

**ENGINEERING GEOLOGY AND
GEOTECHNICAL ENGINEERING REPORT**
FOR PROPOSED IMPROVEMENTS TO THE ATHLETIC FIELD
VENTURA HIGH SCHOOL
VENTURA, CALIFORNIA

PROJECT NO.: 305811-006
MAY 22, 2025
(REVISED: AUGUST 15, 2025)

PREPARED FOR
VENTURA UNIFIED SCHOOL DISTRICT
ATTENTION: RYAN HUGHES

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May 22, 2025
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Project No.: 305811-006
Report No.: 25-5-11

Attention: Ryan Hughes
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Project: Proposed Improvements to the Existing Athletic Stadium
Ventura High School
2 North Catalina Street
Ventura, California
Subject: Engineering Geology and Geotechnical Engineering Report

As authorized, Earth Systems Pacific (Earth Systems) has prepared this Engineering Geology and Geotechnical Engineering Report for proposed improvements to the existing athletic stadium on the Ventura High School campus located at 2 North Catalina Street in Ventura, California. The accompanying Engineering Geology and Geotechnical Engineering Report presents the results of reviewing data provided in previous regional geologic reports for the area and site-specific fault study reports, our subsurface exploration and laboratory testing programs, and our conclusions and recommendations pertaining to geotechnical aspects of project design. This report completes Phase 1 of the scope of services described within our Proposal No. VEN-25-02-003 dated February 18, 2025, and authorized by Ahsan Mirza on February 24, 2025.

We appreciate the opportunity to be of service to you on this project. Please call if you have any questions, or if we can be of further service.

Respectfully submitted,
EARTH SYSTEMS PACIFIC

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INTRODUCTION

This report presents results of an Engineering Geology and Geotechnical Engineering study for proposed improvements to the existing athletic stadium on the Ventura High School campus located at 2 North Catalina Street in Ventura, California (see Vicinity Map in Appendix A). The geographic coordinates of the site are 34.2805° North Latitude and 119.2644° West Longitude. Earth Systems understands that the proposed improvements will include a new ramp for field access on the south side of the track, three new stairs at the north bleachers for egress, new stairs for field access on the south side of the track, a new ramp system north of the track and west of the north bleachers, a new track surface, and a new synthetic turf athletic surface.

The project site is the existing athletic stadium which includes a synthetic turf covered football field surrounded by a rubberized running track, bleachers, and fieldhouse buildings. The larger concrete bleachers, located on the north side of the stadium, are constructed on the ascending slope. The smaller metallic bleachers, located on the south side of the stadium, are constructed on a paved level surface at the top of a slope that descends to the parking lot. The area surrounding these includes landscaping (planters, lawns, and trees) and hardscaping (ramps, walkways, and driveways). Access to the athletic stadium is available from Jardin Street to the west, providing direct entry, and from Poli Street to the south via the southern parking area.

The site topography includes several graded slopes of varying heights and orientations. A descending slope approximately 10 feet in height is located southwest of the stadium, extending toward the existing fieldhouse, with an estimated gradient of 10H:1V (horizontal to vertical). An ascending slope approximately 30 feet in height is situated to the northeast of the stadium, with a gradient of approximately 4H:1V. Another ascending slope, approximately 25 feet in height and currently occupied by a ramp system, is located to the northwest of the stadium, also with a gradient of approximately 4H:1V.

Grading for the proposed project is expected to include cut and fill operations to achieve the required design elevations, excavation for foundations and utility installations, placement and compaction of new fill to support the proposed improvements, and slope stabilization and retaining structures, as necessary to maintain site stability. Additional grading details can be provided based on the final project design.

PURPOSE AND SCOPE OF WORK

The purpose of the geologic/geotechnical study that led to this report was to identify and discuss potential geohazards, analyze soil conditions at the project site, and provide geologic/geotechnical recommendations for design and construction. The soil conditions include surface and subsurface soil types, soil expansion potential, soil strength, bearing capacity, and the presence or absence of subsurface water. The scope of work included:

- Reviewing data provided in previous regional geologic reports for the area and site-specific fault study reports.
- Performing a reconnaissance of the project site.
- Hand-augering, sampling, and logging six (6) borings on the project site to study soil and groundwater conditions. Two of the borings were used for infiltration testing.
- Advancing one (1) cone penetrometer test (CPT) sounding to evaluate the soil properties and conditions.
- Coring of the existing concrete bleachers on the north side of the stadium for determination of the unconfined compressive strength. A total of three cores were taken at the designated locations.
- Laboratory testing of soil samples obtained during the subsurface exploration to determine their physical and engineering properties.
- Performing unconfined compressive strength tests on the concrete cores taken from the north bleachers.
- Performing infiltration testing in the area designated for testing to be performed.
- Consulting with owner representatives and design professionals.
- Analyzing the geotechnical data obtained.
- Preparing this report.

