



PRELIMINARY GEOTECHNICAL INVESTIGATION

LIBRARY LEARNING RESOURCE CENTER
MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

OCTOBER 22, 2025
PROJECT NO. T2875-22-05

PREPARED FOR:
Riverside Community College District
Riverside, California



Project No. T2875-22-05
October 22, 2025

Riverside Community College District
Attn: Carey Demas, Design Manager
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Riverside, CA 92501

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
LIBRARY LEARNING RESOURCE CENTER (LLRC)
MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

Mr. Demas:

In accordance with your authorization of our proposal dated August 8, 2025, we have performed a preliminary geotechnical investigation for the proposed Library Learning Resource Center (LLRC), located at the Moreno Valley College campus, in the City of Moreno Valley, California. The accompanying report presents the findings of our study and our conclusions and recommendations pertaining to the geotechnical aspects of the proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a preliminary geotechnical investigation for the proposed Library Learning Resource Center (LLRC), located at the Moreno Valley College campus, in the City of Moreno Valley, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of the proposed design and construction.

The scope of this investigation included site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on September 15, 2025, by excavating eight 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine to depths ranging between 11.5 and 55 feet beneath the ground surface. The approximate locations of the exploratory borings are depicted on the Geologic Map & Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at Moreno Valley College in Parking Lot B, in Moreno Valley, California. The site is currently comprised of the main parking lot on campus, which has staff and student parking as well as shaded parking stalls on the west end of the parking lot. The site is bounded by College Drive and an access road adjacent to the Welcome Center and Student Activities Center to the north, by Krameria Street adjacent to the Police and Parking Services complex to the east, by the Early Childhood and Education Center complex to the south, and by West Campus Road and the Dental Education Center complex to the west. The site slopes gently to the northwest, and surface water drainage at the site appears to flow to the city streets. At roughly the center of the site, the latitude and longitude are 33.886126, -117.203037.

The existing parking lot includes five rows of staff parking stalls and six handicapped stalls along the east side of the site, with the remaining stalls designated for student parking. The northwest portion of the lot contains three rows of shaded parking spaces and several electric vehicle charging stalls. Minor alligator cracking was observed in the asphalt pavement throughout the site.

Site elevations range from approximately 1,572 feet at the southeast corner to about 1,544 feet at the northwest corner, with the proposed building pad at roughly 1,564 feet. Because school was in session and Parking Lot B served as the main student parking area, access was limited. As a result, boring B-6 was drilled just outside the proposed building footprint, but is considered representative of site conditions.

Based on the limited information provided by the Client, it is our understanding that the proposed development will consist of the Library Learning Resource Center (LLRC) on the east side of the current parking lot, and the remainder of the site will be the new parking lot area. The proposed LLRC building will consist of a 79,290 square-foot, three-story structure constructed at or near present grade. Plans depicting the proposed improvements are provided on the Geologic Map & Site Plan (see Figure 2).

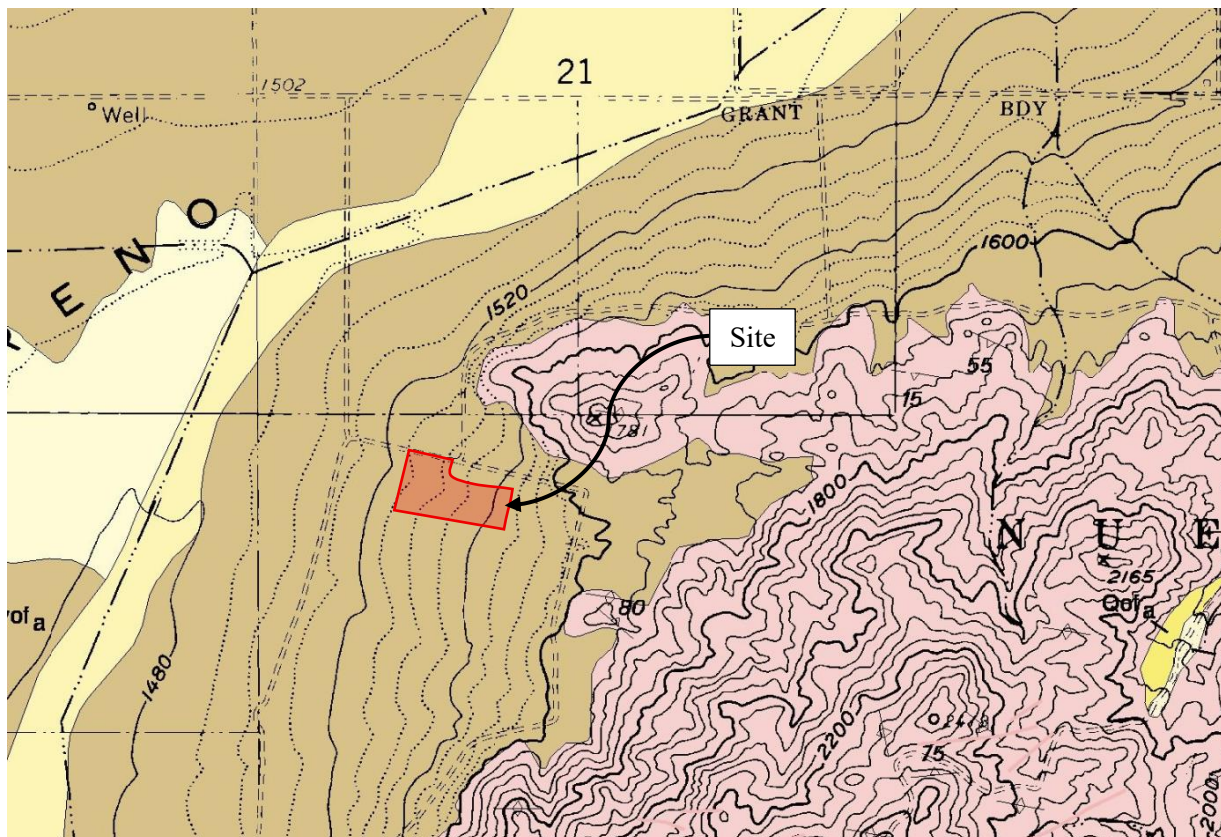
Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 300 kips, and wall loads will be up to 3 kips per linear foot.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, 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.

3. GEOLOGIC SETTING

The site is located within the Perris Block of the Peninsular Ranges Geomorphic Province. The Perris Block is characterized by granitic highlands, which display three elevated erosional surfaces surrounded by alluvial valleys. The Peninsular Ranges are bound by the Transverse Ranges (San Gabriel and San Bernardino Mountains) to the north and the Colorado Desert Geomorphic Province to the east. The Peninsular Ranges extend westward into the Pacific Ocean and southward to the tip of Baja California. Overall, the province is characterized by Cretaceous-age granitic rock and a lesser amount of Mesozoic-age metamorphic rock overlain by terrestrial and marine sediments. Locally, the site is located on the southeastern margin of the Moreno Valley, north of Lake Perris. Granitic hills that surround Lake Perris are present approximately 1,000 feet southeast of the site. The entire site is underlain by older alluvium above granitic basement rock.

REGIONAL GEOLOGIC MAP



Source: Morton, D.M., Matti, J.C., Diep, V.M., and Edwards-Howells, Ursula, 2002, Geologic map of the Sunnymead 7.5' quadrangle, Riverside County, California, U.S. Geological Survey, Open-File Report OF-2001-450, 1:24,000.

4. SOIL AND GEOLOGIC CONDITIONS

Based on published geologic maps of the area, geologic units at the site consist of Older Alluvium above Quartz Diorite (granitic bedrock) (Morton, et. Al., 2002). Asphalt covers the site to depths of 5 to 5.5 inches. Six inches of aggregate base was encountered in one boring (B-4). Artificial fill was present beneath the paving materials to depths of 0 to 5 feet. Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill (af)

Artificial fill was encountered in most borings at depths of approximately 3 to 4 feet below the ground surface, extending to a maximum depth of 5 feet in B-6. No fill was observed in B-8. The fill materials generally consist of light brown to dark brown silty sand and poorly graded sand, ranging from dry to moist and medium dense to dense in consistency. It is possible that deeper fill exists between the explored locations or in other portions of the site that were not directly investigated.

4.2 Older Alluvium (Qoa)

Pleistocene-age older alluvium was encountered in all borings, extending to depths between approximately 20 and 36 feet. These deposits generally consist of light brown to dark brown silty sand with lesser amounts of poorly graded sand containing silt. The alluvial soils are dry to moist and range in density from loose to very dense. Loose zones were observed within the alluvium between depths of about 5 and 11 feet below the existing ground surface in Borings B-6, B-7, and B-8, located in the parking lot areas west of the proposed building. Rootlets were also noted in the sample obtained at a depth of 5 feet in B-6.

4.3 Quartz Diorite Bedrock (no map symbol)

Cretaceous-age quartz diorite bedrock (granitic bedrock) was encountered at depths of 20 feet (B-4 and B-6) and 36 feet (B-1). The rock is highly to moderately weathered, hard, and increasingly difficult to excavate, with drilling refusal occurring at approximately 55 feet. Soil-filled joints were observed in the upper, highly weathered zone.

5. GROUNDWATER

Review of the California Department of Water Resources Water Well Depth Database indicates that the three wells near the site had groundwater measured at depths of 11 to 60 feet below ground surface from 2012 to present.

Groundwater was not encountered in our borings, drilled to a maximum depth of approximately 55 feet below the existing ground surface. Based on the lack of groundwater in our borings and the depth of proposed construction, static groundwater is neither expected to be encountered during construction nor to have a detrimental effect on the project. 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). Groundwater seepage levels encountered during construction may actually be higher than those encountered during our investigation. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the region. Proper surface drainage of irrigation and precipitation will be critical for the future performance of the project. Recommendations for drainage are provided in the *Surface Drainage* section of this report.

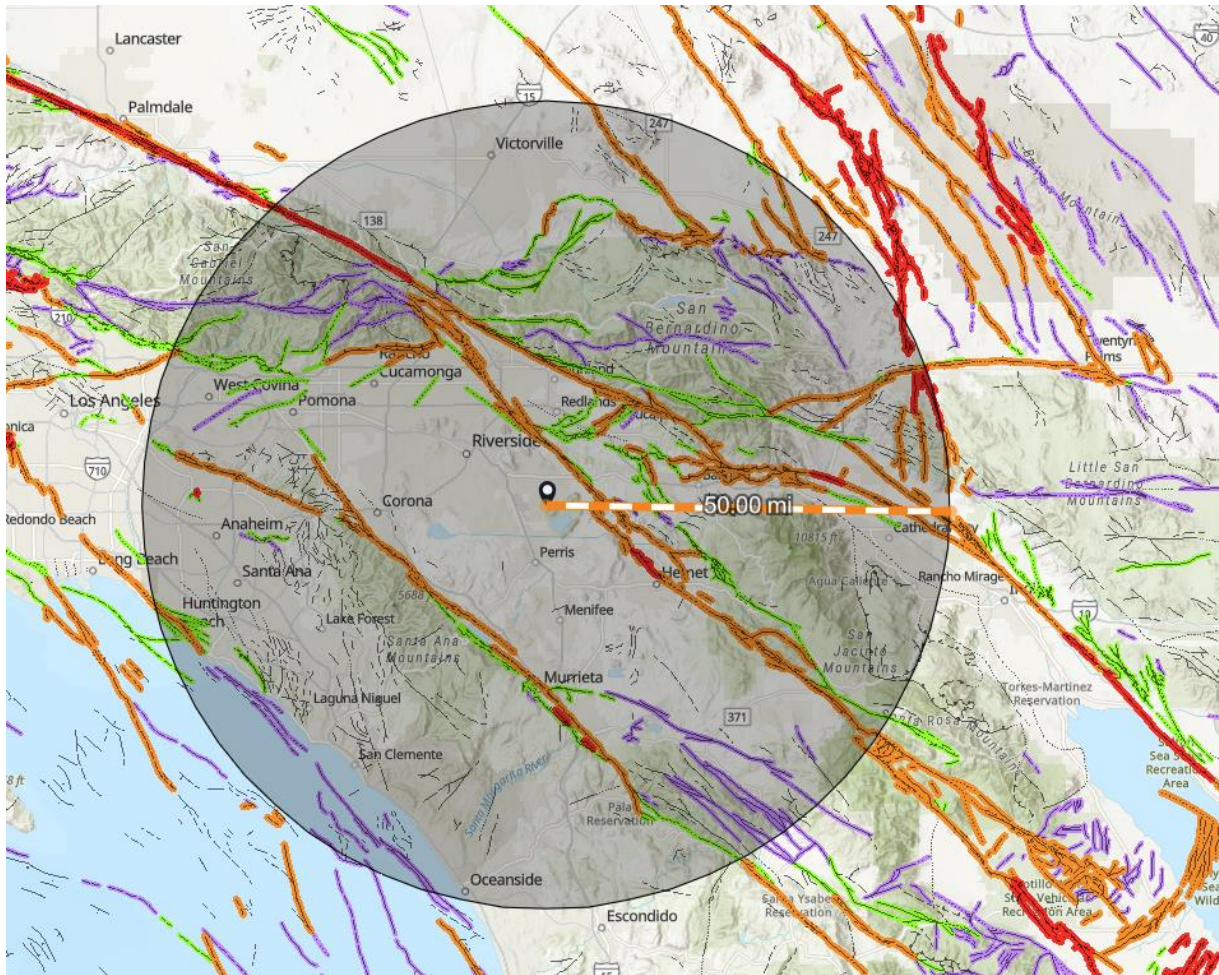
6. GEOLOGIC HAZARDS

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 evidence of 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, 2025a; 2025b; 2017; Riverside County Map My County 2025) 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 on the following Regional Fault Map.

REGIONAL FAULT MAP



Source: Jennings, C. W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.

The closest surface trace of an active fault to the site is the San Jacinto Fault, located approximately 5 miles to the northeast. Other nearby active faults are the Casa Loma Fault, located approximately 5.5 miles east of the site, and the Elsinore fault, located 16 miles southwest. The active San Andreas Fault Zone is located approximately 16 miles east of the site (Bryant, 2010).

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data.

A 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	48	WSW
Tehachapi	July 21, 1952	7.5	129	NW
San Fernando	February 9, 1971	6.6	77	WNW
Whittier Narrows	October 1, 1987	5.9	52	WNW
Sierra Madre	June 28, 1991	5.8	52	WNW
Landers	June 28, 1992	7.3	49	ENE
Big Bear	June 28, 1992	6.4	31	NE
Northridge	January 17, 1994	6.7	79	WNW
Hector Mine	October 16, 1999	7.1	73	NE
Ridgecrest China Lake Fault	July 5, 2019	7.1	132	NNW

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 Seismic Design Criteria

The seismic provisions of the 2025 California Building Code (CBC) are based on the American Society of Civil Engineers (ASCE)/Structural Engineering Institute (SEI) publication: ASCE/SEI 7-22, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-22). For seismic design purposes, sites are classified based on the average shear wave velocity, $\bar{v}_s$, from the ground surface to a depth of 100 feet. The following table presents the Site Classifications.

SITE CLASSIFICATION

Site Class	Average Shear Wave Velocity, $\bar{v}_s$ (Feet/Second)
A – Hard Rock	5,000+
B – Medium Hard Rock	3,000 to 5,000
BC – Soft Rock	2,100 to 3,000
C – Very Dense Sand or Hard Clay	1,450 to 2,100
CD – Dense Sand or Very Stiff Clay	1,000 to 1,450
D – Medium Dense Sand or Stiff Clay	700 to 1,000
DE – Loose Sand or Medium Stiff Clay	500 to 700
E – Very Loose Sand or Soft Clay	Less Than 500
F – Soils Requiring Site Response Analysis	n/a

Where shear wave velocity is not directly measured at the project site, appropriate generalized correlations between shear wave velocity and standard penetration test (SPT) blow counts, Cone Penetration Test (CPT) data, shear strength, or other geotechnical parameters can be used to estimate the shear wave velocity profile.

Seismic design parameters based on estimated values of shear wave velocity shall be derived using the range of $\bar{v}_s$, $\bar{v}_s / 1.3$, and $1.3 \bar{v}_s$ in accordance with ASCE 7-22 Section 20.3. The following table presents a summary of the calculated shear wave velocities and corresponding Site Classes.

ESTIMATED SITE CLASSIFICATION

$\bar{v}_s$ Parameter	Estimated Shear Wave Velocity, $\bar{v}_s$ (Feet/Second)	Site Class
Estimated $\bar{v}_s$	1,334	CD – Dense Sand or Very Stiff Clay
Lower Bound Estimated $\bar{v}_s$ ($\bar{v}_s / 1.3$)	1,027	CD – Dense Sand or Very Stiff Clay
Upper Bound Estimated v_s ($1.3\bar{v}_s$)	1,735	C – Very Dense Sand or Hard Clay

If the estimated shear wave velocity ($\bar{v}_s$), lower bound estimate ($\bar{v}_s / 1.3$), and upper bound estimate ($1.3 \bar{v}_s$) result in different Site Classes (e.g. Site Classes C, CD, and D), the seismic design parameters shall be based on the most critical spectral response acceleration at each period.

We used the online ASCE Hazard Tool (<https://ascehazardtool.org>) to evaluate the code-based seismic design parameters in accordance with ASCE 7-22. The results are summarized in the following table. Appendix C presents the output generated by the ASCE Hazard Tool.

