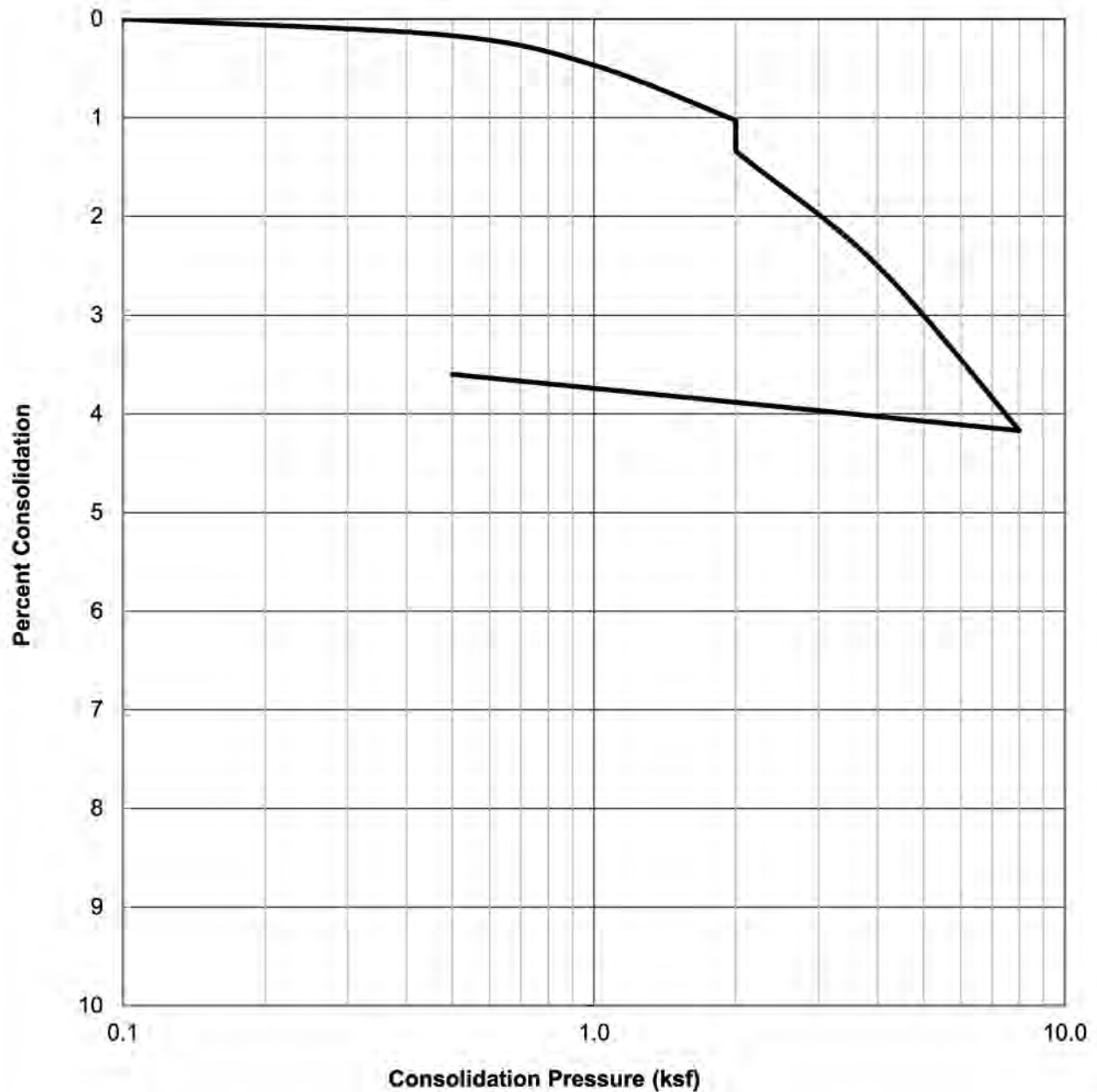
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-4@5	Silty SAND (SM), reddish brown	117.0	11.0	14.8