Contained in this report are:

- Discussion on the results of reviewing previous studies, regional geologic reports, and site-specific fault studies to evaluate potential geologic hazards affecting the proposed project.
- Descriptions and results of field and laboratory tests that were performed.
- Conclusions and recommendations pertaining to site grading and structural design.

SOIL AND GROUNDWATER CONDITIONS

Evaluation of the subsurface conditions based on the borings indicates that much of the project site in the running track area is covered by a layer of artificial fill. The artificial fill in our borings generally consisted of silty sand (SM) and/or sandy silt (ML) with varying amounts of gravel to sandy gravel with silt (GM). The thickness of this artificial fill varies across the track and football field area, ranging from approximately 4 feet in boring HA-2 on the north side of the track and football field area to about 11 feet in boring HA-3 on the south side. Earth Systems is not aware of any documentation about the placement and testing of this artificial fill. Hence, for the purposes of this report, the artificial fill is considered to be non-engineered and uncertified. Beneath the artificial fill, native alluvial deposits consisting of layers of silty sand (SM) and sandy silt (ML) were encountered at the in-situ borings to the maximum depths explored.

Evaluation of the subsurface conditions based on the interpretation of CPT-1 results indicates that the upper 11 feet of the soil profile predominantly consists of layers of silty sand (SM) and/or sandy silt (ML) with interbedded layers clayey soils. This composition suggests the likely presence of artificial fill. Beneath the artificial fill, native alluvial soils were encountered and extended to the maximum explored depth of approximately 80 feet. Based on the results, the alluvium primarily consisted of interbedded layers of sand and silty sand (SP-SM), silty sand (SM), and sandy silt (ML) in a loose to medium dense state. Trace amounts of hard clay were identified in CPT-1 between the depths of approximately 42 to 44 feet, 60.5 to 65.5 feet, and 76 to 78.5 feet below the existing ground surface. Also, a noticeably dense sandy layer was observed between the depths of 67 and 71 feet.

Due to the numerous locations of the proposed improvements, corrosion testing was not performed. Testing performed on the artificial fill encountered at the location of the existing fieldhouse indicates that the artificial fill soils had a sulfate content (35 mg/Kg) that falls in the "S0" exposure class of Table 19.3.1.1 of ACI 318-14. Therefore, special concrete designs will not be necessary for the measured sulfate contents according to Table 19.3.2.1 of ACI 318-14. The measured resistivity on the sample of artificial fill encountered at the location of the existing fieldhouse (7,200 ohms-cm) indicates that that the artificial fill soils are "Moderately Corrosive" to ferrous metal (i.e., cast iron, etc.) pipes. Based on visual and tactile classifications, we anticipate that the artificial soils encountered at the locations of the proposed improvements are similar to those encountered at the location of the existing fieldhouse. Therefore, we anticipate

the corrosion characteristics to be similar also. However, we recommend that samples be collected once grading operations commence and the corrosion tests be performed.

When our field exploration was conducted in March 2025, groundwater was not encountered in any of our onsite borings to the maximum depth explored of about 14 feet below the existing ground surface. According to the Seismic Hazard Zones Report for the Ventura 7.5-Minute Quadrangle, Ventura County, California (CGS, 2003), and based on engineering judgment, the depth of historical high groundwater at the project site is estimated to be about 37 feet (See the Historical High Groundwater Map in Appendix A). It should be noted that fluctuations in groundwater levels may occur because of variations in rainfall regional climate, and other factors.

GEOLOGY

The Ventura High School campus lies within the Ventura Basin, which in turn lies within the western Transverse Ranges. The Ventura Basin and the Transverse Ranges are characterized by ongoing tectonic activity. In the Ventura Basin, Tertiary and Quaternary sediments have been uplifted, folded, and faulted along predominant east-west structural trends for about the last 200,000 years (Rockwell, et al. 1988).

The Ventura High School campus is located approximately halfway between the fold axes of the east-west trending Ventura Anticline and Ventura Syncline (CDMG, 1973). The project site is on the south limb of the Ventura Anticline. During this period of continuous uplift, wide fluctuations in the elevation of sea level have led to sporadic periods of marine and non-marine terrace deposit formation and subsequent dissection (CDMG, 1973). In general, the older deposits have been subjected to a higher degree of warping than the younger deposits and are consequently tilted more steeply southward.

There are several faults located within the Ventura Basin. The Ventura fault is an east-west trending north-dipping reverse fault expressed by a prominent linear bench running from the east end of Ventura to the west end. This fault is best identified from air photo interpretation and oil field records. This fault has been classified as "active" by the California Geological Survey (C.D.M.G. 1972, Revised 2018). The Ventura High School campus is located within the designated "Fault Rupture Hazard Zone" for the fault. Tensional features such as soil cracks and micro faults have been found in other locales above the hanging wall (north of the fault) while the materials on the south side of the fault are less affected (Sarna-Wojcicki, et al. 1976).