SEISMIC DESIGN PARAMETERS

Parameter		Value			ASCE 7-22 Reference
Site Class		C	CD	Controlling	Table 20.1-1
S_s	MCE _R 5% Damped Spectral Response Acceleration at a Period of 0.2 Seconds for Site Class BC	1.91g	1.91g	1.91g	USGS Seismic Design Geodatabase
S_1	MCE _R 5% Damped Spectral Response Acceleration at a Period of 1 Second for Site Class BC	0.67g	0.67g	0.67g	
S_{MS}	MCE _R 5% Damped Spectral Response Acceleration at Short Periods Adjusted for Site Class	2.01g	2.14g	2.14g	
S_{M1}	MCE _R 5% Damped Spectral Response Acceleration at a Period of 1 Second Adjusted for Site Class	0.96g	1.33g	1.33g	
S_{DS}	Design, 5% Damped Spectral Response Acceleration at Short Periods	1.34g	1.43g	1.43g	Equation 11.4-1
S_{D1}	Design, 5% Damped Spectral Response Acceleration at a Period of 1 Second	0.64g	0.89g	0.89g	Equation 11.4-2
PGA_M	Maximum Considered Earthquake Geometric Mean (MCE _G) Peak Ground Acceleration	0.74g	0.76g	0.76g	USGS Seismic Design Geodatabase

If required for structural design, the multi-period MCE_R response spectra are provided in Appendix C and are based on the ASCE Hazard Tool. The two-period MCE_R response spectra may be constructed based on ASCE 7-22 Section 11.4.5.2 using the values in the table above.

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

6.4 Liquefaction Potential & Dry 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. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

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

The Riverside County Mapping Portal website indicates that the site is located in an area designated as having a low susceptibility for liquefaction, noted with “no groundwater data but sediments susceptible to liquefaction”.

Groundwater was not encountered in our borings to a maximum depth of 55 feet. However, a review of available records indicates that historical groundwater levels in the site vicinity range from 11 to 60 feet below ground surface.

Disaggregation of the MCE peak ground acceleration was performed using the USGS online Earthquake Hazard Toolbox, NSHM Conterminous U.S. 2018 edition. The result of the disaggregation 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 8.72 kilometers from the site.

We performed a liquefaction analysis of the soils underlying the site using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. The liquefaction potential evaluation was performed by utilizing a historic high groundwater depth of 11 feet, a Maximum Considered Earthquake (MCE) modal magnitude of 8.10 based on a 2,475-year return period (2% in 50 years), and a Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration of 0.76g for the site.

Our evaluation of liquefaction indicates the alluvium below the planned improvements and anticipated engineered fill depth would not be prone to liquefaction. Additionally, we opine that seismically induced “dry-sand” settlement is not a design consideration ($\leq 1/4$ -inch). An analysis of liquefaction and seismically induced “dry-sand” settlement is included in Appendix C.

6.5 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes very gently to the northwest. Riverside General Safety Plan indicates the site is not within an area identified as having a potential for slope instability (RCTLMA, 2021). Additionally, the site is not within an area identified as having potential for seismic slope instability (CGS 2025a, 2025b). There are no known landslides near the site, nor is the site in the path of any known or potential landslides (USGS, 2025). Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Rock Fall

The site is located approximately 1,200 feet away from the toe of the slope of the Berlesconi Hills with College Park and several structures between the site and the toe of slope. Therefore, the rock fall hazard at the site is low.

6.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by the failure of dams or other water-retaining structures due to earthquakes. Based on a review of the USGS dam inundation database, the site is not located within a potential inundation area for an earthquake-induced dam failure of the Perris or other reservoirs. 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. A large ridge is present between the site and Perris Reservoir, which will direct any overflow within the lake to the southwest in front of the dam and away from the site location. Therefore, flooding resulting from a seismically induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2025).

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, 2025). 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 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 (RCIT, 2025; USGS, 2025) and is underlaid by granitic bedrock. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general vicinity. There appears to be little or no potential for ground subsidence due to the withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

- 7.1.1 Soil or geologic conditions were not encountered during the investigation that would preclude the construction of the proposed improvements, provided the recommendations presented herein are followed and implemented during construction.
- 7.1.2 Potential geologic hazards at the site include seismic shaking. Based on our investigation and available geologic information, active, potentially active, or inactive faults are not present underlying or trending toward the site.
- 7.1.3 The site is located approximately 5 miles from the nearest active fault. Based on our background research, potentially active or inactive faults do not extend across the site. Risks associated with seismic activity consist of the potential for moderate to strong seismic shaking.
- 7.1.4 Our field investigation indicates the site is underlain by artificial fill, older alluvium, and granitic bedrock. The undocumented fill and upper portion of the alluvial material are not considered suitable for the support of compacted fill, pavements, and settlement-sensitive structures. Remedial grading is required as recommended herein. The existing site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed.
- 7.1.5 Soil samples tested for hydrocompression exhibit some collapse potential that can be minimized by the recommended remedial grading of the near-surface soils.
- 7.1.6 The laboratory tests indicate that the site soils should be considered to have a “very low” expansion potential, and are thus considered not to be expansive. If expansive soils are encountered at the site, they should be exported from the site or selectively graded and placed in the deeper fill areas to allow for the placement of less expansive material at the finish pad grade.
- 7.1.7 We did not encounter groundwater during our investigation and do not expect groundwater would impact site improvements. However, wet conditions and seepage could affect proposed construction if grading and improvement operations occur during or shortly after a rain event.

- 7.1.8 Foundations for the proposed structure and associated ancillary structures such as block walls or planter walls (if proposed), may be supported on a shallow conventional foundation system following remedial grading. Over-excavation and compaction of site soils must be observed and approved by a representative of Geocon.
- 7.1.9 As a minimum, it is recommended that the upper 5 feet of existing earth materials within the proposed building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any existing fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). Where space permits, the excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of soil removal will be verified by the Geocon representative during the site grading activities.
- 7.1.10 Subsequent to the recommended grading, the proposed structure may be supported on conventional shallow spread foundations, deriving support on at least 3 feet of newly placed engineered fill. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.
- 7.1.11 Due to the granular nature of soils and potential for caving, the contractor should be prepared to form foundation excavations, if necessary.
- 7.1.12 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. Excavations up to 5 feet are expected during grading operations (exclusive of remedial grading) and utility installation. Excavations should be performed in conformance with OSHA requirements. Some site soils may have little to no cohesion and may be subject to caving in un-shored excavations. The contractor's competent person should evaluate the necessity for the layback of vertical cut areas or the use of shoring.
- 7.1.13 Proper drainage should be maintained in order to preserve the design properties of the fill in the sheet-graded pad and slope areas.

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

7.2 Soil and Excavation Characteristics

- 7.2.1 The in-situ soils and weathered bedrock can be excavated with moderate effort using conventional excavation equipment. Due to the granular nature of the soils, moderate caving should be anticipated in vertical excavations. In addition, the contractor should also be aware that formwork may be required to prevent caving of shallow spread foundation excavations.
- 7.2.2 Excavations up to 5 feet are expected during grading operations and utility installation. Excavations should be performed in conformance with OSHA requirements. Some site soils may have little to no cohesion and may be subject to caving in un-shored excavations. The contractor's competent person should evaluate the necessity for the layback of vertical cut areas as well as the use of formwork and shoring.
- 7.2.3 Based on laboratory testing, the upper 5 feet of soils at the site generally possess a "very low" expansion potential (expansion index (EI) of 0) and are considered "non-expansive" as defined by 2025 CBC Section 1803.5.3. The following table presents soil classifications based on the EI.

SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	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	

- 7.2.4 Laboratory testing was performed on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents the results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess a sulfate content of 0.016 percent, equating to an exposure class of "S0" as defined by 2025 CBC Section 1904.3 and ACI 318-19 (Chapter 19).

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Exposure Class	Water-Soluble Sulfate (SO ₄) Percent by Weight	Cement Type (ASTM C150)		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	SO ₄ >2.00	Option 1	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 The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, landscaping activities along the access roads or from nearby developments (i.e., addition of fertilizers and other soil nutrients) may affect the concentration over time.
- 7.2.6 According to Caltrans *Corrosion Guidelines* (Version 3.2; May 2021), soils are considered corrosive to buried concrete and steel structures if one or more of the following conditions exist: chloride concentration is 500 parts per million (ppm) or greater, or sulfate concentration is 1,500 ppm or greater, or the potential of hydrogen (pH) is 5.5 or less. Resistivity serves as an indicator parameter for the possible presence of soluble salts and is not included as a parameter to define a corrosive area for structures. A minimum resistivity value for soil and/or water less than 1,500 ohm-centimeters may indicate the presence of high quantities of soluble salts and a higher propensity for corrosion.
- 7.2.7 Laboratory testing indicates the site soils have a minimum electrical resistivity of 960 ohm-cm, possess 45 ppm chloride, less than 10 ppm sulfate, and a pH of 8.3. As shown in the table below, the site would be classified as “corrosive” to buried metallic improvements, in accordance with the Caltrans *Corrosion Guidelines* (Version 3.2; May 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

- 7.2.8 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 Earthwork should be observed, and compacted fill tested by representatives of Geocon.
- 7.3.2 Grading should be performed in accordance with the recommendations provided in this report and the grading ordinances of Moreno Valley.
- 7.3.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with a qualified representative of Riverside Community College District, contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.
- 7.3.4 The site soils are suitable for re-use as an engineered fill provided oversized material (greater than 6 inches) and deleterious debris are removed. Deleterious debris must not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the geotechnical engineer. Existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein.
- 7.3.5 Site preparation should commence with the removal of existing improvements from the area to be graded. The areas to receive compacted fill shall be stripped of vegetation, existing artificial fill, loose or disturbed soils, and deleterious material. Loose or disturbed soils encountered at the excavation bottom should be properly compacted for the support of the proposed structures. Excavation bottoms must be observed and approved in writing by a representative of Geocon prior to placing backfill.

- 7.3.6 Artificial fill and the upper portion of the alluvial material within a 1:1 (h:v) projection of the limits of grading should be removed to expose competent alluvial soil having a minimum of 85 percent relative compaction. Where space permits, the excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Removals for the building pad area should extend to a depth of 5 feet below the existing grade, and all foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. Removals in pavement areas and walkway areas should extend at least 2 feet beneath the pavement or flatwork subgrade elevation.
- 7.3.7 Areas of loose, dry, or compressible soils will require deeper excavation and processing prior to fill placement. Excavated site soils should be thoroughly blended and moisture conditioned prior to placement and compaction. The actual depth of removal should be evaluated by a representative of Geocon during grading operations. Where over-excavation and compaction is to be conducted, the excavations should be extended laterally a minimum distance of 5 feet beyond the structure footprint. The bottom of the excavation should be scarified to a depth of at least 6 inches, moisture conditioned to approximately 2 percent above optimum moisture content, and properly compacted to 90 percent of the maximum dry density as determined by ASTM D1557.
- 7.3.8 Import fill (if necessary) should consist of granular materials with a “very low” expansion potential (EI of 20 or less), be non-corrosive, generally free of deleterious material and contain no rock fragments larger than 6 inches. Geocon should be notified of the proposed import soil source so that geotechnical laboratory testing can be performed to evaluate its suitability for use as fill material prior to delivery to the site. In addition, the project environmental consultant should perform environmental testing of the import soil in accordance with Riverside Community College District requirements before the material is brought to the site.
- 7.3.9 Fill and backfill soils should be placed in horizontal loose layers no thicker than will allow for adequate bonding and compaction (approximately 6 to 8 inches thick), moisture conditioned to approximately 2 percent above optimum moisture content, and compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557. Fill materials placed below the moisture content recommended will require additional moisture conditioning prior to placing additional fill.

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 the City of Moreno 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 20) to a depth of at least 1 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. Backfill of utility trenches should not contain rocks greater than 3 inches in size. The use of 2-sack slurry and controlled low strength material (CLSM) are also acceptable as backfill. However, consideration should be given to the possibility of differential settlement where the slurry ends and earthen backfill begins. These transitions should be minimized and additional stabilization should be considered at these transitions.
- 7.5.2 Utility trench backfill should be placed in layers no thicker than will allow for adequate bonding and compaction. Utility backfill should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density and moisture conditioned of approximately 2 percent above optimum moisture content as determined by ASTM D1557. The upper 12 inches of subgrade for new pavements should be compacted to at least 95 percent of the maximum dry density.

7.6 Foundation Design

- 7.6.1 Subsequent to the recommended grading, the proposed structure may be supported on conventional shallow spread foundations, deriving support on at least 3 feet of newly placed engineered fill.
- 7.6.2 Due to the granular nature of soils and potential for caving, the contractor should be prepared to form foundation excavations, if necessary.
- 7.6.3 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 Capacity – Continuous Foundation	2,500 psf
Allowable Bearing Capacity – Isolated Foundation	3,000 psf
Bearing Capacity Increase	250 psf per Foot of Width
	500 psf per Foot of Depth
Maximum Allowable Bearing Capacity	3,500 psf
Estimated Static Total Settlement	1 Inch
Estimated Static Differential Settlement	¾ Inch over 20 Feet
Estimated Combined Static + Seismic Settlement	1 Inch
Estimated Static + Seismic Differential Settlement	¾ Inch over 20 Feet

- 7.6.4 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.
- 7.6.5 Once the design and foundation loading configurations for the proposed structures proceed 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.

- 7.6.6 No special subgrade presaturation is required before placing concrete. However, the foundation subgrade should be lightly sprinkled as needed to maintain a uniformly moist condition at the time of concrete placement, preventing excessive water loss from the concrete due to plastic shrinkage.
- 7.6.7 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (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. If unanticipated soil conditions are encountered, foundation modifications may be required.
- 7.6.8 This office should be provided with a copy of the final construction plans so that the excavation recommendations presented herein can be properly reviewed and revised if necessary.

7.7 Miscellaneous Foundations

- 7.7.1 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 deriving support on a minimum of 12 inches of newly placed engineered fill, which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, foundations may derive support directly in the competent undisturbed alluvial soils at and below a depth of 24 inches and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.
- 7.7.2 If the soils exposed in the excavation bottom are soft, 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. Miscellaneous foundations may be designed for a bearing value of 1,500 psf and should be a minimum of 12 inches in width, 24 inches in depth below the 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.

- 7.7.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (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.

7.8 Lateral Design

- 7.8.1 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 older alluvial soils.
- 7.8.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill or undisturbed older alluvial soils may be computed as an equivalent fluid having a density of 270 pounds per cubic foot (pcf) with a maximum earth pressure of 2,700 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.

7.9 Concrete Slabs-on-Grade

- 7.9.1 Concrete slabs-on-grade subject to vehicle loading, if any, should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report.
- 7.9.2 Unless designed by a qualified structural engineer, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 5 inches thick, and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.
- 7.9.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder selection and design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) as well as ASTM E1745

and should be installed in general conformance with ASTM E1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils of extruded polyolefin plastic is recommended; vapor retarders that contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning is recommended. The vapor retarder should be installed in direct contact with the concrete slab with a proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture-resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve as a capillary break and may reduce the potential for punctures and damage to the vapor barrier.

- 7.9.4 For seismic design purposes, a coefficient of friction of 0.4 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.
- 7.9.5 Exterior slabs for walkways and flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to the construction of slabs, the upper 12 inches of subgrade should be moistened to approximately 2 percent above optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by the project structural engineer.
- 7.9.6 No special subgrade presaturation is required before placing concrete. However, the slab subgrade should be lightly sprinkled as needed to maintain a uniformly moist condition at the time of concrete placement, preventing excessive water loss from the concrete due to plastic shrinkage.

- 7.9.7 The recommendations of this report are intended to reduce the potential for cracking slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals.

7.10 Pavement Design

- 7.10.1 The final pavement sections for roadways should be based on the R-value of the subgrade soils encountered at final subgrade elevation. The civil engineer should evaluate the traffic index for pavements. Pavements should be designed in accordance with the City of Moreno Valley's 2022 *Standard Plans, Street Improvements (Section 1)*, when final Traffic Indices and R-value test results of subgrade soil are completed.

Based on preliminary testing, the upper soils from boring B8 were found to have an R-value of 71. An assumed R-value of 50 was used in our preliminary evaluation of pavements. Calculated preliminary flexible pavement sections are presented in the following table (these sections are based on City of Moreno Valley minimum requirements).

PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Aggregate Base (inches)
Local Street / Modified Local Street	6	50	3.3	4.0
Collector	7		3.9	4.0

- 7.10.2 Prior to the construction of new pavement sections, remedial grading should be performed in accordance with the *Grading* section in this report.
- 7.10.3 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 2 percent above optimum moisture content (ASTM D1557).

- 7.10.4 The crushed aggregate base and asphalt concrete materials should conform to Section 200-2.2 and Section 203-6, respectively, of the latest edition of the *Standard Specifications for Public Works Construction* (Greenbook). Aggregate base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density, at 2 percent above optimum moisture content. Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density in accordance with ASTM D1561.
- 7.10.5 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

- 7.10.6 We used the parameters presented in the following table 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	500 psi
Concrete Compressive Strength	3,000 psi
Concrete Modulus of Elasticity, E	3,150,000

- 7.10.7 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	5½
B = Entrance and Truck Service Lanes	10	6
	50	6½
	100	6½
C = School or City Buses	50	9½
	100	9½
D = Heavy Duty Trucks	50	6½
	100	7
E = Garbage or Fire Truck Lanes	5	6½
	10	7

- 7.10.8 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 2 percent above optimum moisture content.
- 7.10.9 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)
$5 \leq T < 6$	12.5
$6 \leq T$	15

- 7.10.10 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

- 7.10.11 Reinforcing steel will not be necessary within the concrete for geotechnical purposes, with the possible exception of dowels at construction joints as discussed herein.
- 7.10.12 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.
- 7.10.13 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.
- 7.10.14 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, at 2 percent above optimum moisture content. Cross-gutters that receive vehicular traffic 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.

7.11 Retaining Wall Design

- 7.11.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. If walls higher than 5 feet are planned, Geocon should be contacted for additional recommendations.
- 7.11.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report.
- 7.11.3 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). 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). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained.

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 5	34	58

- 7.11.4 The wall pressures provided above assume that the proposed retaining walls will support engineered fill or relatively undisturbed alluvial soils with an Expansion Index of 50 or less.
- 7.11.5 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 92 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.
- 7.11.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

- 7.12.1 Retaining walls not designed for hydrostatic pressures should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface. An image depicting typical retaining wall drainage is provided below. The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.



- 7.12.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap. These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.
- 7.12.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

- 7.12.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained to recommend a product or method which would provide protection to subterranean walls, floor slabs and foundations.