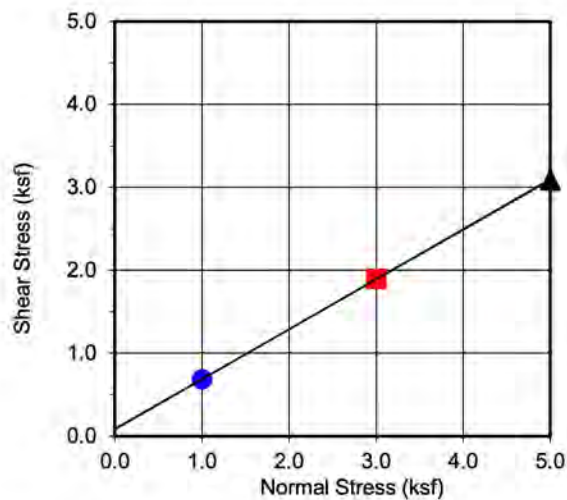
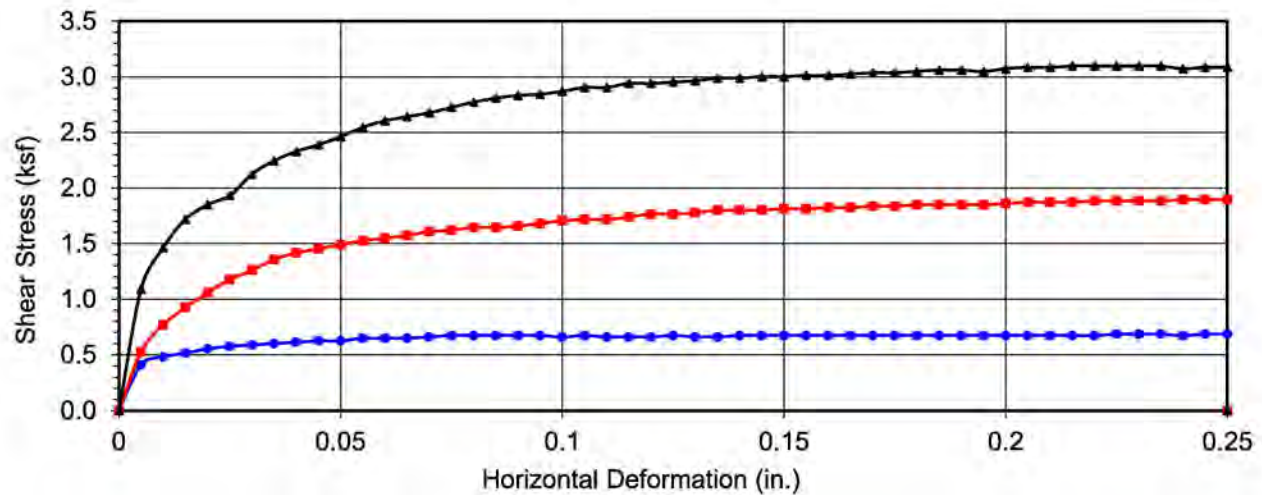
 GEOCON	CONSOLIDATION TEST RESULTS		Project No.: T2875-22-02
	ASTM D-2435		PROPOSED INLAND EMPIRE TECHNICAL TRADE CENTER - ADESSA SITE 6464 33RD STREET, JURUPA VALLEY, CALIFORNIA
	Checked by: ADL		June 2024 Figure B-12

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-6@2.5	Silty SAND (SM), reddish brown	115.0	8.6	16.2

 GEOCON	CONSOLIDATION TEST RESULTS		Project No.: T2875-22-02
	ASTM D-2435		PROPOSED INLAND EMPIRE TECHNICAL TRADE CENTER - ADESSA SITE 6464 33RD STREET, JURUPA VALLEY, CALIFORNIA
	Checked by: ADL	June 2024	Figure B-13