The track and field area of the Ventura High School campus is positioned near the toe of the Ventura Hills where relatively youthful, nearly horizontal, alluvial sediments overlie gently south dipping older terrace deposits and steeper south dipping soft bedrock units of the San Pedro Formation (USGS, 1987, CDMG, 1973, Sarna-Wojcicki, et al. 1976). Other mappers refer to the local bedrock as Saugus Formation (Dibblee, 1988, Rockwell, 1988).

GEOLOGIC HAZARDS

Geologic hazards that may impact a site include seismic shaking, fault rupture, landsliding, rock fall, liquefaction, seismic induced settlement, flooding, and expansive soils.

A. Seismic Shaking

1. The Ventura High School campus is located in an active seismic region where large numbers of earthquakes are recorded each year. Historically, major earthquakes (i.e., those with Richter magnitudes greater than 7.0) felt in the vicinity of subject site have originated from faults outside the area. These include the December 21, 1812 "Santa Barbara Region" earthquake, that was presumably centered in the Santa Barbara Channel, the 1857 Fort Tejon earthquake, the 1872 Owens Valley earthquake, and the 1952 Arvin-Tehachapi earthquake.
2. It is assumed that the 2022 CBC and ASCE 7-16 guidelines will apply for the seismic design parameters. The 2022 CBC includes several seismic design parameters that are influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The seismic design parameters presented herein were determined by the U.S. Seismic Design Maps "risk-targeted" calculator on the USGS website for the jobsite coordinates (34.2805° North Latitude and -119.2644 West Longitude). The calculator adjusts for Soil Site Class D (for stiff soils) and Occupancy (Risk) Category II. (A listing of the calculated 2022 CBC and ASCE 7-16 Seismic Parameters is presented below and again in Appendix C.)

Summary of Seismic Parameters – 2022 CBC “General Procedure”

Site Class (ASCE 7-16)	D
Occupancy (Risk) Category	II
Seismic Design Category	D

Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	1.990 g
Spectral Response Acceleration at 1 sec. – S_1	0.749 g
Site Coefficient – F_a	1.00
Site Coefficient – F_v	1.70
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	1.990 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.273 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.327 g
One Second Spectral Response – S_{D1}	0.849 g
Site Modified Peak Ground Acceleration - PGA_M	0.961 g
Values appropriate for a 2% probability of exceedance in 50 years	

The seismic factor S_1 is greater than 0.2g and the Site Class is D. Therefore, a site-specific ground motion hazard analysis may be required. Per ASCE 7-16, Table 11.4-2, 1.7 has been used for F_v in the summary above since S_1 is greater than 0.6. The project structural engineer shall determine if an exemption can be applied in accordance with ASCE 7-16 Section 11.4.8. If it is determined that a site-specific ground motion hazard analysis is necessary, Earth Systems can provide that analysis under a separate cover.

The Fault Parameters table in Appendix C lists the significant "active" and "potentially active" faults within an approximate 36-mile radius of the project site. The distance between the project site and the nearest portion of each fault is shown as well as the respective estimated maximum earthquake magnitudes based on the USGS Earthquake Scenario Map (BSSC, 2014).

3. Southern Ventura County has been mapped by the California Division of Mines and Geology to delineate areas of varying predicted seismic response. The alluvial deposits that underlie the subject area are mapped as having a probable maximum intensity of earthquake response of approximately IX on the Modified Mercalli Scale. Historically, the highest observed intensity of ground response has been VII in the Ventura area (C.D.M.G., 1975).
4. The San Andreas is the dominant active fault in California. The fault extends from the Gulf of California to Cape Mendocino in northern California. That portion of the zone

extending southward from Parkfield, California is estimated to have been active for the last 12 million years. As much as 190 miles of right lateral displacement has occurred across the zone (Crowell, 1975). This displacement includes offsets on the actual San Andreas Fault and related faults that include the Imperial, Banning, Mission Creek, and San Jacinto faults.