7.13 Elevator Pit Design

- 7.13.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report.
- 7.13.2 Additional active 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.
- 7.13.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report.
- 7.13.4 Subdrainage pipes at the base of the retaining wall drainage system should outlet to a location acceptable to the building official.
- 7.13.5 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation are not the responsibility of the geotechnical engineer.

7.14 Elevator Piston

- 7.14.1 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, especially if the drilling is performed subsequent to the foundation construction.

7.14.2 The site soils are granular and subject to caving. Casing may be required if caving is experienced in the drilled excavation. 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 (a representative of Geocon West, Inc.) is required.

7.14.3 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.

7.15 Temporary Excavations

7.15.1 Excavations on the order of 5 feet in height may be required during grading and construction operations. The excavations are expected to expose fill and alluvial soils, which may 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; however, the contractor should be prepared for caving, sloughing, and raveling in open excavations.

7.15.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. We anticipate that stable excavations for the construction of the proposed improvements can be achieved and maintained with sloping measures. Where sufficient space is available, temporary unsurcharged embankments could be sloped. The inclination should be selected by the contractor's competent person based on the soils exposed in the excavation in accordance with OSHA regulations.

7.15.3 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.

7.15.4 Performing continuous vertical excavations along property lines or adjacent to an existing structure could remove support from the property and/or structure, which is not acceptable. Excavations in close proximity to an adjacent property line or structure will require special excavation measures, such as shoring. Shoring recommendations will be provided under separate cover, if necessary.

7.16 Surface Drainage

- 7.16.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of excess irrigation and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.
- 7.16.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2025 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains, and scuppers is not recommended onto unprotected soils within 5 feet of the building perimeter. Planters that are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils that provide foundation support. Planters that saturate the subsurface or stormwater infiltration structures should not be used within 20 feet of the proposed buildings or other on-grade improvements. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.
- 7.16.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. Localized surface settlement should be expected in the vicinity of the stormwater infiltration structures or other areas where water is allowed to infiltrate to the subsurface.
- 7.16.4 Landscaping planters immediately 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. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall or an impermeable geosynthetic along the edge of the pavement that extends at least 12 inches below the base material.

7.17 Plan Review

- 7.17.1 Grading and foundation plans should be reviewed by the Geotechnical Engineer (a representative of 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.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

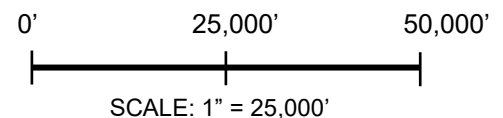
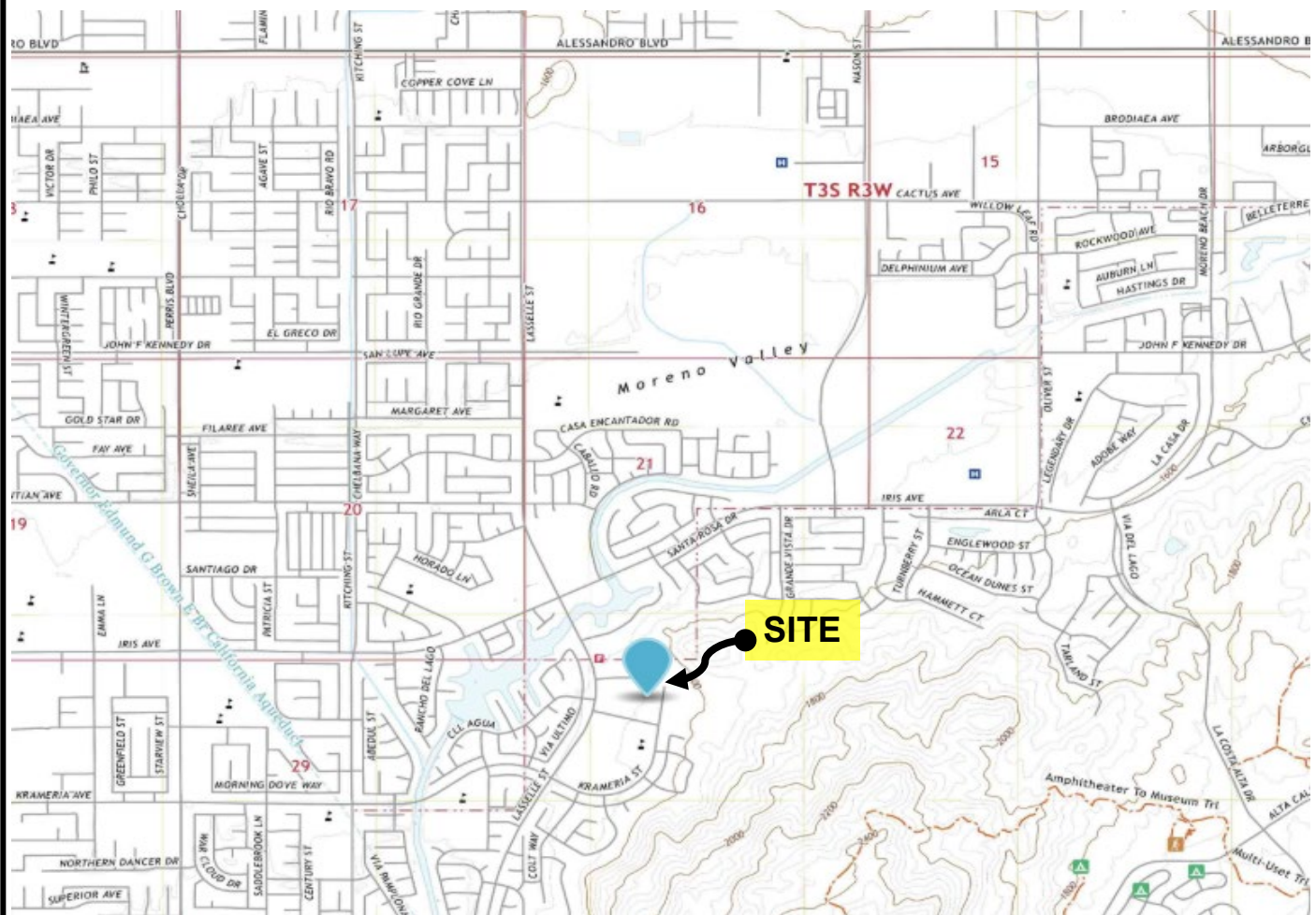
1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.
2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.
3. This report is issued with the understanding that it is the responsibility of the owner or his representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.
4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

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SOURCE: California Sunnymead Quadrangle, U.S. Geological Survey, 2021

VICINITY MAP

GEOCON
WEST, INC.



GEOTECHNICAL, ENVIRONMENTAL, MATERIALS
41571 CORNING PLACE #101, MURRIETA, CALIFORNIA 92562
PHONE 951-304-2300 www.geoconinc.com

DRAFTED BY: HD

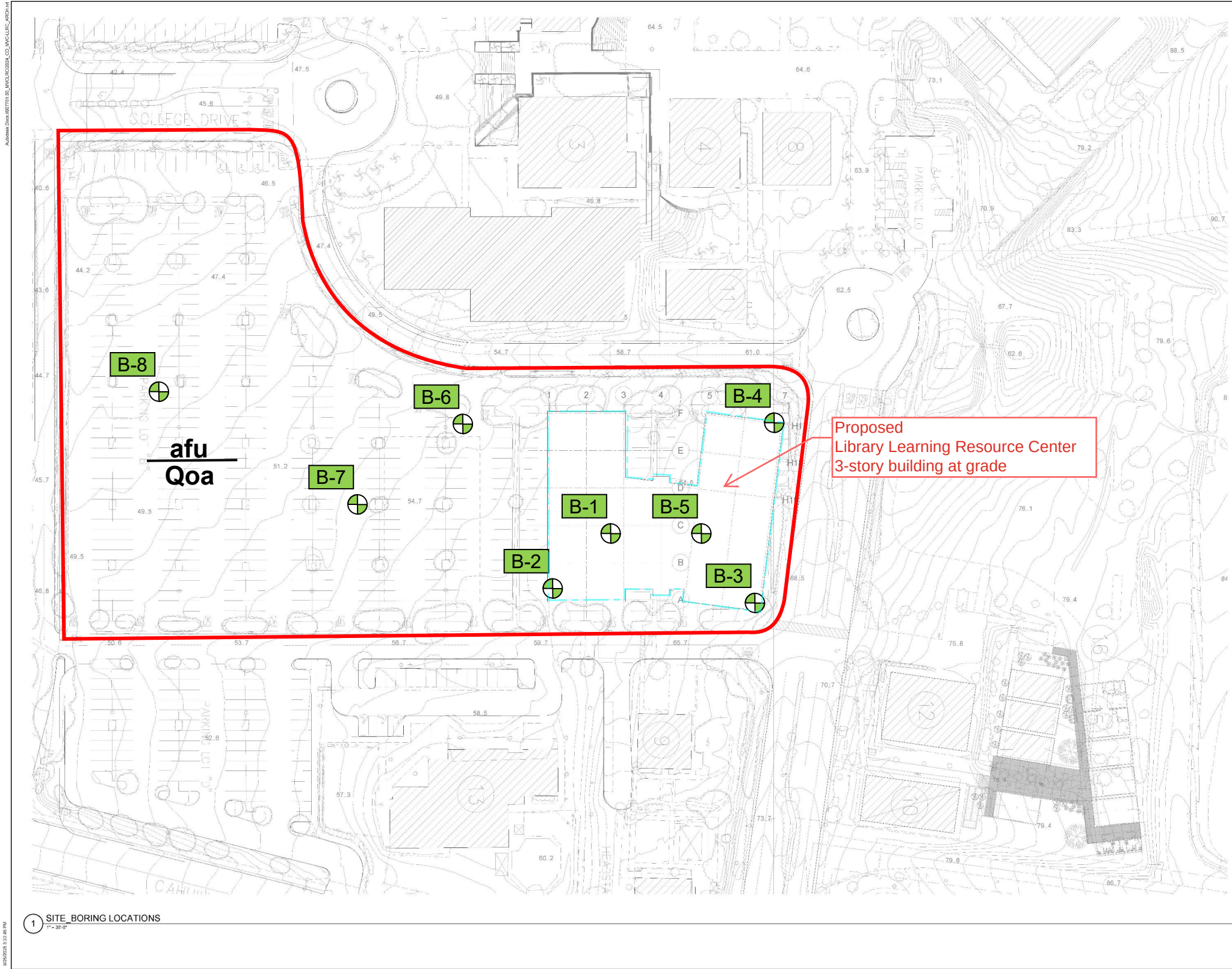
REVIEWED BY: LAB

MORENO VALLEY COLLEGE – LLRC
SW OF LA SALLE STREET & COLLEGE DRIVE
MORENO VALLEY, CALIFORNIA

OCTOBER 2025

PROJECT NO. T2875-22-05

FIG. 1



GEOCON LEGEND

Locations are approximate

- B-8 GEOTECHNICAL BORING
- afu UNDOCUMENTED ARTIFICIAL FILL
- Qoa OLDER ALLUVIUM
- LIMITS OF THIS INVESTIGATION

Moreno Valley College
Library Learning
Resource Center (LLRC)

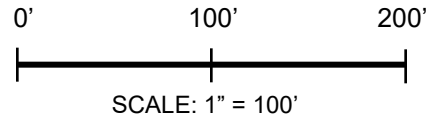
CANNONDESIGN
2170 Whitehaven Road
Grand Island, New York 14072
T: 716.773.6600
F: 716.773.5500

No. Description Date

Drawing Title:
SITE PLAN - BORING
LOCATIONS

Project No. 000000.00 Checked by: Checker

A0051



Source: CANNONDESIGN, Site Plan – Boring Locations, Sheet A0051, 8/25/2025



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41571 CORNING PLACE #101, MURRIETA, CALIFORNIA 92562
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GEOLOGIC MAP & SITE PLAN

MORENO VALLEY COLLEGE – LLRC
SW OF LA SALLE STREET & COLLEGE DRIVE
MORENO VALLEY, CALIFORNIA

DRAFTED BY: HD

REVIEWED BY: LAB

OCTOBER 2025

PROJECT NO. T2875-22-05

FIG. 2

APPENDIX

A

APPENDIX A

FIELD INVESTIGATION

The site was explored on September 15, 2025, by excavating eight 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine to depths ranging between 11.5 and 55 feet beneath the ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A8. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The approximate locations of the borings are shown on Figure 2.



PROJECT NAME Moreno Valley College LLRC

LOGGED BY KD

PROJECT NUMBER T2875-22-05

LATITUDE / LONGITUDE 33.886064, -117.202500

BORING DATE 09/15/2025

FIGURE NUMBER A-1

DEPTH 56.5'

SURFACE ELEVATION ~1563'

LOCATION Moreno Valley

CLIENT NAME RCCD

DRILLING FIRM 2R DRILLING

COMPLETED 09/15/2025

EQUIPMENT CME-75

-

METHOD Cal-Mod / SPT

BORING DIAMETER 8 in

HAMMER TYPE Auto

NOTES -

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
1563											
2	1560			SM	ASPHALT, 5" ARTIFICIAL FILL (af) Silty SAND, dense, slightly moist, dark brown, fine to coarse, micaceous, slightly cemented			B-1@0-5'			
4								B-1@2.5'	5 11 40	123.9	10.5
6				SM	OLDER ALLUVIUM (Qoa) Silty SAND, medium dense, slightly moist, dark brown, fine to coarse, with some gravel, micaceous, slightly porous cemented			B-1@5'	10 19 22		8.2
8	1555				some porosity, trace gravels, up to 1"			B-1@7.5'	22 29 21	116.0	7.3
10								B-1@10'	9 11 10		9.1
12	1550										
14											
16					very dense, brown, pinhole porosity, slightly cemented			B-1@15'	20 50/6"	134.5	8.5
18	1545										
20											
22					medium dense			B-1@20'	9 12 13		8.6
24	1540										
26				SM	Silty SAND, dense, moist, brown, fine to coarse, micaceous, cemented			B-1@25'	14 37 34	133.8	9.1
28	1535				increase in fine						

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.



SOIL BORING: B-1

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
32	1530			SM	dense, moist, dark brown, fine to coarse, micaceous, cemented			B-1@30'	18 28 23		8.0
34											
36											
38	1525				QUARTZ DIORITE (Qdx) hard, highly weathered			B-1@35'	18 25 30	119.1	15.1
40					light gray, in filled joint						
42	1520							B-1@40'	50/4"		4.2
44					extremely hard, moderately weathered, drilling slowed						
46					water added to help drilling						
48	1515							B-1@45'	50/3"	118.3	2.9
50					very hard drilling						
52	1510							B-1@50'	50/4"		1.6
54											
56					No recovery on sampler			B-1@55'	50/2"		
Total Depth: 55' 2" (Refusal) No groundwater encountered Backfilled with cuttings and capped with AC											

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.



PROJECT NAME Moreno Valley College LLRC

LOGGED BY KD

PROJECT NUMBER T2875-22-05

LATITUDE / LONGITUDE 33.885899, -117.202575

BORING DATE 09/15/2025

FIGURE NUMBER A-2

DEPTH 21.5'

SURFACE ELEVATION ~1566'

LOCATION Moreno Valley

CLIENT NAME RCCD

DRILLING FIRM 2R DRILLING

COMPLETED 09/15/2025

EQUIPMENT CME-75

-

METHOD Cal-Mod / SPT

BORING DIAMETER 8 in

HAMMER TYPE Auto

NOTES -

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
1566											
2	1565			SM	ASPHALT, 5" ARTIFICIAL FILL (af) Silty SAND, loose, slightly moist, dark brown, fine to medium, few coarse sand			B-2@0-5'			
4				SM	OLDER ALLUVIUM (Qoa) Silty SAND, medium dense, dry, light brown, fine to medium, some coarse sand, mottled coloring, pinhole porosity, micaceous			B-2@2.5'	7 8 8	116.0	3.2
6	1560							B-2@5'	6 8 10	114.1	2.3
8								B-2@7.5'	5 7 9	110.1	2.5
10	1555							B-2@10'	7 10 10	115.5	2.2
12											
14											
16	1550			SP-SM	Poorly Graded SAND w/ Silt, medium dense, slightly moist, light brown, fine to coarse, micaceous			B-2@15'	5 8 12	104.8	4.1
18											
20	1545										
				SM	Silty SAND, medium dense, dry, dark brown with light brown mottling, fine to coarse, micaceous, slightly laminated Total Depth: 21.5' No groundwater encountered Backfilled with cuttings and capped with AC			B-2@20'	7 12 10	109.7	2.4

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.



Soil Boring Number: B-3

Page 1 of 1

PROJECT NAME Moreno Valley College LLRC

LOGGED BY KD

PROJECT NUMBER T2875-22-05

LATITUDE / LONGITUDE 33.885883, -117.202056

BORING DATE 09/15/2025

FIGURE NUMBER A-3

DEPTH 21.5'

SURFACE ELEVATION ~1572'

LOCATION Moreno Valley

CLIENT NAME RCCD

DRILLING FIRM 2R DRILLING

COMPLETED 09/15/2025

EQUIPMENT CME-75

-

METHOD Cal-Mod / SPT

BORING DIAMETER 8 in

HAMMER TYPE Auto

NOTES -

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
1572											
2	1570			SM	ASPHALT, 5.5" ARTIFICIAL FILL (af) Silty SAND, very dense, slightly moist, dark brown, fine to medium sand, few coarse sand, pinhole porosity, moderately cemented, micaceous			B-3@0-5'			
4				SM	OLDER ALLUVIUM (Qoa) Silty SAND, very dense, slightly moist, dark brown, fine to coarse, slightly porous, calcium carbonate stringers, cemented, micaceous			B-3@2.5'	30 50/5"	130.2	5.8
6	1565							B-3@5'	40 50/6"	116.9	4.7
8								B-3@7.5'	40 35 50	116.0	3.3
10								B-3@10'	26 40 50/3"	116.8	3.3
12	1560										
14											
16	1555			SP	Poorly Graded SAND, very dense, slightly moist, light brown, fine to coarse, few calcium carbonate stringers, micaceous			B-3@15'	23 33 48	121.9	3.1
18											
20				SM	Silty SAND, very dense, slightly moist, light brown to dark brown, fine to coarse, slightly cemented, calcium carbonate stringers, micaceous			B-3@20'	49 50/6"	124.7	5.4

Total Depth: 21.5'

No groundwater encountered

Backfilled with cuttings and capped with AC

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.