Boring No.	B-2
Sample No.	B-2@1-5
Depth (ft)	1-5
Sample Type:	Bulk

Soil Identification:		
Silty SAND (SM), reddish brown		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	83	31
Ultimate	88	31

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.68	■ 1.90	▲ 3.10
Shear Stress @ End of Test (ksf)	○ 0.68	□ 1.90	△ 3.08
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	7.9	8.1	8.1
Initial Dry Density (pcf)	119.0	119.0	119.0
Initial Degree of Saturation (%)	51.3	52.6	52.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	12.7	11.6	10.2



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

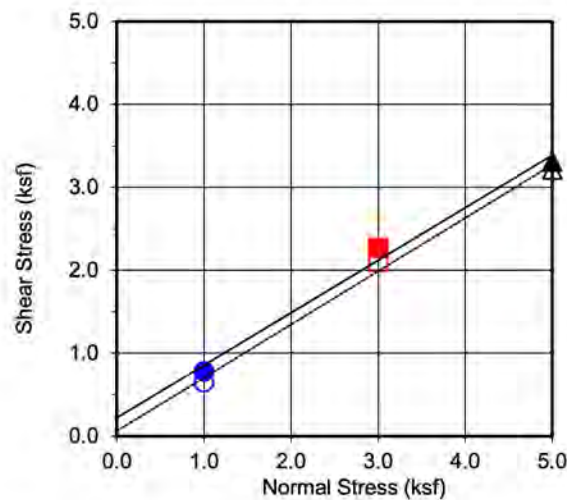
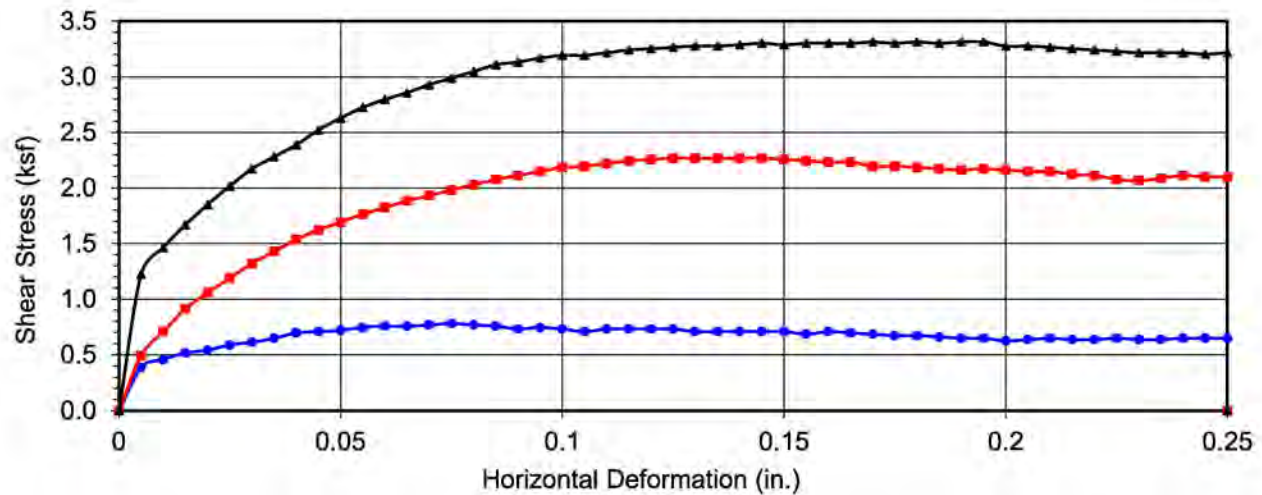
Checked by: ADL

Project No.: T2875-22-02

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER - ADESSA SITE
6464 33RD STREET, JURUPA VALLEY, CALIFORNIA

June 2024

Figure B-14



Boring No.	B-3
Sample No.	B-3@2.5
Depth (ft)	2.5
Sample Type:	Ring

Soil Identification:		
Silty SAND (SM), reddish brown		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	221	32
Ultimate	62	33