5. Historically, the San Andreas Fault is responsible for two of the three "great" earthquakes experienced in California. ("Great" earthquakes are defined as having Richter magnitudes that are equal to or greater than 8.0.) These are the 1857 Fort Tejon and 1906 San Francisco earthquakes. Each event is credited with approximately 200 miles of surface rupture and horizontal displacements of up to 30 feet. Ground shaking was very intense and damage to man-made structures very widespread. The 1857 rupture extended along the San Andreas Fault from near Bakersfield to Cajon Pass and was felt throughout most of California. Horizontal displacements of 10 to 13 feet were observed along the fault in the Palmdale area.
6. Recurrence intervals for major earthquakes in southern California are best documented for the San Andreas Fault. It is estimated that a major earthquake has occurred along the southern portion of the San Andreas Fault every 100 to 200 years (Sieh, 1978). The average recurrence interval is estimated to be 140 years. The last major earthquake on the San Andreas Fault in the southern California area occurred in 1857; therefore, the occurrence of a major event in the same general area is considered likely within the estimated lifetime of any new construction.
7. On December 21, 1812, an estimated 7.0 Richter magnitude event occurred in an area believed to be offshore in the western part of the Santa Barbara Channel. This earthquake caused considerable shaking in the area of the proposed project.
8. On March 26, 1872, the greatest recorded earthquake in the western United States, excluding Alaska, occurred along the Owens Valley Fault near Lone Pine. The earthquake is estimated to have had a Richter magnitude of 8.25 and significantly shook most of California.
9. On July 21, 1952, the Arvin-Tehachapi earthquake occurred on the White Wolf Fault. The earthquake registered 7.7 on the Richter Scale and was felt throughout southern California.

B. Fault Rupture

Surficial displacement along a fault trace is known as fault rupture. Fault rupture typically occurs along previously existing fault traces. As mentioned in the "Structure" section above, the Ventura Fault traverses the campus and the project site is located within the designated Fault Rupture Hazard Zone

During the end of June and early part of July, 2001, an exploratory trench that was about 200 feet long and up to 20 feet deep was excavated in the baseball fields to the west of the track and field. The attached Site Plan / Geologic Map shows a mapping of the surficial geologic units, the location of the fault exploration trench, and the surface projection of the most significant shear. Because the shear dips to the north at about 26 degrees, the eastward portion of the surface intercept migrates to the south with the rise in elevation. In the southern portions of the trench, the younger alluvial fan deposits appear to be undeformed.

Building/structures (if any) should be set back a minimum of 50 feet from the surface intercept of the fault, as shown on the Site Plan and on the surveyed location map included in Appendix A. If the proposed improvements on the north side of the stadium are sited as recommended herein, it is the opinion of this firm that the potential hazard posed by fault rupture is low. However, based on the surface projection of the fault trace across the athletic stadium, the proposed new ramp and stairs on the south side of the stadium for field access may fall within 50 feet of the surface projection of the fault trace.

C. Landsliding

No landslides were observed to be on the subject site. Two of the regional geologic mappings (Sarna Wojcicki et al. 1978 and USGS, 1987) show a large, queried landslide upslope from, and encroaching onto the northern portion of the campus. To our knowledge, if this feature is a landslide, it is inactive. Other geologic mappings show the area to be undisturbed bedrock. (CDMG 1973, Dibblee, 1988).

As part of another Ventura High School geotechnical study in 2001 for a classroom building on campus, the City of Ventura was contacted. The Engineering Department had no knowledge of any regional instability. The points where the queried feature cross streets and other rigid improvements were observed. No indications of movement were detected. The subsurface relationships exposed by the trenching for this study showed

no indications of landsliding. This indicates that if there are landslide units under the younger alluvial fan units exposed in the trenching, last movement would have been more than about 36,000 years ago, the estimated age of the oldest sediments exposed in the trenching. Another possibility is the toe of the slide could be to the north of the area that was explored.

D. Rock Fall

Rock fall occurs when loose rocks on slopes become dislodged by erosion or by seismic events and mobilized. When these rocks can become a hazard to structures or to life downslope.

No loose rocks were observed on the upslope of the proposed improvements. As a result, it appears that the hazard posed by rock fall is low.

E. Liquefaction and Lateral Spreading

Earthquake-induced cyclic loading can be the cause of several significant phenomena, including liquefaction in fine sands and silty sands. Liquefaction results in a loss of soil strength and can cause structures to settle and, in extreme cases, to experience bearing failure. Cyclic softening in clays during earthquakes has resulted in buildings experiencing foundation failure and ground surface deformation similar to that resultant from liquefaction. If liquefaction or cyclic softening occurs beneath sloping ground, a phenomenon known as lateral spreading can occur. Liquefaction and cyclic softening typically are limited to the upper 50 feet of the subsurface soils.

There are a number of conditions that need to be satisfied for liquefaction or cyclic softening to occur. Of primary importance is that groundwater, perched or otherwise, usually must be within the upper 50 feet of soils. For liquefaction to occur, a potentially liquefiable soil must be saturated and subjected to rapid cyclic loading that is sufficiently intense to overcome a soil's internal resistance to liquefaction. Fine sands and silty sands that are poorly-graded and lie below the groundwater table are the soils most susceptible to liquefaction.