Soil Boring Number: B-4

Page 1 of 1

PROJECT NAME Moreno Valley College LLRC

LOGGED BY KD

PROJECT NUMBER T2875-22-05

LATITUDE / LONGITUDE 33.886336, -117.201977

BORING DATE 09/15/2025

FIGURE NUMBER A-4

DEPTH 21.5'

SURFACE ELEVATION ~1566'

LOCATION Moreno Valley

CLIENT NAME RCCD

DRILLING FIRM 2R DRILLING

COMPLETED 09/15/2025

EQUIPMENT CME-75

-

METHOD Cal-Mod / SPT

BORING DIAMETER 8 in

HAMMER TYPE Auto

NOTES -

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
1566											
2	1565			SM	ASPHALT, 5" AGGREGATE BASE, 6" ARTIFICIAL FILL (af) Silty SAND, very dense, moist, dark brown, fine to medium, few coarse sand, moderately cemented, micaceous			B-4@0-5'			
4				SM	OLDER ALLUVIUM (Qoa) Silty SAND, dense, dry to moist, light brown, fine to coarse, moderately cemented, micaceous			B-4@2.5'	30 50/6"	127.6	8.6
6	1560							B-4@5'	25 35 45	128.9	6.3
8					medium dense			B-4@7.5'	18 22 26	126.7	3.5
10	1555							B-4@10'	18 24 24	118.7	8.6
12											
14											
16	1550				dense			B-4@15'	18 26 33	133.1	6.1
18											
20	1545				QUARTZ DIORITE (Qdx) strong brown, fine- to coarse-grained, weak, moderately hard, highly weathered, iron oxidation, excavates as Poorly-graded SAND, slightly moist			B-4@20'	28 31 28	126.1	5.8

Total Depth: 21.5'

No groundwater encountered

Backfilled with cuttings and capped with AC

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.



PROJECT NAME Moreno Valley College LLRC

LOGGED BY KD

PROJECT NUMBER T2875-22-05

LATITUDE / LONGITUDE 33.886040, -117.202168

BORING DATE 09/15/2025

FIGURE NUMBER A-5

DEPTH 21.5'

SURFACE ELEVATION ~1566'

LOCATION Moreno Valley

CLIENT NAME RCCD

DRILLING FIRM 2R DRILLING

COMPLETED 09/15/2025

EQUIPMENT CME-75

-

METHOD Cal-Mod / SPT

BORING DIAMETER 8 in

HAMMER TYPE Auto

NOTES -

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
1566											
2	1565			SP-SM	ASPHALT, 5" ARTIFICIAL FILL (af) Poorly Graded SAND w/ Silt, dense, moist, light brown, fine to medium sand, moderately cemented, micaceous, oxidation staining			B-5@0-5'			
4				SP	OLDER ALLUVIUM (Qoa) Poorly Graded SAND, dense, slightly moist, brown with oxidation staining, fine to coarse, slightly cemented, micaceous brown mottles			B-5@2.5'	23 35 45	129.5	6.2
6	1560							B-5@5'	33 36 50	136.3	7.1
8								B-5@7.5'	19 25 33	125.0	9.4
10	1555				very dense			B-5@10'	21 35 50	129.6	9.4
12											
14											
16	1550				becomes medium dense			B-5@15'	28 21 23	132.5	5.5
18											
20	1545			SM	Silty SAND, dense, moist, dark brown, fine to medium, trace pinhole porosity, moderately cemented, micaceous			B-5@20'	15 25 38	130.6	7.5
Total Depth: 21.5' No groundwater encountered Backfilled with cuttings and capped with AC											

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.



Soil Boring Number: B-6

Page 1 of 1

PROJECT NAME Moreno Valley College LLRC

LOGGED BY KD

PROJECT NUMBER T2875-22-05

LATITUDE / LONGITUDE 33.886336, -117.202648

BORING DATE 09/15/2025

FIGURE NUMBER A-6

DEPTH 21.5'

SURFACE ELEVATION ~1562'

LOCATION Moreno Valley

CLIENT NAME RCCD

DRILLING FIRM 2R DRILLING

COMPLETED 09/15/2025

EQUIPMENT CME-75

-

METHOD Cal-Mod / SPT

BORING DIAMETER 8 in

HAMMER TYPE Auto

NOTES -

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
1562											
2	1560			SM	ASPHALT, 5" ARTIFICIAL FILL (af) Silty SAND, medium dense, dry, dark brown, fine to medium, few coarse sand, slightly cemented, micaceous, laminated			B-6@0-5'			
4								B-6@2.5'	9 23 18	125.4	2.8
6	1555			SP-SM	OLDER ALLUVIUM (Qoa) Poorly Graded SAND w/ Silt, loose, slightly moist to moist, dark brown, fine to medium, some coarse sand, micaceous, some rootlets			B-6@5'	5 6 9	91.4	3.9
8								B-6@7.5'	5 6 9	104.4	8.9
10								B-6@10'	6 6 8	113.8	2.8
12	1550			SM	Silty SAND, loose, dry, dark brown, fine to coarse, micaceous, porous						
14											
16	1545				becomes medium dense			B-6@15'	7 12 16	113.3	4.9
18											
20								B-6@20'	25 50/6"	97.4	1.3

Total Depth: 21.5'

No groundwater encountered

Backfilled with cuttings and capped with AC

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.



Soil Boring Number: B-7

Page 1 of 1

PROJECT NAME		Moreno Valley College LLRC		LOGGED BY		KD	
PROJECT NUMBER		T2875-22-05		LATITUDE / LONGITUDE		33.886302, -117.202870	
BORING DATE		09/15/2025		FIGURE NUMBER		A-7	
DEPTH		11.5'		SURFACE ELEVATION		~1557'	
LOCATION		Moreno Valley		CLIENT NAME		RCCD	
DRILLING FIRM		2R DRILLING		COMPLETED		09/15/2025	
EQUIPMENT		CME-75		-			
METHOD		Cal-Mod / SPT		BORING DIAMETER		8 in	
HAMMER TYPE		Auto		NOTES		-	
HAMMER WEIGHT / DROP		140 / 30					

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
1557											
2	1555			SM	ASPHALT, 5" ARTIFICIAL FILL (af) Silty SAND, medium dense, moist, dark brown, fine to medium, some coarse sand, trace pinhole porosity, micaceous			B-7@0-5'			
4				SP-SM	OLDER ALLUVIUM (Qoa) Poorly Graded SAND w/ Silt, loose, slightly moist, dark brown, fine to coarse, micaceous			B-7@2.5'	6 10 11	127.4	7.4
6	1550							B-7@5'	3 5 6	111.3	4.3
8				SP	Poorly Graded SAND, loose, dry, dark brown, medium to coarse			B-7@7.5'	6 7 8	119.5	3.2
10								B-7@10'	5 7 10	113.5	3.5

Total Depth: 11.5'
No groundwater encountered
Backfilled with cuttings and capped with AC

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.



Soil Boring Number: B-8

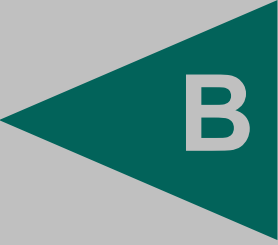
Page 1 of 1

PROJECT NAME	Moreno Valley College LLRC	LOGGED BY	KD
PROJECT NUMBER	T2875-22-05	LATITUDE / LONGITUDE	33.886402, -117.203759
BORING DATE	09/15/2025	FIGURE NUMBER	A-8
DEPTH	11.5'	SURFACE ELEVATION	~1550'
LOCATION	Moreno Valley	CLIENT NAME	RCCD
DRILLING FIRM	2R DRILLING	COMPLETED	09/15/2025
EQUIPMENT	CME-75		-
METHOD	Cal-Mod / SPT	BORING DIAMETER	8 in
HAMMER TYPE	Auto	NOTES	-
HAMMER WEIGHT / DROP	140 / 30		

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
1550											
2				SP-SM	ASPHALT, 5" <u>OLDER ALLUVIUM (Qoa)</u> Poorly Graded SAND w/ Silt, medium dense, slightly moist, brown to dark brown, fine to coarse, micaceous, slightly cemented			B-8@0-5'			
4					loose			B-8@2.5'	10 7 7	125.3	7.1
6	1545							B-8@5'	5 6 5	121.5	7.5
8					very dense			B-8@7.5'	19 38 50	130.0	7.5
10	1540							B-8@10'	28 38 36	130.4	5.9
Total Depth: 11.5' No groundwater encountered Backfilled with cuttings and capped with AC											

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

APPENDIX

**B**

APPENDIX B

LABORATORY TESTING

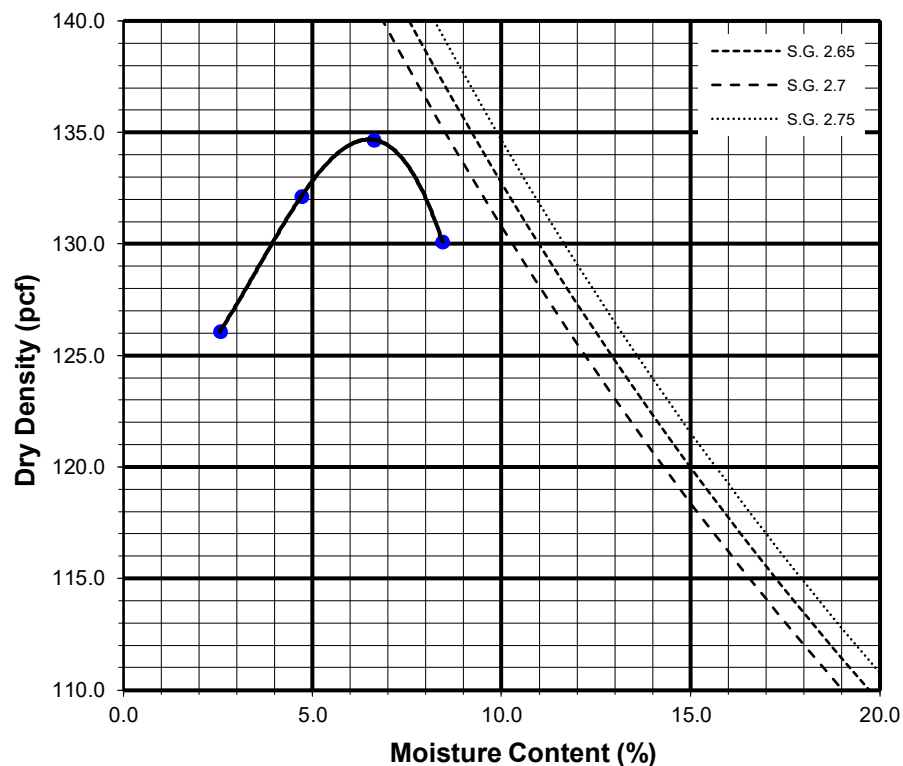
We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place dry density/moisture content, maximum density/optimum moisture, expansion index, water-soluble sulfate, pH, resistivity, water-soluble chloride ion content, grain size analysis, consolidation, direct shear strength, and R-value. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.

Sample No:

B2@0-5'	Silty Sand (SM)
----------------	-----------------

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6195	6332	6411	6373		
Weight of Mold	(g)	4242	4242	4242	4242	4242	
Net Weight of Soil	(g)	1953	2090	2169	2131	-4242	
Wet Weight of Soil + Cont.	(g)	648.3	677.7	618.0	615.4		
Dry Weight of Soil + Cont.	(g)	638.6	658.8	595.6	587.6		
Weight of Container	(g)	259.3	257.7	257.7	258.1		
Moisture Content	(%)	2.6	4.7	6.6	8.4		
Wet Density	(pcf)	129.3	138.4	143.6	141.1	-280.8	
Dry Density	(pcf)	126.1	132.1	134.7	130.1		

Maximum Dry Density (pcf)	135.0	Optimum Moisture Content (%)	6.5
----------------------------------	--------------	-------------------------------------	------------



Preparation Method: A

	COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS		Project No.: T2875-22-05	
	ASTM D-1557		LIBRARY LEARNING RESOURCE CENTER MORENO VALLEY COLLEGE MORENO VALLEY, CALIFORNIA	
	Checked by: KG		October 2025 Figure B1	

B3@0-5'

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)	631.2	647.5
Wt. of Mold	(gm)	196.7	196.7
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	473.7	647.5
Dry Wt. of Soil + Cont.	(gm)	453.8	405.7
Wt. of Container	(gm)	173.7	196.7
Moisture Content	(%)	7.1	11.1
Wet Density	(pcf)	131.1	135.8
Dry Density	(pcf)	122.4	122.2
Void Ratio		0.4	0.4
Total Porosity		0.3	0.3
Pore Volume	(cc)	56.7	56.2
Degree of Saturation	(%) [S_{meas}]	51.2	80.2

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
9/24/2025	10:00	1.0	0	0.3243
9/24/2025	10:10	1.0	10	0.3243
Add Distilled Water to the Specimen				
9/25/2025	10:00	1.0	1430	0.3219
9/25/2025	11:00	1.0	1490	0.3219

Expansion Index (EI meas) =	-2.4
Expansion Index (Report) =	0

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.

	EXPANSION INDEX TEST RESULTS		Project No.: T2875-22-05
	ASTM D-4829		LIBRARY LEARNING RESOURCE CENTER MORENO VALLEY COLLEGE MORENO VALLEY, CALIFORNIA
	Checked by: KG	October 2025	Figure B2

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)
B3@0-5'	8.3	960

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

Sample No.	Chloride Ion Content (%)
B3@0-5'	0.045

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

Sample No.	Water Soluble Sulfate (% SO ₄)	Sulfate Exposure
B3@0-5	0.016	S0



CORROSIVITY TEST RESULTS

Checked by: KG

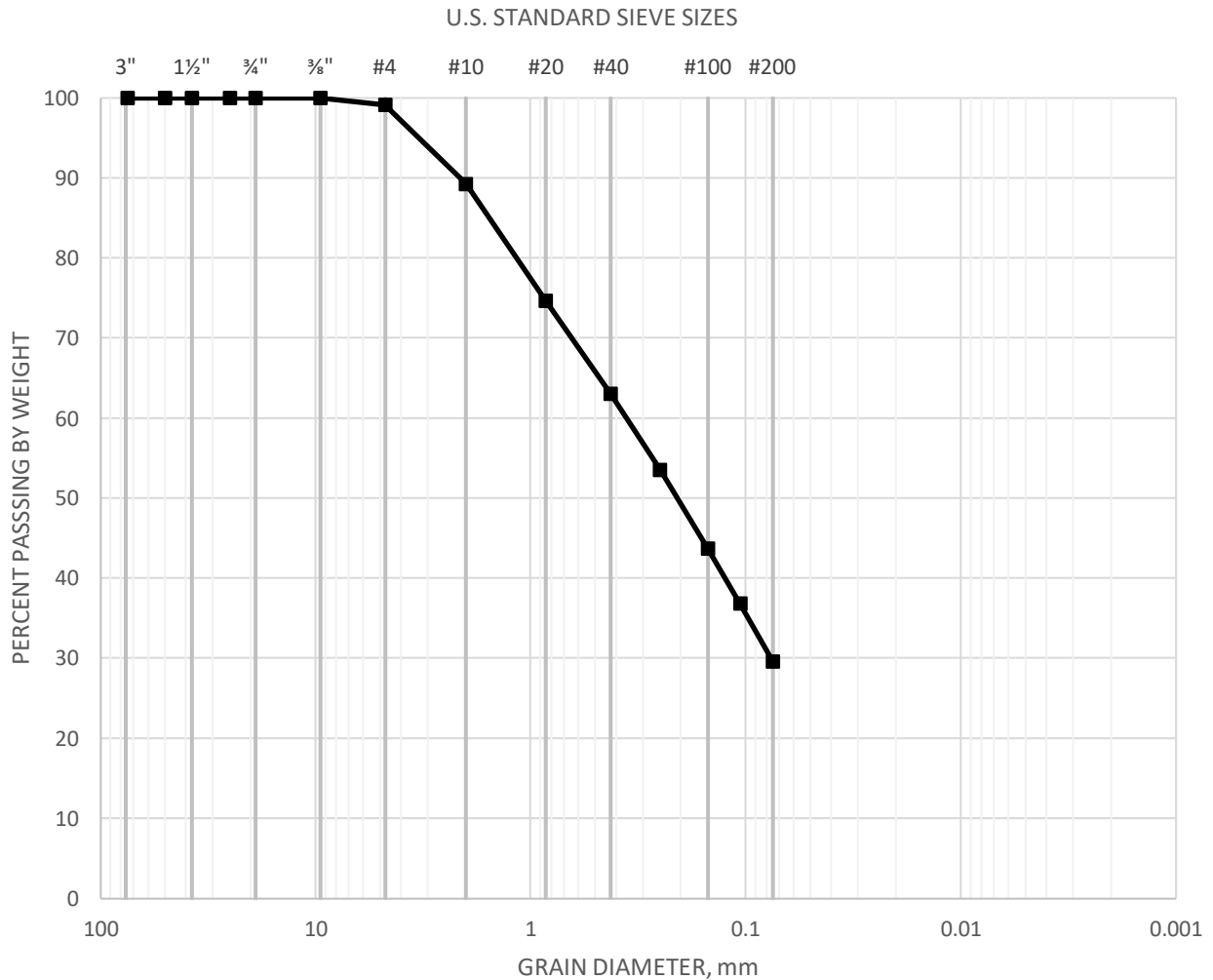
Project No.: T2875-22-05

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MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