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.78	■ 2.27	▲ 3.31
Shear Stress @ End of Test (ksf)	○ 0.65	□ 2.10	△ 3.22
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.1	9.7	11.0
Initial Dry Density (pcf)	111.4	115.7	114.9
Initial Degree of Saturation (%)	53.4	57.0	63.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	15.3	13.5	14.1



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: ADL

Project No.: T2875-22-02

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER - ADESSA SITE
6464 33RD STREET, JURUPA VALLEY, CALIFORNIA

June 2024

Figure B-15

Sample No:

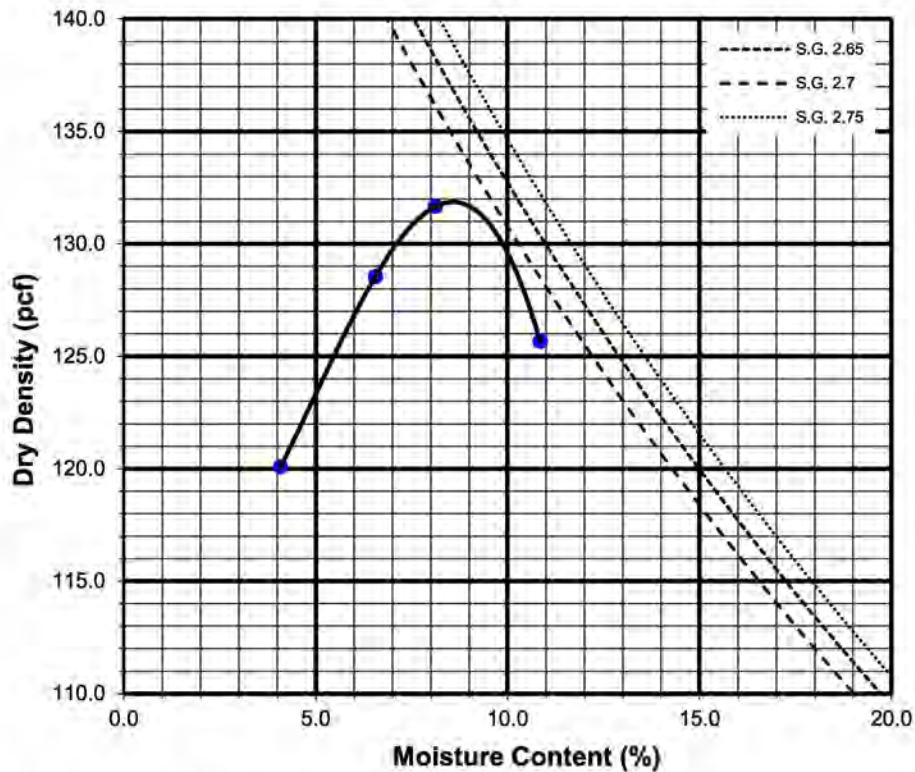
B-2@1-5

Silty SAND (SM), reddish brown

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6405	6359	6324	6143		
Weight of Mold	(g)	4255	4255	4255	4255		
Net Weight of Soil	(g)	2150	2104	2069	1888		
Wet Weight of Soil + Cont.	(g)	764.1	679.2	753.5	694.0		
Dry Weight of Soil + Cont.	(g)	726.0	637.9	723.0	676.9		
Weight of Container	(g)	256.8	257.1	257.6	258.2		
Moisture Content	(%)	8.1	10.8	6.6	4.1		
Wet Density	(pcf)	142.4	139.3	137.0	125.0		
Dry Density	(pcf)	131.7	125.7	128.6	120.1		