Using CPT (Cone Penetration Test) data, soils prone to liquefaction can be identified using the material index (I_c). The soil behavior index (I_c) is calculated based on CPT parameters such as tip resistance (q_c), sleeve friction (f_s), and pore pressure (u_2). This index helps

distinguish between cohesive (clay-like) and granular (sand-like) soils, which is crucial for liquefaction susceptibility assessments. The material index (I_c) is derived directly from raw CPT data using the following equation:

$$I_c = \sqrt{(3.47 - \log Q_t)^2 + (1.22 + \log F_r)^2}$$

Where:

Q_t = normalized cone tip resistance

F_r = normalized friction ratio

Soils that have I_c values greater than 2.6 (clayey-like soils), sufficiently dense soils, soils that have plasticity indices greater than 7, and/or soils located above the groundwater table are not generally susceptible to liquefaction.

An examination of the conditions existing at the project site, in relation to the criteria listed above, indicates the following:

1. The subject site is located within one of the Liquefaction Hazard Zones delineated by the State of California (CGS, 2003b).
2. Groundwater was not encountered in any of the exploratory borings drilled for this study. According to the Seismic Hazard Zones Report for the Ventura 7.5-Minute Quadrangle, Ventura County, California (CGS, 2003), the depth of historical high groundwater at the project site is assumed to be approximately 37 feet. Therefore, a groundwater depth of 37 feet was used in our analysis.
3. Interpretations of the data collected during site investigations and laboratory testing show that the soil profile within the project site consist primarily of interbedded layers of sands and silts.
4. The interpretation of CPT-1 results indicates that the sand and silt layers within the explored depth are predominantly in a loose to medium dense state.

Based on the above, cyclic mobility analyses were performed with Earth Systems' proprietary spreadsheets to assess the liquefaction potential of soil layers within the project site. The analysis was performed in general accordance with an updated version of the NCEER (1998) method which is known as the Robertson (2009) method. Also,

liquefaction-induced settlements were estimated using the approach developed by Zhang et al. (2002).

The analysis assumed a moment magnitude 7.4 design earthquake and a peak ground acceleration (PGA) of 0.961g, as discussed in the Seismicity and Shaking section of this report. In the liquefaction analysis, the peak ground acceleration is used for soil layers below the groundwater level to evaluate the liquefaction potential and liquefaction-induced settlement, whereas two-thirds of the peak ground acceleration is applied to soil layers above the groundwater level to evaluate the seismic-induced settlement of dry sands. Assuming a historical high groundwater depth of 37 feet, two sets of CPT-Based Liquefaction Analyses were performed. Because liquefaction is typically limited to the upper 50 feet of the subsurface soils, the liquefaction potential was evaluated to a depth of 50 feet.

Our analysis assessed the liquefaction potential and potential seismic-induced settlement within the upper 50 feet of the soil profile. According to C.G.S. (2008) and SCEC (1999), zones with factors of safety below 1.3 are considered potentially liquefiable. The results identified 9.4 feet of potentially liquefiable soil within the upper 50 feet of the soil profile, with a main potentially liquefiable zone between the depths of 37 and 50 feet below the ground surface.

For the 50-foot soil profile, the estimated volumetric strain within the 9.4 feet of liquefiable zones resulted in approximately 2.6 inches of liquefaction-induced settlement, combined with 2.0 inches of dry sand settlement, leading to a total seismic-induced settlement of 4.6 inches. Detailed calculations supporting these findings are presented in Appendix D of this report.

Assessment of field data suggests that ground damage, such as sand boils, ground cracks, and other related issues, may not be expected at the site due to the substantial thickness of the overlying non-liquefiable soil layers at the CPT-1 location. According to Ishihara (National Academy Press, 1985), sufficient overburden can prevent surface manifestations due to liquefaction.

Although the analysis indicates no anticipated ground damage, the findings suggest a potential for differential areal settlement. As previously mentioned, the total

liquefaction-related settlements could reach approximately 4.6 inches for the 50-foot soil profile. According to SCEC (1999), up to half of the total settlement could manifest as differential settlement, meaning the differential settlement at the ground surface could be up to 2.3 inches over a horizontal distance of 30 feet (0.0064L).

Lateral spreading is a ground failure phenomenon where loose, water-saturated soils lose strength and displace horizontally, often due to seismic shaking and when the soil has been temporarily weakened because of liquefaction. It commonly occurs near free faces (free face lateral spreading), such as riverbanks, but can also happen on gently sloped grounds without a free face (ground oscillation), causing ground cracking, tilting, and infrastructure damage. The extent of movement depends on soil properties, slope gradient, and earthquake intensity.