October 2025

Figure B3

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
B1@10'	Silty Sand (SM)	0.36	0.075	0.075



GRAIN SIZE DISTRIBUTION

ASTM D 6913

Checked by: KG

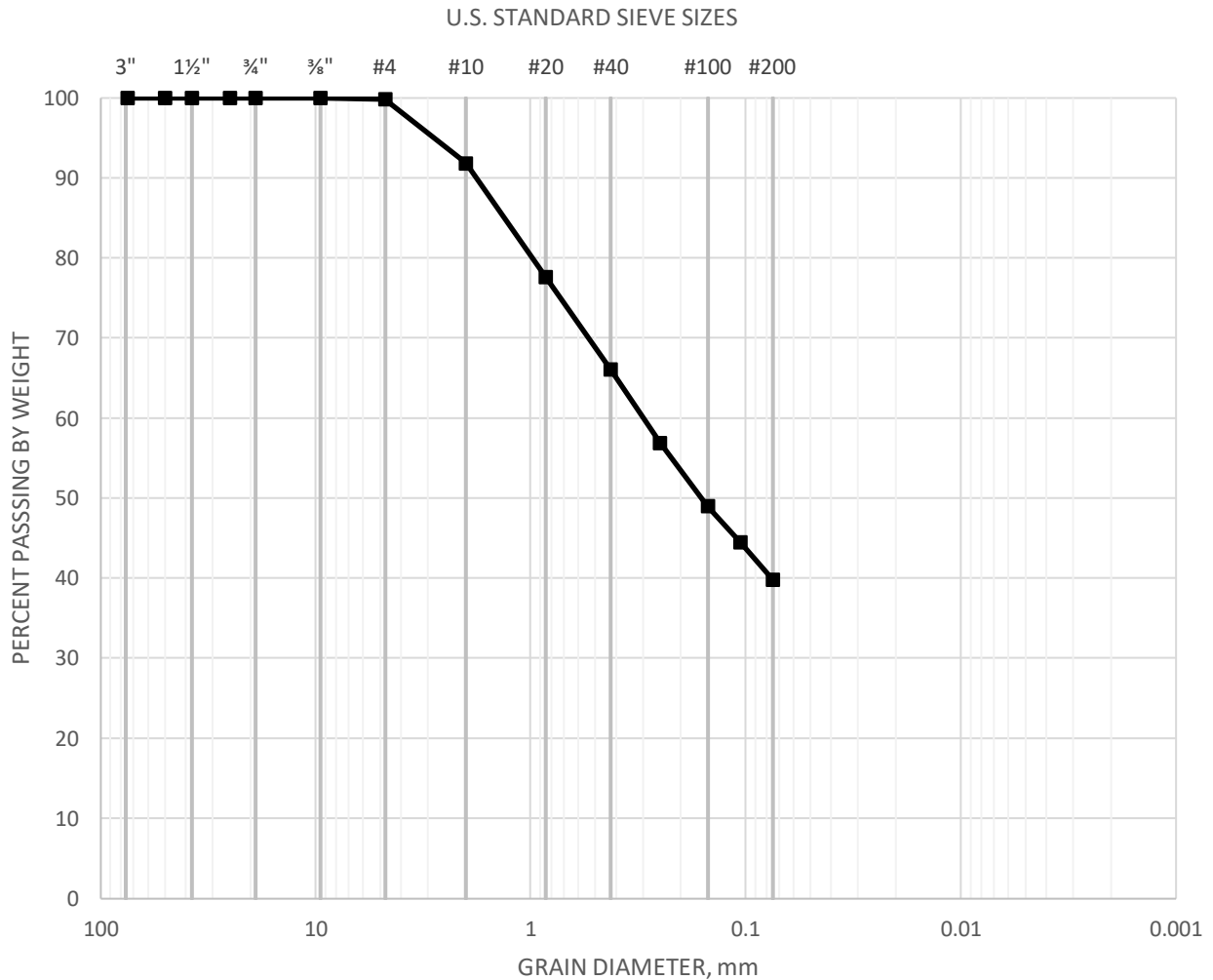
Project No.: T2875-22-05

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MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

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Figure B4

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
B1@25'	Silty Sand (SM)	0.3	0.075	0.075



GRAIN SIZE DISTRIBUTION

ASTM D 6913

Checked by: KG

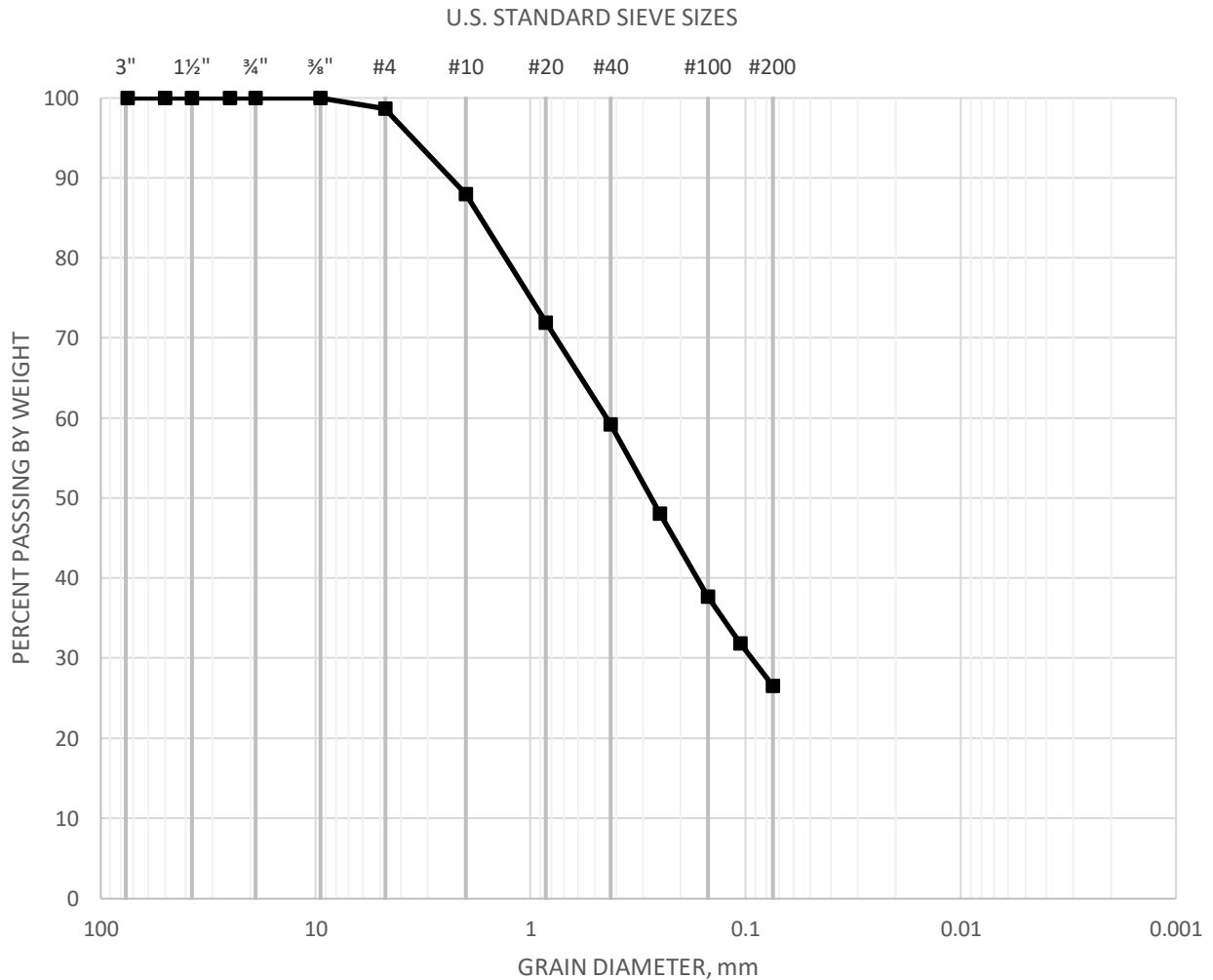
Project No.: T2875-22-05

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MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

Oct 25

Figure B5

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
B1@30'	Silty Sand (SM)	0.43	0.093	0.075



GRAIN SIZE DISTRIBUTION

ASTM D 6913

Checked by: KG

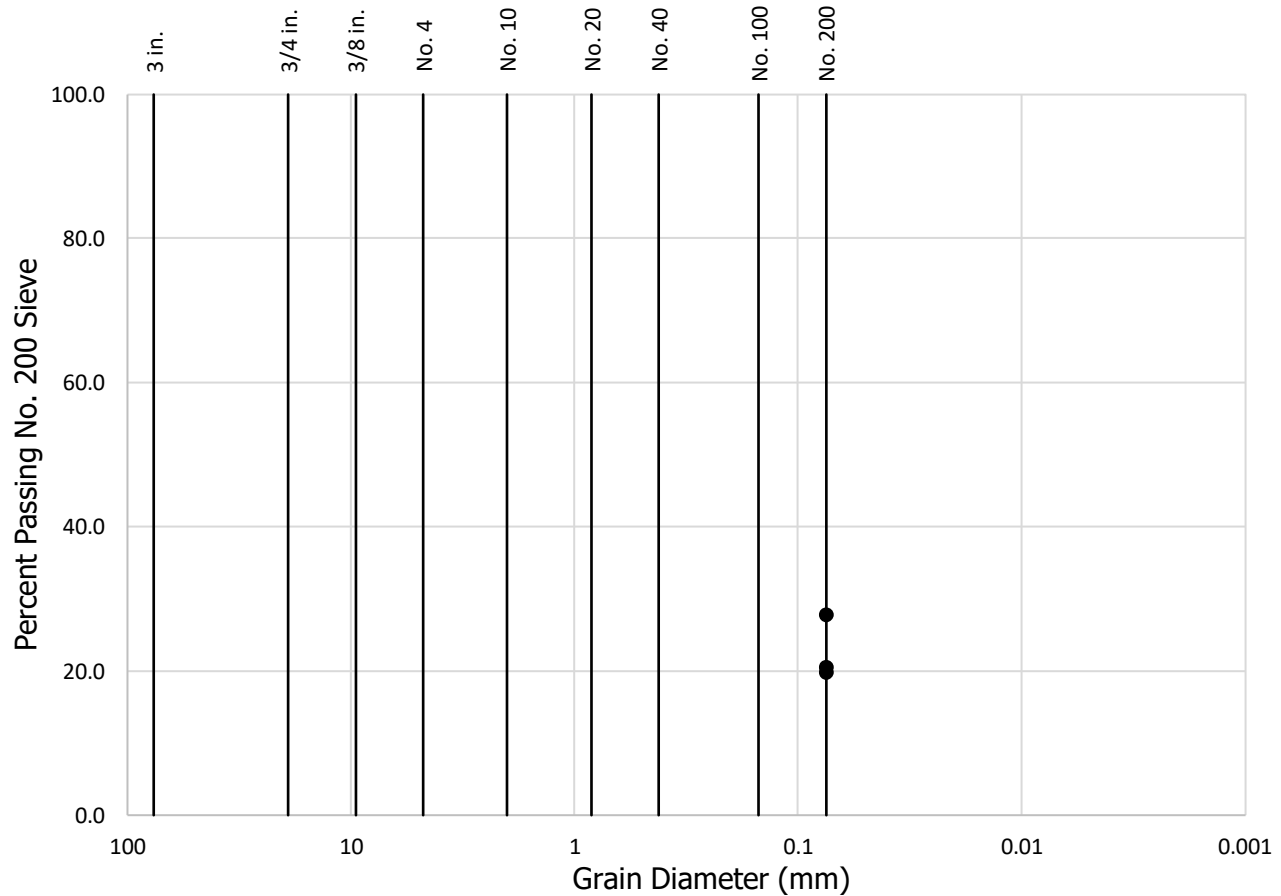
Project No.: T2875-22-05

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MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

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Figure B6

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Sample No.	Percent Passing No. 200 Sieve
B3 @ 10'	20.5
B4 @ 5'	27.8
B6 @ 10'	19.8



GRAIN SIZE ANALYSIS

ASTM D-1140

Checked by: KG

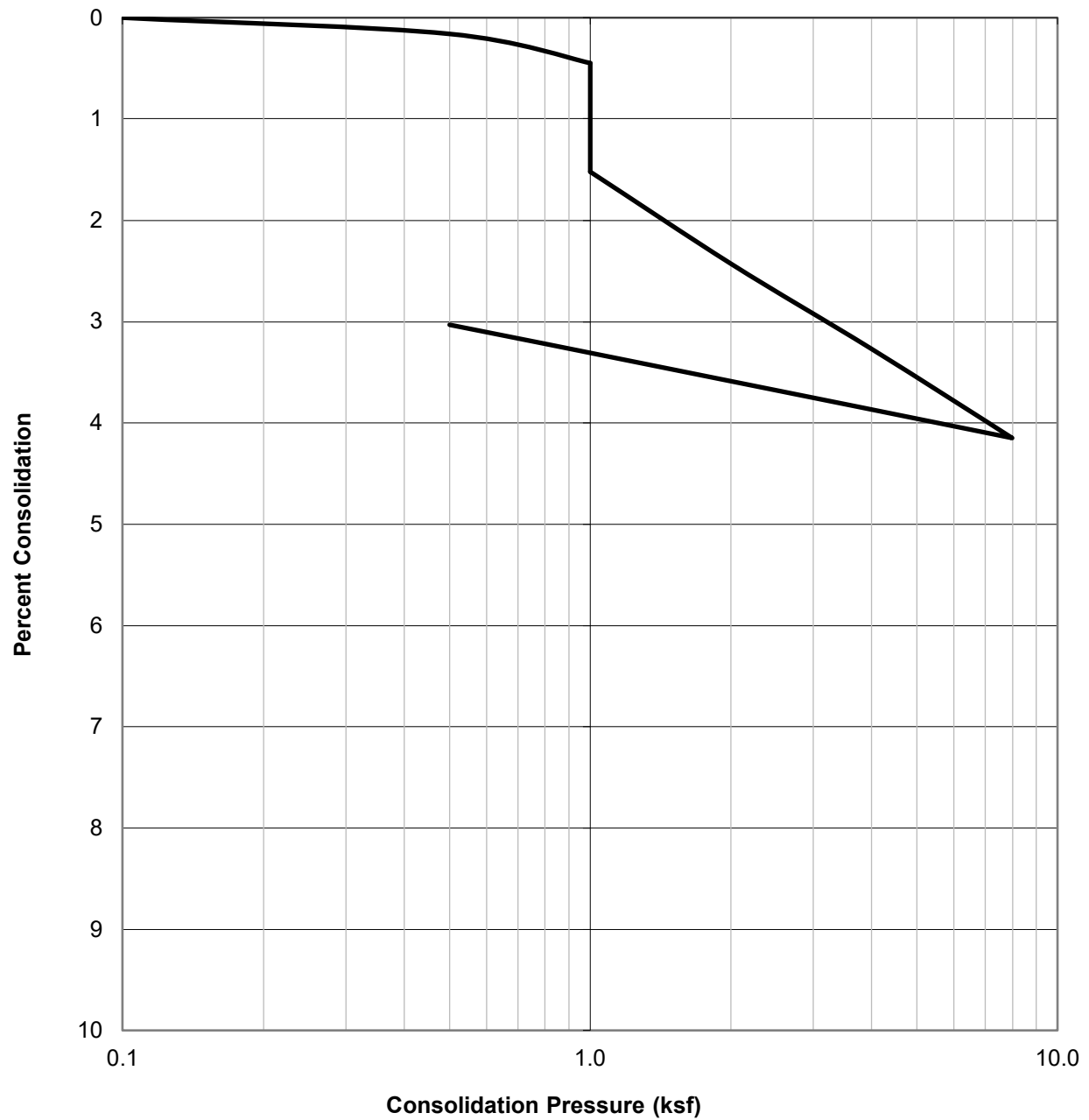
Project No.: T2875-22-05

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MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

October 2025

Figure B7

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@7.5	Silty SAND (SM), yellowish brown	116.6	3.3	11.9



GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

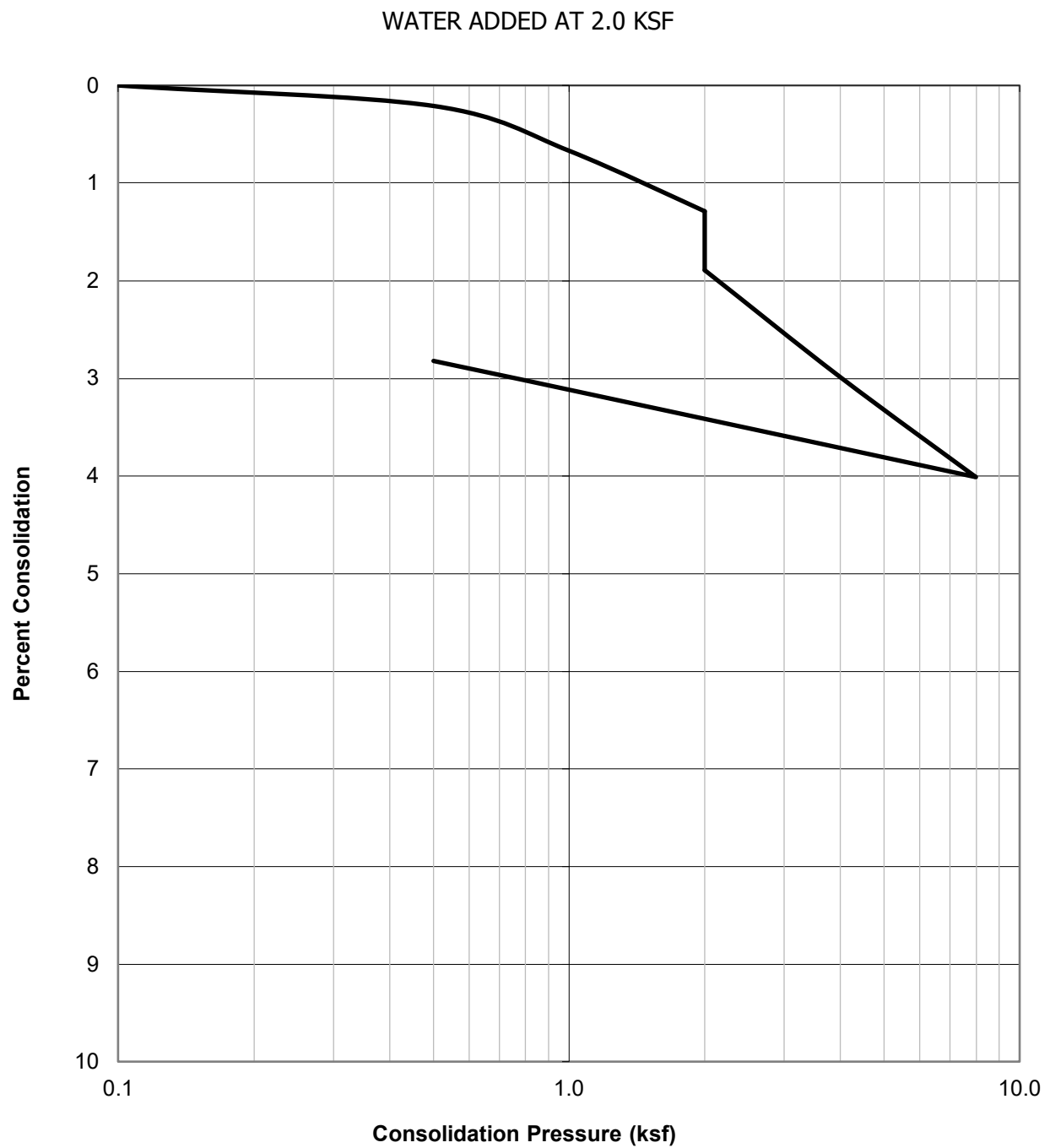
Checked by: KG

Project No.: T2875-22-05

LIBRARY LEARNING RESOURCE CENTER
MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