Maximum Dry Density (pcf) 132.0

Optimum Moisture Content (%) 8.0



Preparation Method: A



**COMPACTION CHARACTERISTICS USING
MODIFIED EFFORT TEST RESULTS**

ASTM D-1557

Checked by: ADL

Project No.: T2875-22-02

PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER - ADESSA SITE
6464 33RD STREET, JURUPA VALLEY, CALIFORNIA

June 2024

Figure B-16

APPENDIX

C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

**PROPOSED INLAND EMPIRE TECHNICAL
TRADE CENTER – ADESA SITE
6363 34TH STREET
JURUPA VALLEY, CALIFORNIA**

PROJECT NO. T2875-22-02

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 $\frac{3}{4}$ 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 $\frac{3}{4}$ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.
- 3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.
- 3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9

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

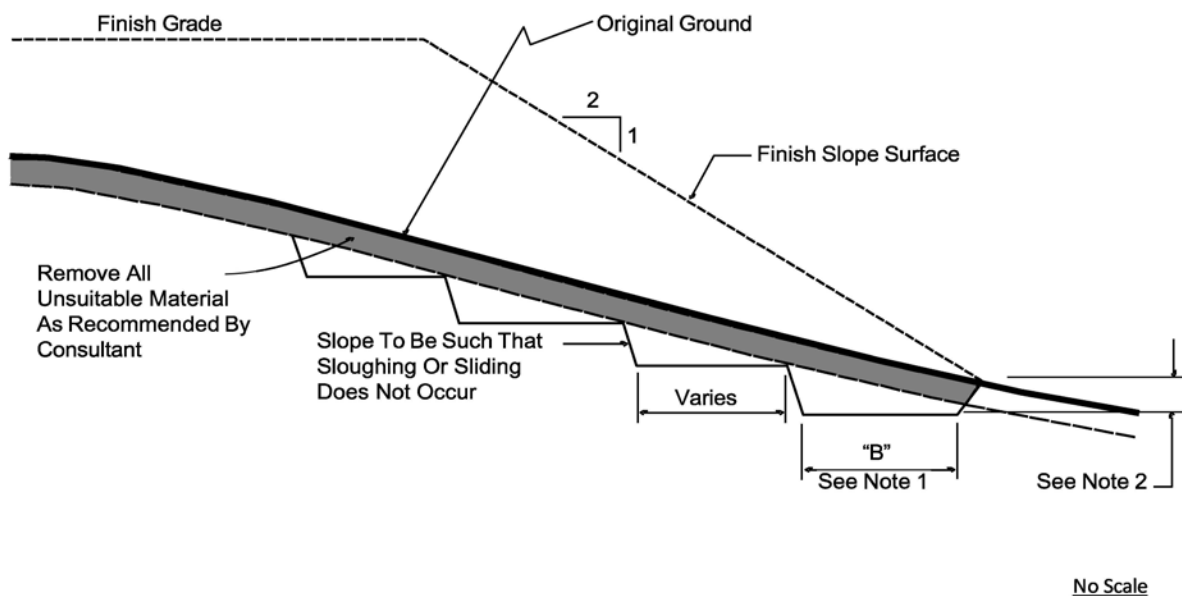
- 3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.
- 3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.
- 3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

4. CLEARING AND PREPARING AREAS TO BE FILLED

- 4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.
- 4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

- 4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.
- 4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



- DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.
- (2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

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

5. COMPACTION EQUIPMENT

- 5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.
- 5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

- 6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:
 - 6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.
 - 6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557.
 - 6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.
 - 6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.
 - 6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

- 6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.
 - 6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.
 - 6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.
- 6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:
- 6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.
 - 6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.
 - 6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.
 - 6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

- 6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.
- 6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.
- 6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:
- 6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.
- 6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.
- 6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection

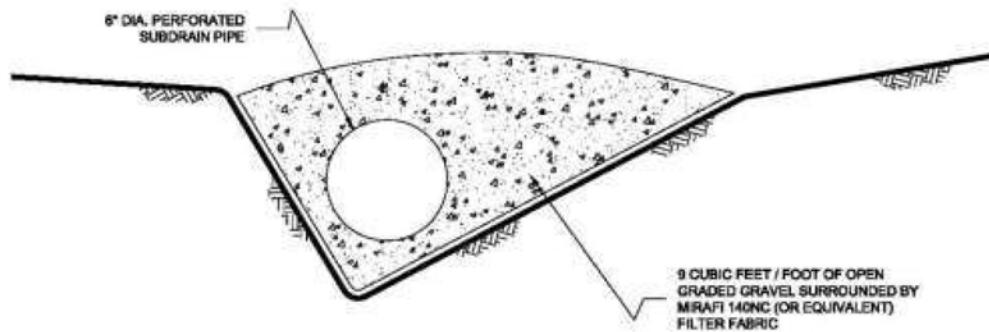
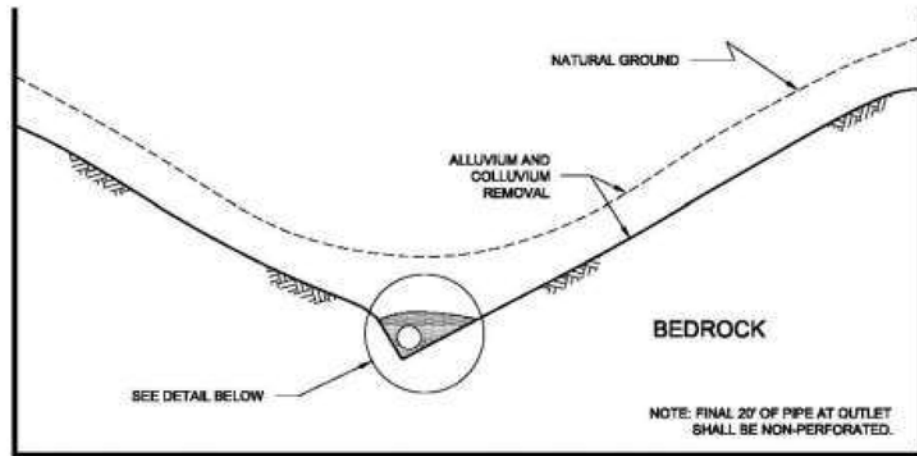
variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

- 6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of “passes” have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.
- 6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.
- 6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.
- 6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL



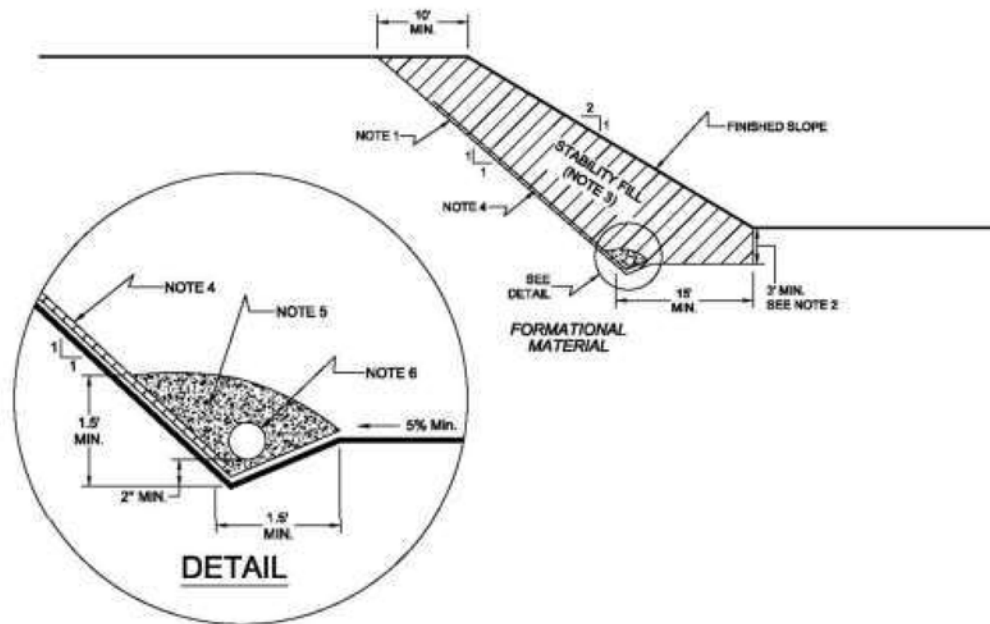
NOTES:

- 1.....8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.
- 2.....6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

- 1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).
- 2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.
- 3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.
- 4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.
- 5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).
- 6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

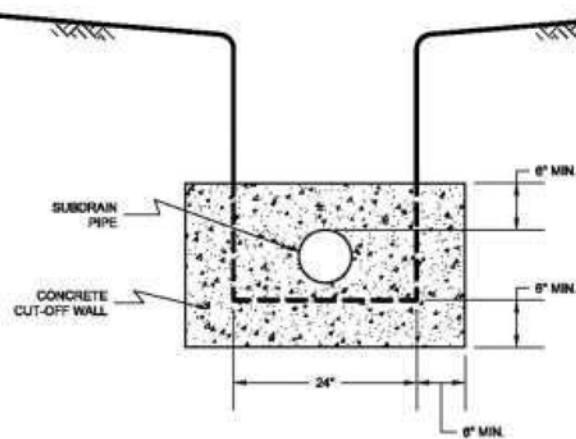
NO SCALE

- 7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.
- 7.4 *Rock fill or soil-rock fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. Rock fill drains should be constructed using the same requirements as canyon subdrains.*

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

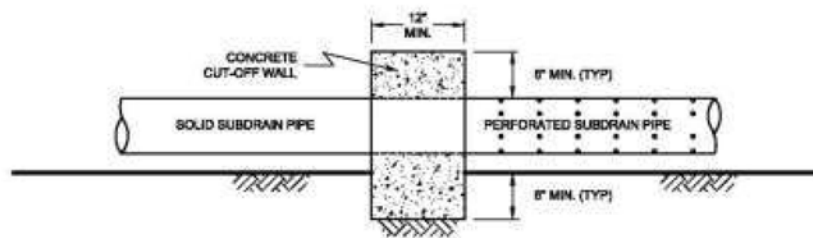
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW

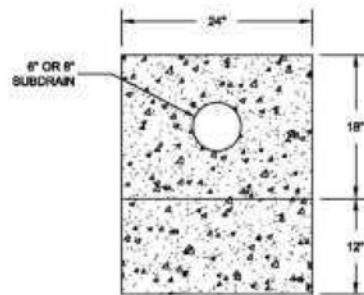


NO SCALE

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

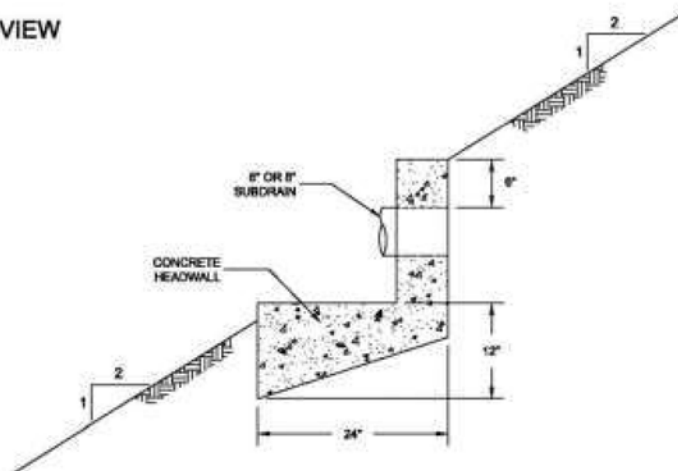
TYPICAL HEADWALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW



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

NO SCALE

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

8. OBSERVATION AND TESTING

- 8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.
- 8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.
- 8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.
- 8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.
- 8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.
- 8.6 Testing procedures shall conform to the following Standards as appropriate:

8.6.1 Soil and Soil-Rock Fills:

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

- 8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth)*.
- 8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.
- 8.6.1.4 Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

- 9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.
- 9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

10. CERTIFICATIONS AND FINAL REPORTS

- 10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.
- 10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.