Based on procedures developed by Youd, Hansen and Bartlett (2002), the depth to the top of the liquefied layer must be between 1 and 10 meters. Therefore, to evaluate the permanent horizontal displacement at the ground surface, we evaluated the liquefaction-induced lateral spreading by limiting the depth of the analysis to 33 feet (or 10 meters). Because the depth of historical high groundwater at the project site is assumed to be approximately 37 feet, the soils within the upper 33 feet are not expected to undergo liquefaction. Therefore, the potential for lateral spreading at the project site is low.

F. Dry Sand Settlement

Dry sands tend to settle and densify when subjected to earthquake shaking. The amount of settlement is a function of relative density, cyclic shear strain magnitude, and the number of strain cycles. Procedures to evaluate this type of settlement were developed by Seed and Silver (1972) and later modified by Pyke, et al. (1975). Tokimatsu and Seed (1987) presented a simplified procedure that has been reduced to a series of equations by Pradel (1998).

Analysis using CPT results was conducted with Earth Systems' proprietary spreadsheets, applying two-thirds of the peak ground acceleration to soil layers above the groundwater level to assess seismic-induced settlement of dry sands. With the groundwater table at a depth of 37 feet, the maximum total dry sand settlement was calculated to be 2 inches. Detailed calculations for the combined liquefaction-induced and dry sand settlement are provided as Appendix D of this report.

G. Flooding

Earthquake-induced flooding types include tsunamis, seiches, and reservoir failure. Due to the inland location of the site, tsunamis do not pose a hazard to the project.

There are no nearby lakes; thus, seiches do not pose a hazard to the project.

According to the mapping included in the Hazards Appendix of the Ventura County General Plan (2013), the site is not located within a potential reservoir failure inundation area. As a result, the hazard posed by reservoir failure is considered low.

The site is located within an area designated by FEMA Flood Map Service Center website as Zone X, which is designated as an "area of minimal flood hazard". As a result, it appears that the hazard posed by storm-induced flooding is low.

INFILTRATION TESTING

On March 24, 2025, two hand-augered borings (IT-1 and IT-2), each approximately 6 inches in diameter, were advanced to depths of about 2 feet and 3 feet, respectively, below the existing ground surface for infiltration testing. Augering was terminated at these depths due to refusal caused by the cobbles in the near-surface soils at this location. Refer to the Site Plan in Appendix A for the infiltration test locations.

After drilling was completed, 4-inch diameter slotted PVC casings were lowered into the boreholes. The annuli between the casings and boring walls were then filled with pea gravel. The holes were then presaturated. The falling-head borehole infiltration test procedure was used for infiltration testing. To start the tests, water was added to the holes until it was about 2 feet deep, and then the drop in the water surface was monitored by taking periodic measurements. Readings were taken at reasonable time intervals based on the infiltrating rate, and after each of these intervals, water was added to return the water level to its approximate original depth above the hole bottom. The tests were run until the infiltration rates were reasonably stable.

It should be noted that the rate the water surface drops in a borehole is a percolation rate, which is related to, but is not an infiltration rate. Percolation rate ignores the wetted soil surface area into which the water is infiltrating and does not account for the volume of water infiltrated. An

infiltration rate considers both factors. Hence, percolation rates (in unit length per unit time) are an overestimation of infiltration rates (also in unit length per unit time).

Earth Systems uses the Porchet equation to account for the wetted surface area and volume of water infiltrated to estimate an infiltration rate. Forms of the equation can be found in the Riverside County - Low Impact Development BMP Design Handbook (2001), the South Orange County Version, Technical Guidance Documents Appendices (2017), or in a paper by J.W. Van Hoorn, "Determining Hydraulic Conductivity with the Inversed Auger Hole and Infiltrometer Methods." The Porchet equation in its most simple form is the volume of water infiltrated divided by the product of the change in time and the wetted surface area. By substitution, the equation can be shown to be equal to:

$$\text{Infiltration Rate (inches /hr.)} = 60\Delta H \times r / (r + 2H_{avg}) \times \Delta t$$

Where: ΔH = Change in water level (inches)
 Δt = Change in time (minutes)
 r = Radius of test hole (inches)
 H_{avg} = Average height of water in test hole (inches)

The above equation does not account for the gravel pack in the annulus between the borehole wall and the slotted pipe fitted in the test hole. Ignoring the gravel pack inflates the amount of water infiltrated and, hence, yields an unconservative infiltration rate. A method to account for the volume occupied by the gravel (and the slotted pipe) and adjust the infiltration rate accordingly is presented in Caltrans Test 750. Earth Systems makes this additional adjustment to our test data. The equation is:

$$\text{Correction Factor} = n \times [1 - (O/D)^2] + (I/D)^2$$

Where: n = Pea gravel porosity
 O = Outside diameter of slotted pipe (inches)
 D = Test hole diameter (inches)
 I = Inside diameter of slotted pipe (inches)

Earth Systems has determined an average porosity for the pea gravel used in our testing. The other values are simple measurements.