October 2025

Figure B8



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@15	Silty SAND (SM), yellowish brown	109.9	3.1	13.2



GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: KG

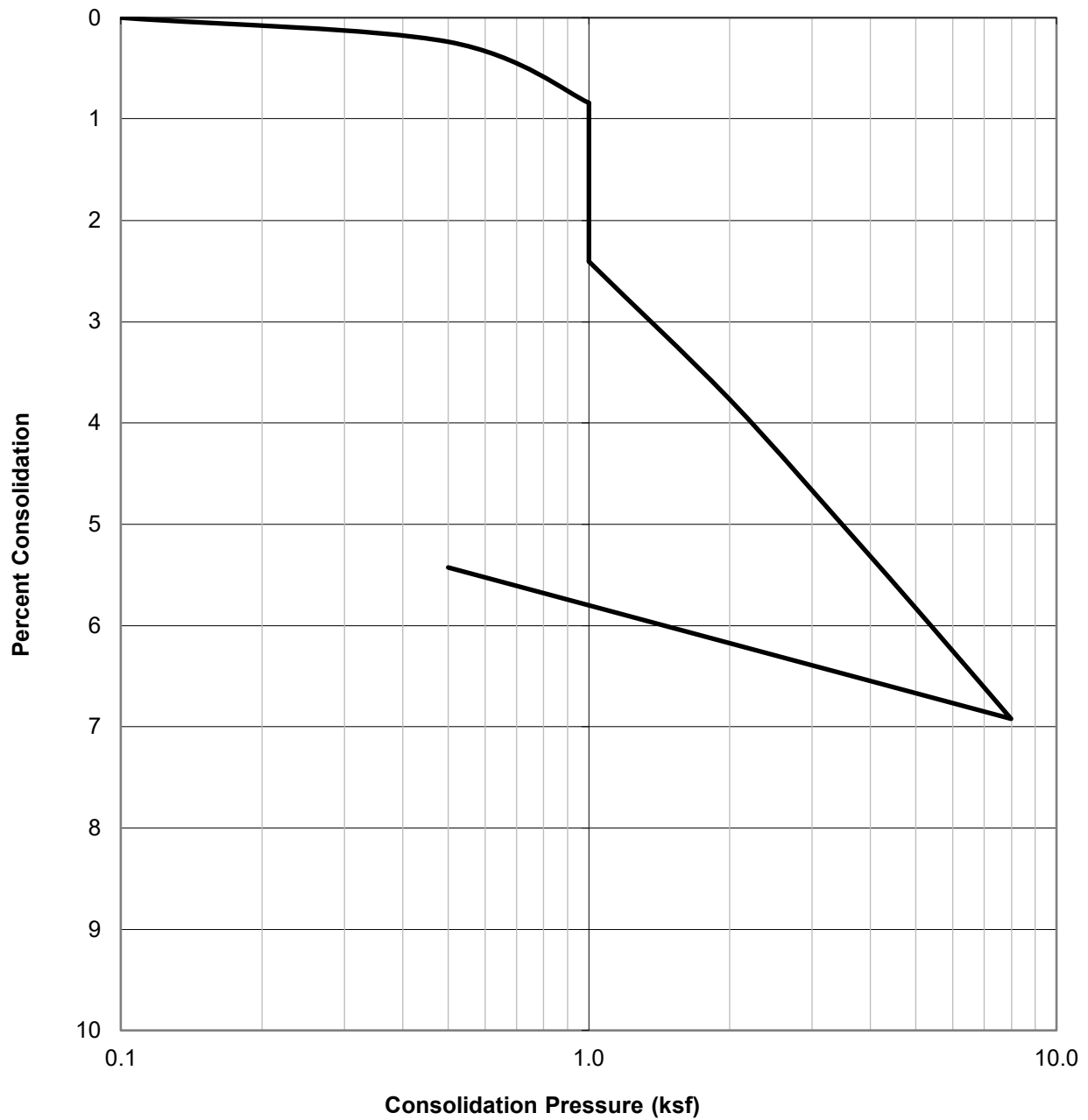
Project No.: T2875-22-05

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MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

October 2025

Figure B9

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@5	Silty SAND (SM), yellowish brown	99.8	3.8	15.8



GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: KG

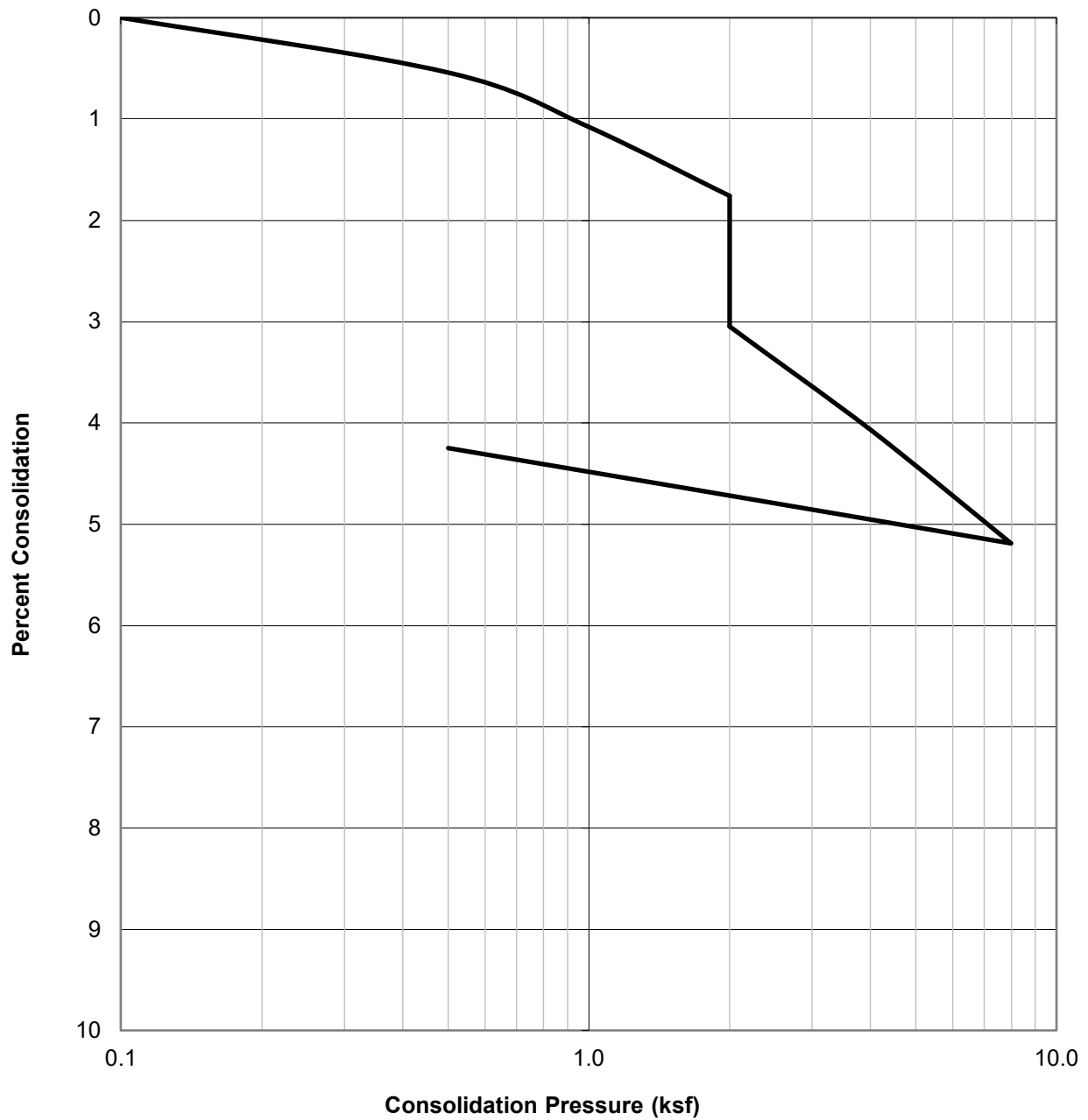
Project No.: T2875-22-05

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MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

October 2025

Figure B10

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@10	Silty SAND (SM), yellowish brown	106.6	3.1	14.7



GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

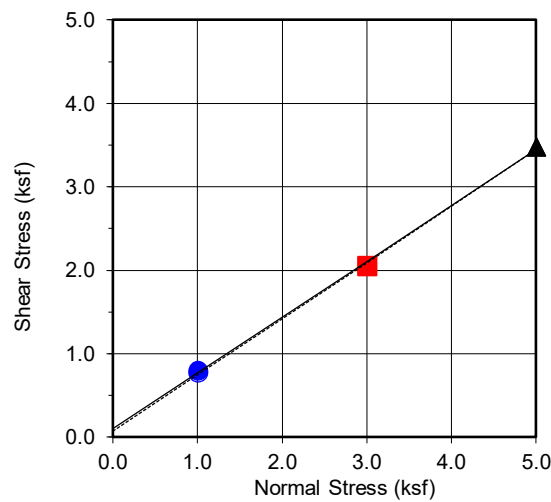
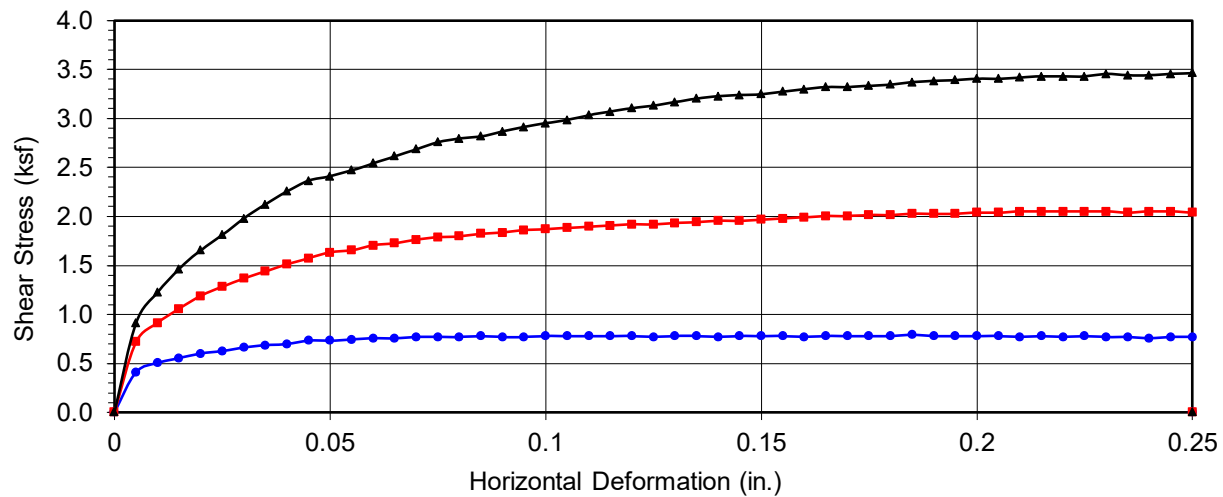
Checked by: KG

Project No.: T2875-22-05

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MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

October 2025

Figure B11



Boring No.	B2
Sample No.	B2@0-5'
Depth (ft)	0-5'
Sample Type:	Ring

<u>Soil Identification:</u>		
Silty SAND (SM), strong brown		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	97	34
Ultimate	67	34

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.79	■ 2.05	▲ 3.47
Shear Stress @ End of Test (ksf)	○ 0.77	□ 2.04	△ 3.47
Deformation Rate (in./min.)	0.005	0.005	0.005
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	6.4	6.4	6.4
Initial Dry Density (pcf)	121.4	121.5	121.6
Initial Degree of Saturation (%)	44.3	44.8	44.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	12.3	11.6	11.3



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

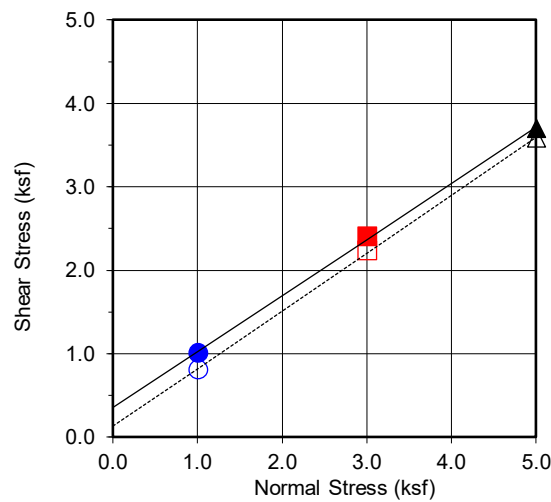
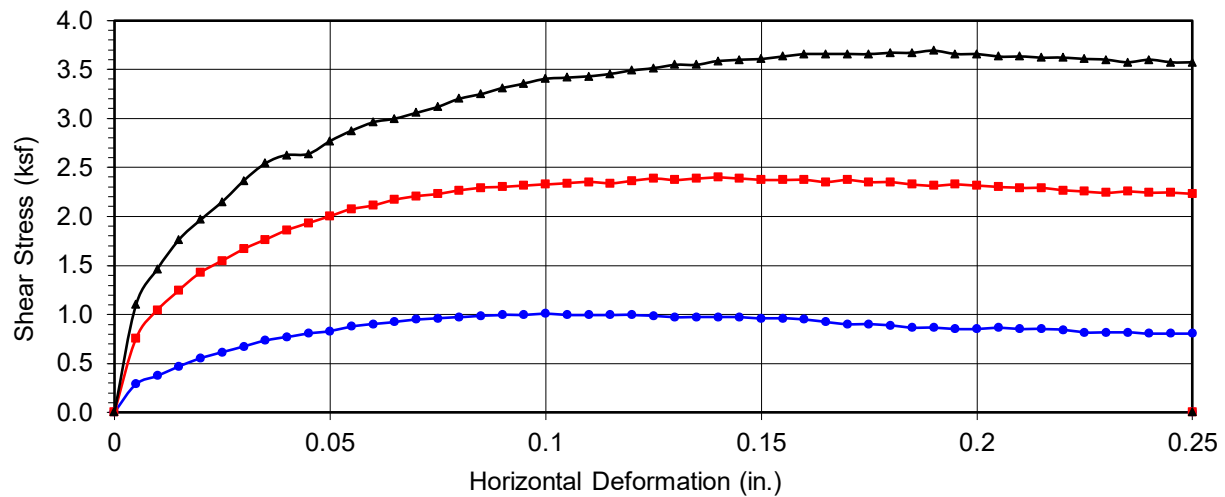
Checked by: KG

Project No.: T2875-22-05

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MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

October 2025

Figure B12



Boring No.	B6
Sample No.	B6@7.5'
Depth (ft)	7.5
Sample Type:	Ring

<u>Soil Identification:</u>		
Silty SAND (SM), brown		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	352	34
Ultimate	125	35

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.01	■ 2.40	▲ 3.70
Shear Stress @ End of Test (ksf)	○ 0.80	□ 2.23	△ 3.58
Deformation Rate (in./min.)	0.005	0.005	0.005
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.9	10.3	6.8
Initial Dry Density (pcf)	91.9	93.6	93.7
Initial Degree of Saturation (%)	28.8	34.8	22.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.2	17.0	15.6



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: KG

Project No.: T2875-22-05

LIBRARY LEARNING RESOURCE CENTER
MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