Based on the infiltration testing results in Appendix E, the measured test infiltration rates for the depths tested and boring locations are summarized in the following table:

Boring	Boring Depth (feet)	Infiltration Rate (inch/hour)
IT-1	3	1.74
IT-2	2	0.08

Based on the Ventura County Technical Guidance Manual for Stormwater Quality Control Measures (2010), infiltration facilities require a minimum soil infiltration rate of 0.5 inches/hour. If infiltration rates exceed 2.4 inches/hour, runoff should be fully treated in an upstream BMP (Best Management Practice) prior to infiltration to protect groundwater quality.

The infiltration rate from boring IT-1 met the minimum requirement, as the rates exceed 0.5 inches/hour. The minimum recorded infiltration rate for boring IT-2 was 0.08 inches/hour which is below the required minimum soil infiltration rate of 0.5 inches/hour, indicating that the soil does not meet the infiltration criteria. Based on the soil type encountered in each of the infiltration test holes (i.e., sandy gravel) and the close proximity of the test holes to each other, the failing test for boring IT-2 may be the result of a cobble present in the bottom of the test hole that prevented downward movement of the water. Therefore, we believe that the calculated infiltration rate for boring IT-1 is more representative of the near-surface soils. However, it may be prudent to either perform additional infiltration testing or conservatively use the average of the two tests (i.e., 0.91 inch/hour).

It should be noted that proper drainage is essential for the long-term performance of the proposed synthetic turf field and surrounding track and field areas. The drainage system should be designed to efficiently collect and convey stormwater away from the playing surfaces, prevent water-related damage to the turf system, and limit the potential for saturation of the underlying soils. We recommend that the drainage system be designed to collect and discharge stormwater through subsurface piping rather than relying solely on infiltration.

There are many factors that influence the infiltration rate. Clear water was used in our tests, whereas deleterious material will likely be contained in the storm water. Variations in soil conditions within the limits of the proposed infiltration system will likely affect infiltration characteristics. The designer who utilizes the infiltration results should consider these factors, as well as apply a factor-of-safety to the infiltration rate to account for future disposal bed siltation.

COMPRESSIVE STRENGTH TESTING OF CONCRETE

As part of our study, a total of three (3) cores were collected from the existing concrete bleachers on the north side of the stadium. The cores were taken at the designated locations shown on the Site Plan in Appendix A.

The unconfined compressive strengths were as follows:

Core Number	Unconfined Compressive Strength (psi)
1	6,148
2	7,286
3	6,740

The test results are presented in the Compression Test Report provided in Appendix B.

GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

The site is suitable for the proposed development from both Engineering Geology and Geotechnical Engineering standpoints, provided that the recommendations outlined in this report are successfully implemented into the project. The primary geological and geotechnical considerations from a development standpoint are as follows:

- The potential for about as much as 4.6 inches of total seismic-induced settlement due to liquefaction and settlement of dry sands.
- The potential for 11 feet of undocumented fill in the areas of the proposed ramp and stairs on the south side of the stadium for field access.
- The possibility that the proposed ramp and stairs on the south side of the stadium for field access are within 50 feet of the projected fault trace. A fault study is planned for the proposed fieldhouse that will better define the location of the surface projection of the fault trace across the athletic stadium.

Specific conclusions and recommendations addressing these geologic and geotechnical considerations, along with general geotechnical design and construction guidelines, are presented in the following sections.

A. Grading

1. Pre-Grading Considerations

- a. Plans and specifications should be provided to Earth Systems prior to grading. Plans should include the grading plans, foundation plans, and foundation details.
- b. Drainage systems should be designed so that water is not discharged into the bearing soil or near structures.
- c. Final site grade should be designed so that all water is diverted away from the structures over paved surfaces, or over landscaped surfaces in accordance with current codes. Water should not be allowed to pond anywhere on the pads. Nor should water be allowed to flow over the top of adjacent descending slopes.
- d. Shrinkage due to compaction of the site soils is expected to be approximately 11 percent of any excavated or scarified site soils. This estimate is based upon compactive effort needed to produce an average degree of relative compaction of approximately 92 percent and may vary depending on contractor methods. Losses from site clearing and grubbing operations may affect quantity calculations and should also be considered. Bulking for export purposes is estimated to be approximately 22 percent. The grading contractor should verify shrinkage and earthwork yardage estimates.
- e. It is recommended that Earth Systems be retained to provide Geotechnical Engineering services during site development and grading, and foundation construction phases of the work to observe compliance with the design concepts, specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.
- f. Compaction tests shall be made to determine the relative compaction of the fills in accordance with the following minimum guidelines: one test for each two-foot vertical lift; one test for each 1,000 cubic yards of material placed; and two tests at finished subgrade elevation.