October 2025

Figure B13

SUMMARY OF LABORATORY R-VALUE TEST RESULTS

Sample No.	Soil Description	R-Value
B8@0-5'	Silty SAND (SM), dark brown	71



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R-VALUE TEST RESULTS

ASTM D-2844

Checked by: KG

Project No.: T2875-22-05

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MORENO VALLEY, CALIFORNIA

October 2025

Figure B14

APPENDIX

C

APPENDIX C

SEISMIC ANALYSES

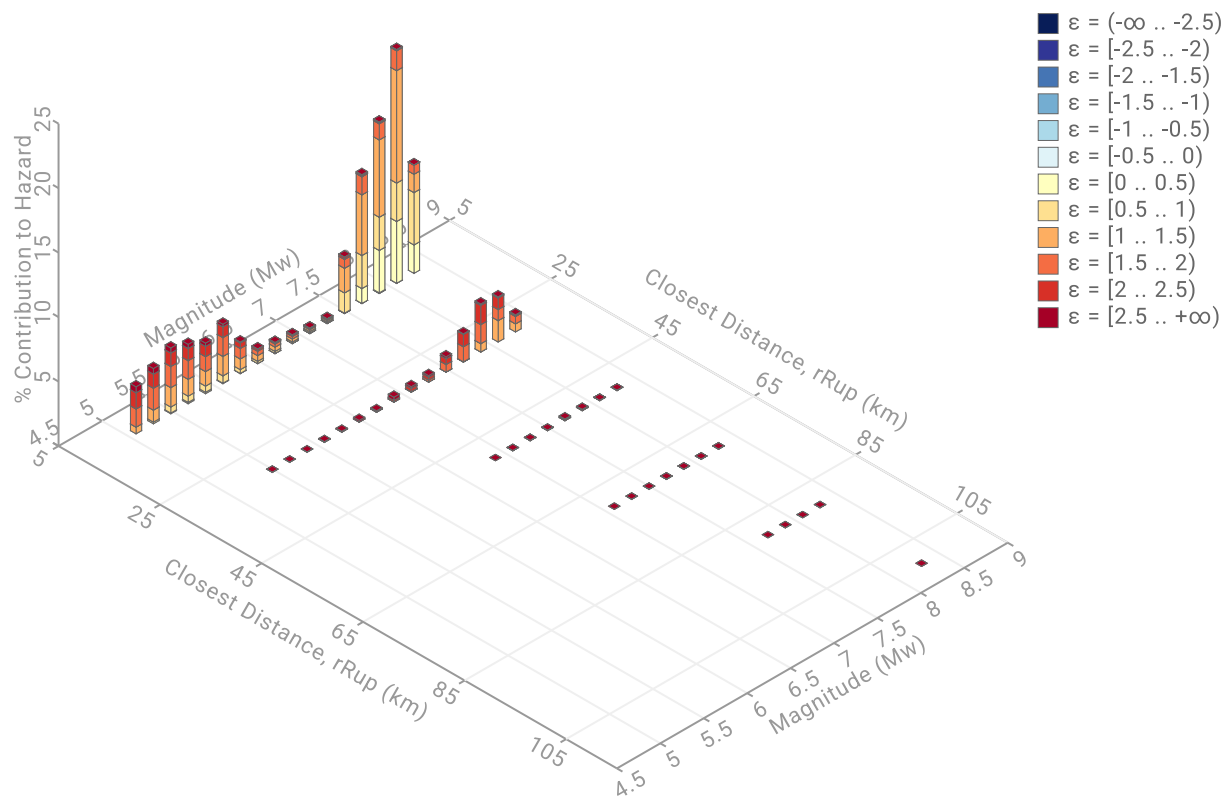
FOR

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MORENO VALLEY COLLEGE
MORENO VALLEY, CALIFORNIA

PROJECT NO. T2875-22-05

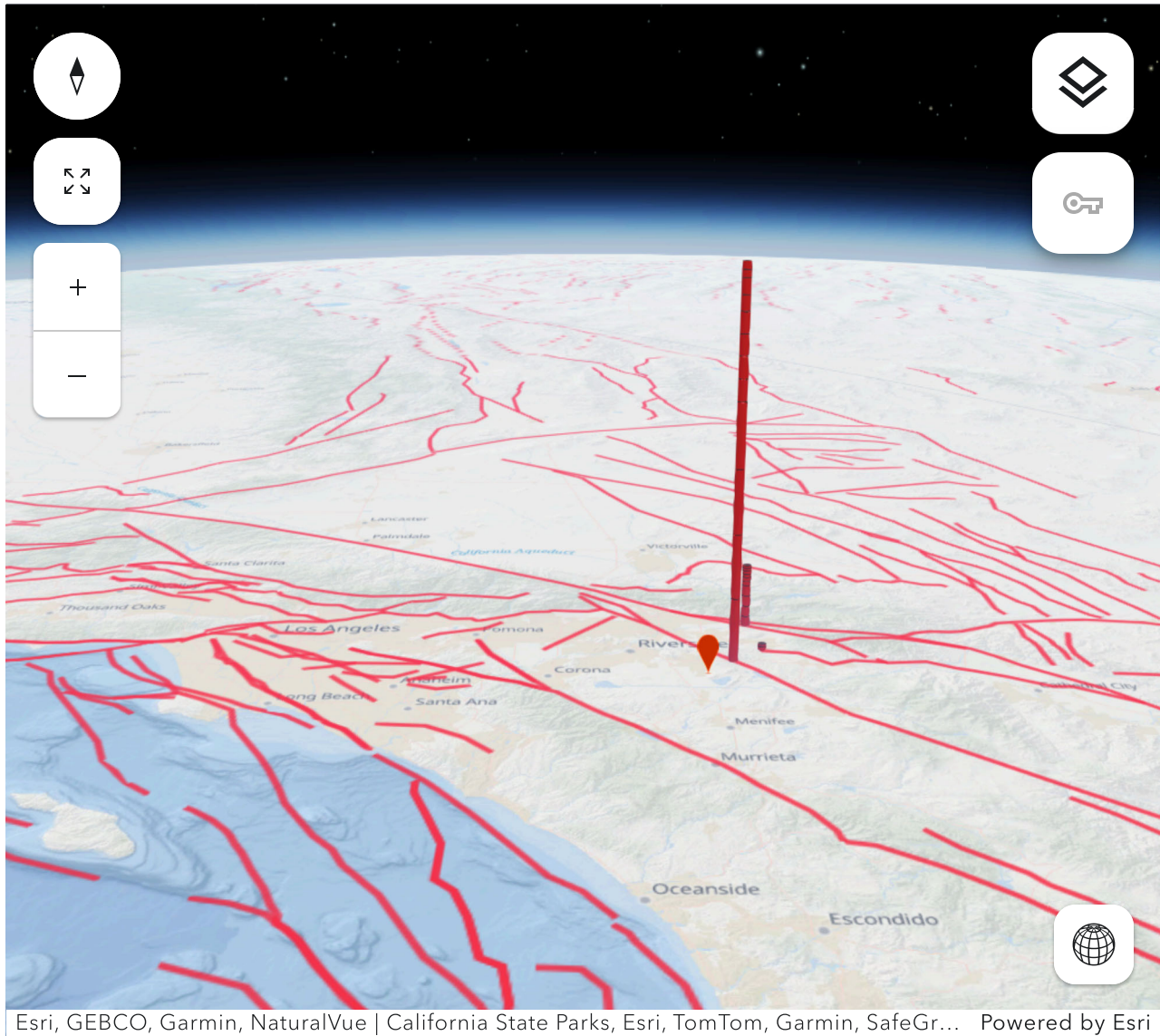
Disaggregation Report

Disaggregation





Geographical Disaggregation



Parameter Summary

Model: NSHM Conterminous U.S. 2018

Latitude: 33.886126 °

Longitude: -117.203037 °

Site Class: CD (Vs30 365)

Intensity Measure Type: PGA

Return Period: 2475 (2% in 50)

Component: Total

Disaggregation Summary: Total

Disaggregation targets

Return period : 2475 yrs
Exceedance rate : 4.040e-4 yr⁻¹
PGA ground motion : 8.551e-1 g

Totals

Binned : 100 %
Residual : 0 %
Trace : 0.09 %

Mode (largest m-r bin)

m : 8.1
r : 8.74 km
ε₀ : 0.9 σ
Contribution : 18.13 %

Discretization

r : min = 0.0, max = 1000.0, Δ = 20.0 km
m : min = 4.4, max = 9.4, Δ = 0.2
ε : min = -3.0, max = 3.0, Δ = 0.5 σ

Recovered targets

Return period : 2394.4895 yrs
Exceedance rate : 4.176e-4 yr⁻¹

Mean (over all sources)

m : 7.24
r : 11.46 km
ε₀ : 1.33 σ

Mode (largest m-r-ε₀ bin)

m : 8.1
r : 8.72 km
ε₀ : 1.11 σ
Contribution : 8.72 %

Epsilon keys

ε0 : [-∞ .. -2.5)
ε1 : [-2.5 .. -2.0)
ε2 : [-2.0 .. -1.5)
ε3 : [-1.5 .. -1.0)
ε4 : [-1.0 .. -0.5)
ε5 : [-0.5 .. 0.0)
ε6 : [0.0 .. 0.5)
ε7 : [0.5 .. 1.0)
ε8 : [1.0 .. 1.5)
ε9 : [1.5 .. 2.0)
ε10 : [2.0 .. 2.5)
ε11 : [2.5 .. +∞]



Disaggregation Contributions: Total

Source Set	↗	Source	Type	r	m	ϵ_0	lon	lat	az	%
UCERF3 Fault Model 3.1			FaultSystem							34.14
		San Jacinto (San Jacinto Valley) (2)		8.65	7.99	0.93	117.138°W	33.942°N	43.66	24.7
		San Andreas (San Bernardino, south) (2)		26.11	7.89	1.85	117.090°W	34.101°N	23.48	4.16
		San Gorgonio Pass (2)		17.67	7.67	1.63	117.064°W	33.995°N	46.66	1.06
UCERF3 Fault Model 3.2			FaultSystem							34.11
		San Jacinto (San Jacinto Valley) (2)		8.65	7.98	0.93	117.138°W	33.942°N	43.66	24.64
		San Andreas (San Bernardino, south) (2)		26.11	7.88	1.85	117.090°W	34.101°N	23.48	4.21
UCERF3 Fault Model 3.1 (opt)			Grid							15.88
UCERF3 Fault Model 3.2 (opt)			Grid							15.87



Application Metadata

Application: Disaggregation

URL: <https://earthquake.usgs.gov/nshmp/hazard/disagg>

Repository: nshmp-apps

Version: 20.8.0

URL: <https://code.usgs.gov/ghsc/nshmp/nshmp-apps>

Repository : nshmp-haz

Version: 2.6.18

URL: <https://code.usgs.gov/ghsc/nshmp/nshmp-haz>

Repository : nshmp-lib

Version: 1.7.14

URL: <https://code.usgs.gov/ghsc/nshmp/nshmp-lib>

Repository : nshm-conus

Version: 5.2.4

URL: <https://code.usgs.gov/ghsc/nshmp/nshms/nshm-conus.git>

October 19, 2025, 09:07 PM

Period	Site Class C	Site Class CD	Controlling
0.00	0.73	0.74	0.74
0.01	0.73	0.75	0.75
0.02	0.74	0.75	0.75
0.03	0.79	0.78	0.79
0.05	0.96	0.89	0.96
0.08	1.19	1.08	1.19
0.10	1.37	1.24	1.37
0.15	1.61	1.50	1.61
0.20	1.72	1.68	1.72
0.25	1.73	1.82	1.82
0.30	1.66	1.89	1.89
0.40	1.53	1.82	1.85
0.50	1.38	1.73	1.80
0.75	1.06	1.41	1.57
1.00	0.82	1.14	1.39
1.50	0.54	0.77	1.01
2.00	0.40	0.57	0.78
3.00	0.27	0.38	0.53
4.00	0.20	0.29	0.39
5.00	0.16	0.23	0.31
7.50	0.10	0.14	0.18
10.00	0.07	0.09	0.12

Values are from <https://ascehazardtool.org/>



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MULTI-PERIOD MCER SPECTRUM

Checked by: JTA

Project No.: T2875-22-05

MORENO VALLEY COLLEGE – LLRC
SW OF LA SALLE STREET & COLLEGE DRIVE
MORENO VALLEY, CALIFORNIA

Oct 25

Figure C1



Project Name : Proposed LLRC Moreno Valley College
Project No : T2875-22-05
Boring : B-1

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):	0.760
Magnitude Scaling Factor:	0.821
Historic High Groundwater:	11.0
Groundwater Depth During Exploration:	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

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. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	136.9	0	32	2.5	1	40	129	1.700	78.7	136.9	Infin.	1.000	0.494	--
2.0	136.9	0	32	2.5	1	40	129	1.700	78.7	136.9	Infin.	0.998	0.493	--
3.0	136.9	0	32	2.5	1	40	129	1.700	78.7	136.9	Infin.	0.996	0.492	--
4.0	136.9	0	32	2.5	1	40	129	1.700	78.7	136.9	Infin.	0.994	0.491	--
5.0	136.9	0	41	5.0	1	40	138	1.700	99.1	136.9	Infin.	0.991	0.490	--
6.0	136.9	0	41	5.0	1	40	138	1.665	97.2	136.9	Infin.	0.989	0.489	--
7.0	136.9	0	41	5.0	1	40	138	1.532	89.8	136.9	Infin.	0.987	0.487	--
8.0	124.5	0	32	7.5	1	40	112	1.430	65.8	124.5	Infin.	0.985	0.486	--
9.0	124.5	0	32	7.5	1	40	112	1.350	62.4	124.5	Infin.	0.982	0.485	--
10.0	124.5	0	21	10.0	1	30	88	1.282	39.6	124.5	Infin.	0.980	0.484	--
11.0	124.5	0	21	10.0	1	30	88	1.224	38.0	124.5	Infin.	0.978	0.483	--
12.0	124.5	1	21	10.0	1	30	88	1.173	36.6	62.1	Infin.	0.976	0.492	Non-Liq.
13.0	124.5	1	21	10.0	1	30	88	1.127	35.3	62.1	Infin.	0.974	0.510	Non-Liq.
14.0	124.5	1	21	10.0	1	30	88	1.087	34.2	62.1	Infin.	0.972	0.527	Non-Liq.
15.0	145.9	1	45	15.0	1	20	121	1.048	64.8	83.5	Infin.	0.970	0.541	Non-Liq.
16.0	145.9	1	45	15.0	1	20	121	1.010	62.6	83.5	Infin.	0.967	0.554	Non-Liq.
17.0	145.9	1	45	15.0	1	20	121	0.975	60.6	83.5	Infin.	0.965	0.565	Non-Liq.
18.0	145.9	1	45	15.0	1	20	121	0.945	58.8	83.5	Infin.	0.963	0.575	Non-Liq.
19.0	145.9	1	45	15.0	1	20	121	0.916	57.1	83.5	Infin.	0.961	0.585	Non-Liq.
20.0	145.9	1	25	20.0	1	20	86	0.891	35.9	83.5	Infin.	0.958	0.593	Non-Liq.
21.0	145.9	1	25	20.0	1	20	86	0.867	35.0	83.5	Infin.	0.956	0.600	Non-Liq.
22.0	145.9	1	25	20.0	1	20	86	0.845	34.2	83.5	Infin.	0.953	0.607	Non-Liq.
23.0	145.9	1	25	20.0	1	20	86	0.825	33.5	83.5	Infin.	0.950	0.612	Non-Liq.
24.0	145.9	1	25	20.0	1	20	86	0.806	32.8	83.5	Infin.	0.947	0.617	Non-Liq.
25.0	146.0	1	45	25.0	1	40	109	0.788	65.6	83.6	Infin.	0.944	0.622	Non-Liq.
26.0	146.0	1	45	25.0	1	40	109	0.772	64.3	83.6	Infin.	0.940	0.626	Non-Liq.
27.0	146.0	1	45	25.0	1	40	109	0.756	63.1	83.6	Infin.	0.936	0.629	Non-Liq.
28.0	146.0	1	45	25.0	1	40	109	0.741	62.0	83.6	Infin.	0.932	0.632	Non-Liq.
29.0	146.0	1	45	25.0	1	40	109	0.727	61.0	83.6	Infin.	0.928	0.634	Non-Liq.
30.0	146.0	1	51	30.0	1	40	112	0.714	70.6	83.6	Infin.	0.923	0.635	Non-Liq.
31.0	146.0	1	51	30.0	1	40	112	0.702	69.4	83.6	Infin.	0.918	0.636	Non-Liq.
32.0	146.0	1	51	30.0	1	40	112	0.690	68.4	83.6	Infin.	0.912	0.636	Non-Liq.
33.0	146.0	1	51	30.0	1	40	112	0.679	67.3	83.6	Infin.	0.907	0.636	Non-Liq.
34.0	146.0	1	51	30.0	1	40	112	0.668	66.3	83.6	Infin.	0.900	0.636	Non-Liq.
35.0	146.0	1	51	35.0	1	40	107	0.658	65.4	83.6	Infin.	0.894	0.635	Non-Liq.
36.0	121.7	1	35	35.0	1	60	88	0.649	45.5	59.3	Infin.	0.887	0.634	Non-Liq.
37.0	121.7	1	35	35.0	1	60	88	0.641	45.0	59.3	Infin.	0.880	0.633	Non-Liq.
38.0	121.7	1	35	35.0	1	60	88	0.634	44.5	59.3	Infin.	0.872	0.632	Non-Liq.
39.0	121.7	1	35	35.0	1	60	88	0.626	44.1	59.3	Infin.	0.864	0.630	Non-Liq.
40.0	150.0	1	100	40.0	1	100	146	0.619	116.3	87.6	Infin.	0.855	0.627	Non-Liq.
41.0	150.0	1	100	40.0	1	100	146	0.610	114.8	87.6	Infin.	0.846	0.622	Non-Liq.
42.0	150.0	1	100	40.0	1	100	146	0.602	113.4	87.6	Infin.	0.837	0.618	Non-Liq.
43.0	150.0	1	100	40.0	1	100	146	0.595	112.0	87.6	Infin.	0.828	0.613	Non-Liq.
44.0	150.0	1	100	40.0	1	100	146	0.587	110.7	87.6	Infin.	0.818	0.608	Non-Liq.
45.0	150.0	1	100	40.0	1	100	146	0.580	109.4	87.6	Infin.	0.808	0.602	Non-Liq.
46.0	150.0	1	100	40.0	1	100	146	0.573	108.2	87.6	Infin.	0.798	0.596	Non-Liq.
47.0	150.0	1	100	40.0	1	100	146	0.566	107.0	87.6	Infin.	0.788	0.590	Non-Liq.
48.0	150.0	1	100	40.0	1	100	146	0.560	105.8	87.6	Infin.	0.778	0.584	Non-Liq.
49.0	150.0	1	100	40.0	1	100	146	0.554	104.7	87.6	Infin.	0.768	0.578	Non-Liq.
50.0	150.0	1	100	50.0	1	100	135	0.548	103.6	87.6	Infin.	0.757	0.572	Non-Liq.

Figure C2



LIQUEFACTION SETTLEMENT ANALYSIS MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	8.10
PGA _M (g):	0.760
Magnitude Scaling Factor:	0.821
Historic High Groundwater:	11.0
Groundwater @ Exploration:	100.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ' _o	LIQUEFACTION SAFETY FACTOR	VOL. STRAIN [e _{1s}] (%)	EQ. SETTLE. Pe (in.)
1.0	32	136.9	0.034	0.034	129	79	0.494	--	0.00	0.00
2.0	32	136.9	0.103	0.103	129	79	0.494	--	0.00	0.00
3.0	32	136.9	0.171	0.171	129	79	0.494	--	0.00	0.00
4.0	32	136.9	0.240	0.240	129	79	0.494	--	0.00	0.00
5.0	41	136.9	0.308	0.308	138	99	0.494	--	0.00	0.00
6.0	41	136.9	0.377	0.377	138	97	0.494	--	0.00	0.00
7.0	41	136.9	0.445	0.445	138	90	0.494	--	0.00	0.00
8.0	32	124.5	0.510	0.510	112	66	0.494	--	0.00	0.00
9.0	32	124.5	0.573	0.573	112	62	0.494	--	0.00	0.00
10.0	21	124.5	0.635	0.635	88	40	0.494	--	0.00	0.00
11.0	21	124.5	0.697	0.697	88	38	0.494	--	0.00	0.00
12.0	21	124.5	0.759	0.744	88	37	0.504	Non-Liq.	0.00	0.00
13.0	21	124.5	0.821	0.775	88	35	0.524	Non-Liq.	0.00	0.00
14.0	21	124.5	0.884	0.806	88	34	0.542	Non-Liq.	0.00	0.00
15.0	45	145.9	0.951	0.842	121	65	0.558	Non-Liq.	0.00	0.00
16.0	45	145.9	1.024	0.884	121	63	0.572	Non-Liq.	0.00	0.00
17.0	45	145.9	1.097	0.926	121	61	0.586	Non-Liq.	0.00	0.00
18.0	45	145.9	1.170	0.967	121	59	0.598	Non-Liq.	0.00	0.00
19.0	45	145.9	1.243	1.009	121	57	0.609	Non-Liq.	0.00	0.00
20.0	25	145.9	1.316	1.051	86	36	0.619	Non-Liq.	0.00	0.00
21.0	25	145.9	1.389	1.093	86	35	0.628	Non-Liq.	0.00	0.00
22.0	25	145.9	1.462	1.134	86	34	0.637	Non-Liq.	0.00	0.00
23.0	25	145.9	1.535	1.176	86	33	0.645	Non-Liq.	0.00	0.00
24.0	25	145.9	1.608	1.218	86	33	0.652	Non-Liq.	0.00	0.00
25.0	45	146.0	1.681	1.260	109	66	0.659	Non-Liq.	0.00	0.00
26.0	45	146.0	1.754	1.302	109	64	0.666	Non-Liq.	0.00	0.00
27.0	45	146.0	1.827	1.343	109	63	0.672	Non-Liq.	0.00	0.00
28.0	45	146.0	1.900	1.385	109	62	0.678	Non-Liq.	0.00	0.00
29.0	45	146.0	1.973	1.427	109	61	0.683	Non-Liq.	0.00	0.00
30.0	51	146.0	2.046	1.469	112	71	0.688	Non-Liq.	0.00	0.00
31.0	51	146.0	2.119	1.511	112	69	0.693	Non-Liq.	0.00	0.00
32.0	51	146.0	2.192	1.552	112	68	0.698	Non-Liq.	0.00	0.00
33.0	51	146.0	2.265	1.594	112	67	0.702	Non-Liq.	0.00	0.00
34.0	51	146.0	2.338	1.636	112	66	0.706	Non-Liq.	0.00	0.00
35.0	51	146.0	2.411	1.678	107	65	0.710	Non-Liq.	0.00	0.00
36.0	35	121.7	2.478	1.713	88	45	0.714	Non-Liq.	0.00	0.00
37.0	35	121.7	2.539	1.743	88	45	0.719	Non-Liq.	0.00	0.00
38.0	35	121.7	2.600	1.773	88	45	0.724	Non-Liq.	0.00	0.00
39.0	35	121.7	2.660	1.802	88	44	0.729	Non-Liq.	0.00	0.00
40.0	100	150.0	2.728	1.839	146	116	0.733	Non-Liq.	0.00	0.00
41.0	100	150.0	2.803	1.883	146	115	0.735	Non-Liq.	0.00	0.00
42.0	100	150.0	2.878	1.927	146	113	0.738	Non-Liq.	0.00	0.00
43.0	100	150.0	2.953	1.971	146	112	0.740	Non-Liq.	0.00	0.00
44.0	100	150.0	3.028	2.014	146	111	0.743	Non-Liq.	0.00	0.00
45.0	100	150.0	3.103	2.058	146	109	0.745	Non-Liq.	0.00	0.00
46.0	100	150.0	3.178	2.102	146	108	0.747	Non-Liq.	0.00	0.00
47.0	100	150.0	3.253	2.146	146	107	0.749	Non-Liq.	0.00	0.00
48.0	100	150.0	3.328	2.190	146	106	0.751	Non-Liq.	0.00	0.00
49.0	100	150.0	3.403	2.233	146	105	0.753	Non-Liq.	0.00	0.00
50.0	100	150.0	3.478	2.277	135	104	0.755	Non-Liq.	0.00	0.00
TOTAL SETTLEMENT =									0.0 INCHES	

Figure C3

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

EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS

MAXIMUM CONSIDERED EARTHQUAKE

MCE EARTHQUAKE INFORMATION:

Earthquake Magnitude:	8.10
Peak Horiz. Acceleration (g):	0.760

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [C _{er}]	Relative Density [D _r] (%)	Correction Factor [C _n]	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[veff]*[Geff]	veff Shear Strain	[veff]*100%	Volumetric Strain M7.5 [E15] (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [E _c]	Estimated Settlement [SI] (inches)
1.0	1.0	0.5	136.9	0.03	0.02	0.017	32.13	1.25	128.7	1.7	78.7	1.0	290.1	5.77E-05	7.80E-05	0.008	1.51E-03	21.4	1.77E-03	0.00
2.0	1.0	1.5	136.9	0.10	0.07	0.051	32.13	1.25	128.7	1.7	78.7	1.0	502.5	9.80E-05	1.90E-04	0.019	3.67E-03	21.4	4.30E-03	0.00
3.0	1.0	2.5	136.9	0.17	0.11	0.084	32.13	1.25	128.7	1.7	78.7	1.0	648.8	1.24E-04	1.70E-04	0.017	3.28E-03	21.4	3.85E-03	0.00
4.0	1.0	3.5	136.9	0.24	0.16	0.118	32.13	1.25	128.7	1.7	78.7	1.0	767.6	1.44E-04	1.70E-04	0.017	3.28E-03	21.4	3.85E-03	0.00
5.0	1.0	4.5	136.9	0.31	0.21	0.152	41	1.25	137.6	1.7	99.1	1.0	939.7	1.48E-04	1.50E-04	0.015	2.20E-03	21.4	2.58E-03	0.00
6.0	1.0	5.5	136.9	0.38	0.25	0.185	41	1.25	137.6	1.7	97.2	1.0	1032.1	1.62E-04	1.50E-04	0.015	2.25E-03	21.4	2.64E-03	0.00
7.0	1.0	6.5	136.9	0.44	0.30	0.219	41	1.25	137.6	1.5	89.8	1.0	1092.9	1.77E-04	1.50E-04	0.015	2.47E-03	21.4	2.90E-03	0.00
8.0	1.0	7.5	124.5	0.51	0.34	0.251	31.5	1.25	112.3	1.4	65.8	1.0	1055.3	2.07E-04	4.50E-04	0.045	1.08E-02	21.4	1.26E-02	0.00
9.0	1.0	8.5	124.5	0.57	0.38	0.281	31.5	1.25	112.3	1.4	62.4	1.0	1098.2	2.19E-04	4.50E-04	0.045	1.15E-02	21.4	1.35E-02	0.00
10.0	1.0	9.5	124.5	0.63	0.43	0.311	21	1.25	88.1	1.3	39.6	1.0	993.2	2.63E-04	4.50E-04	0.045	1.99E-02	21.4	2.33E-02	0.01
11.0	1.0	10.5	124.5	0.70	0.47	0.341	21	1.25	88.1	1.2	38.0	1.0	1026.6	2.75E-04	4.50E-04	0.045	2.09E-02	21.4	2.45E-02	0.01
12.0	1.0	11.5	124.5	0.76	0.51	0.370	21	1.25	88.1	1.2	36.6	1.0	1058.2	2.85E-04	3.70E-04	0.037	1.79E-02	21.4	2.10E-02	0.00
13.0	1.0	12.5	124.5	0.82	0.55	0.400	21	1.25	88.1	1.1	35.3	1.0	1088.2	2.95E-04	3.70E-04	0.037	1.87E-02	21.4	2.19E-02	0.00
14.0	1.0	13.5	124.5	0.88	0.59	0.429	21	1.25	88.1	1.1	34.2	1.0	1116.8	3.03E-04	7.10E-04	0.071	3.72E-02	21.4	4.37E-02	0.00
15.0	1.0	14.5	145.9	0.95	0.64	0.460	44.73	1.25	121.1	1.0	64.8	1.0	1433.4	2.50E-04	3.70E-04	0.037	9.03E-03	21.4	1.06E-02	0.00
16.0	1.0	15.5	145.9	1.02	0.69	0.494	44.73	1.25	121.1	1.0	62.6	1.0	1470.2	2.58E-04	3.70E-04	0.037	9.41E-03	21.4	1.10E-02	0.00
17.0	1.0	16.5	145.9	1.10	0.74	0.528	44.73	1.25	121.1	1.0	60.6	1.0	1505.3	2.65E-04	3.70E-04	0.037	9.78E-03	21.4	1.15E-02	0.00
18.0	1.0	17.5	145.9	1.17	0.78	0.561	44.73	1.25	121.1	0.9	58.8	1.0	1539.0	2.72E-04	3.70E-04	0.037	1.01E-02	21.4	1.19E-02	0.00
19.0	1.0	18.5	145.9	1.24	0.83	0.594	44.73	1.25	121.1	0.9	57.1	1.0	1571.3	2.78E-04	3.70E-04	0.037	1.05E-02	21.4	1.23E-02	0.00
20.0	1.0	19.5	145.9	1.32	0.88	0.626	25	1.25	85.8	0.9	35.9	1.0	1384.5	3.29E-04	7.10E-04	0.071	3.52E-02	21.4	4.13E-02	0.00
21.0	1.0	20.5	145.9	1.39	0.93	0.659	25	1.25	85.8	0.9	35.0	1.0	1410.9	3.35E-04	7.10E-04	0.071	3.62E-02	21.4	4.25E-02	0.00
22.0	1.0	21.5	145.9	1.46	0.98	0.690	25	1.25	85.8	0.8	34.2	1.0	1436.4	3.41E-04	7.10E-04	0.071	3.73E-02	21.4	4.37E-02	0.00
23.0	1.0	22.5	145.9	1.54	1.03	0.722	25	1.25	85.8	0.8	33.5	0.9	1461.2	3.46E-04	5.20E-04	0.052	2.80E-02	21.4	3.28E-02	0.00
24.0	1.0	23.5	145.9	1.61	1.08	0.753	25	1.25	85.8	0.8	32.8	0.9	1485.3	3.51E-04	5.20E-04	0.052	2.87E-02	21.4	3.37E-02	0.00
25.0	1.0	24.5	146.0	1.68	1.13	0.783	44.73	1.25	109.4	0.8	65.6	0.9	1913.4	2.80E-04	3.00E-04	0.030	7.21E-03	21.4	8.45E-03	0.00
26.0	1.0	25.5	146.0	1.75	1.18	0.814	44.73	1.25	109.4	0.8	64.3	0.9	1941.7	2.83E-04	3.00E-04	0.030	7.38E-03	21.4	8.66E-03	0.00
27.0	1.0	26.5	146.0	1.83	1.22	0.843	44.73	1.25	109.4	0.8	63.1	0.9	1969.3	2.86E-04	3.00E-04	0.030	7.55E-03	21.4	8.85E-03	0.00
28.0	1.0	27.5	146.0	1.90	1.27	0.873	44.73	1.25	109.4	0.7	62.0	0.9	1996.3	2.89E-04	3.00E-04	0.030	7.72E-03	21.4	9.05E-03	0.00
29.0	1.0	28.5	146.0	1.97	1.32	0.901	44.73	1.25	109.4	0.7	61.0	0.9	2022.5	2.92E-04	3.00E-04	0.030	7.88E-03	21.4	9.24E-03	0.00
30.0	1.0	29.5	146.0	2.05	1.37	0.930	51	1.25	111.7	0.7	70.6	0.9	2162.8	2.79E-04	3.00E-04	0.030	6.61E-03	21.4	7.75E-03	0.00
31.0	1.0	30.5	146.0	2.12	1.42	0.958	51	1.25	111.7	0.7	69.4	0.9	2189.1	2.81E-04	3.00E-04	0.030	6.74E-03	21.4	7.90E-03	0.00
32.0	1.0	31.5	146.0	2.19	1.47	0.985	51	1.25	111.7	0.7	68.4	0.9	2214.9	2.83E-04	3.00E-04	0.030	6.86E-03	21.4	8.05E-03	0.00
33.0	1.0	32.5	146.0	2.26	1.52	1.012	51	1.25	111.7	0.7	67.3	0.9	2240.1	2.84E-04	3.00E-04	0.030	6.99E-03	21.4	8.20E-03	0.00
34.0	1.0	33.5	146.0	2.34	1.57	1.038	51	1.25	111.7	0.7	66.3	0.9	2264.8	2.86E-04	3.00E-04	0.030	7.12E-03	21.4	8.34E-03	0.00
35.0	1.0	34.5	146.0	2.41	1.62	1.064	51	1.25	107.3	0.7	65.4	0.9	2289.0	2.88E-04	3.00E-04	0.030	7.24E-03	21.4	8.49E-03	0.00
36.0	1.0	35.5	121.7	2.48	1.66	1.087	34.65	1.25	88.4	0.6	45.5	0.9	2055.9	3.24E-04	5.20E-04	0.052	1.94E-02	21.4	2.27E-02	0.00
37.0	1.0	36.5	121.7	2.54	1.70	1.107	34.65	1.25	88.4	0.6	45.0	0.9	2073.5	3.25E-04	5.20E-04	0.052	1.97E-02	21.4	2.30E-02	0.00
38.0	1.0	37.5	121.7	2.60	1.74	1.126	34.65	1.25	88.4	0.6	44.5	0.9	2090.9	3.25E-04	5.20E-04	0.052	1.99E-02	21.4	2.33E-02	0.00
39.0	1.0	38.5	121.7	2.66	1.78	1.145	34.65	1.25	88.4	0.6	44.1	0.9	2108.0	3.25E-04	5.20E-04	0.052	2.01E-02	21.4	2.36E-02	0.00
40.0	1.0	39.5	150.0	2.73	1.83	1.166	100	1.25	145.9	0.6	116.3	0.9	2950.4	2.35E-04	3.00E-04	0.030	3.63E-03	21.4	4.25E-03	0.00
41.0	1.0	40.5	150.0	2.80	1.88	1.190	100	1.25	145.9	0.6	114.8	0.8	2977.7	2.36E-04	3.00E-04	0.030	3.68E-03	21.4	4.32E-03	0.00
42.0	1.0	41.5	150.0	2.88	1.93	1.213	100	1.25	145.9	0.6	113.4	0.8	3004.6	2.36E-04	3.00E-04	0.030	3.74E-03	21.4	4.38E-03	0.00
43.0	1.0	42.5	150.0	2.95	1.98	1.236	100	1.25	145.9	0.6	112.0	0.8	3031.1	2.37E-04	3.00E-04	0.030	3.79E-03	21.4	4.45E-03	0.00
44.0	1.0	43.5	150.0	3.03	2.03	1.258	100	1.25	145.9	0.6	110.7	0.8	3057.1	2.38E-04	1.00E-02	1.000	1.28E-01	21.4	1.50E-01	0.00
45.0	1.0	44.5	150.0	3.10	2.08	1.280	100	1.25	145.9	0.6	109.4	0.8	3082.7	2.38E-04	1.00E-02	1.000	1.30E-01	21.4	1.53E-01	0.00
46.0	1.0	45.5	150.0	3.18	2.13	1.301	100	1.25	145.9	0.6	108.2	0.8	3107.9	2.39E-04	1.00E-02	1.000	1.32E-01	21.4	1.55E-01	0.00
47.0	1.0	46.5	150.0	3.25	2.18	1.321	100	1.25	145.9	0.6	107.0	0.8	3132.7	2.39E-04	1.00E-02	1.000	1.34E-01	21.4	1.57E-01	0.00
48.0	1.0	47.5	150.0	3.33	2.23	1.342	100	1.25	145.9	0.6	105.8	0.8	3157.2	2.39E-04	1.00E-02	1.000	1.35E-01	21.4	1.59E-01	0.00
49.0	1.0	48.5	150.0	3.40	2.28	1.361	100	1.25	145.9	0.6	104.7	0.8	3181.3	2.40E-04	1.00E-02	1.000	1.37E-01	21.4	1.61E-01	0.00
50.0	1.0	49.5	150.0	3.48	2.33	1.380	100	1.25	135.3	0.5	103.6	0.8	3205.1	2.40E-04	1.00E-02	1.000	1.39E-01	21.4	1.63E-01	0.00

TOTAL SETTLEMENT = **0.02**