2. Rough Grading/Areas of Development

- a. Grading at a minimum should conform to Appendix J in the 2022 CBC, and with the recommendations of the Geotechnical Engineer during construction. Where the recommendations of this report and the cited section of the 2022 CBC are in conflict, the Owner should request clarification from the Geotechnical Engineer.

- b. The existing ground surface in the area of the proposed improvements should be initially prepared for grading by removing all existing structures within the construction limits, abandoned utility lines (if any), debris, vegetation and other organic materials (if any) and non-complying fill from construction areas. Debris should be stockpiled away from areas to be graded and ultimately removed from the project site to prevent their inclusion in fills. Voids created by removal of such material should be properly backfilled and compacted. No compacted fill should be placed unless the underlying soil has been observed by the Geotechnical Engineer or his or her representative.
- c. For the proposed improvements on the north side of the stadium (i.e., new stairs and new ramp system), soil should be overexcavated to a depth of 2 feet below the bottom of the footings or as deep as necessary to remove all existing fill material, whichever is deeper. Overexcavation should be extended to a distance of at least 2 feet laterally, but not less than a distance equal to the depth of removal, beyond the outside edge of the proposed improvement.
- d. Given the proximity of the new improvements on the north side of the stadium to the existing 7-foot-high retaining wall, it may be necessary to complete the remedial grading by means of a series of slot cuts. Slot cuts should be completed in an "A-B-C" sequence. The width of any slot cut should not exceed 8 feet. The complete construction and backfill of the 'A' slot must be performed prior to excavation of the 'B' slot, and likewise the 'C' slot. After the cut for each slot is made, bracing and possibly shoring of the near vertical cut slope **may** be required for continued temporary stability of slot excavation and safety of personnel.
- e. For the proposed improvements on the south side of the stadium (i.e., new stairs and ramp for field access), the depth of the undocumented fill may be as much as 11 feet or more based on our borings (HA-3 and HA-4). Removal of all the existing fill may be impractical due to the presence of existing structures and space limitations. Therefore, we recommend that soil should be overexcavated to a depth of 3 feet below the bottom of the footings or as deep as necessary to remove all existing fill material, whichever is deeper. Overexcavation should be extended to a distance of at least 5 feet laterally, but not less than a distance equal to the depth of removal, beyond the outside edge of the proposed improvement.

- f. Soils beneath synthetic turf field, running track, and any proposed traffic-bearing pavement, including a minimum lateral distance of at least 3 feet beyond improvement edges, should be excavated a minimum of 2 feet below the existing grade or finished subgrade, whichever is deeper. The bottom of the remedial excavation should then be scarified 6 inches. The scarified and excavated soils should be moisture conditioned to near optimum moisture content and be uniformly compacted to at least 90 percent of maximum dry density using mechanical compaction equipment.
- g. Soils beneath any proposed exterior non-traffic bearing concrete flatwork (sidewalks, patios, walkways etc.), including a minimum lateral distance of at least 3 feet beyond flatwork edges, should be excavated a minimum of 2 feet below the existing grade or finished subgrade, whichever is deeper.
- h. The bottoms of all excavations should be observed by a representative of this firm prior to processing (i.e., scarification and recompaction). Once observed and approved, the bottom of the remedial excavation should then be scarified 6 inches. The scarified soils should be moisture conditioned to above the optimum moisture content and uniformly compacted to at least 90 percent of the maximum dry density using mechanical compaction equipment. Compaction of the prepared subgrade at the bottom of the remedial excavations should be verified by testing prior to the placement of engineered fill.
- i. Engineered fill should be placed in a series of horizontal layers not exceeding 8 inches in loose thickness, uniformly moisture-conditioned to above the optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D 1557 maximum dry density. Compaction of the engineered fill should be verified by testing. Additional fill lifts should not be placed if the previous lift did not meet the required relative compaction or if soil conditions are not stable. Discing, tilling, and/or blending may be required to uniformly moisture-condition soils used for engineered fill.
- j. Voids created by dislodging cobbles and boulders during excavation should be backfilled and recompacted and the dislodged cobbles and boulders larger than 6 inches in diameter should be removed from the subgrade
- k. On-site soils may be used for fill once they are cleaned of all organic material, rock, debris and irreducible material larger than 6 inches
- l. Import soils used to raise site grade should be equal to, or better than, on-site soils in strength, expansion, and compressibility characteristics. Import soil can

be evaluated but will not be prequalified by the Geotechnical Engineer. Final comments on the characteristics of the import will be given after the material is at the project site.

- m. Backfill around or adjacent to confined areas (i.e. interior utility trench excavations, etc.) may be performed with a lean sand/cement slurry (maximum 28-day compressive strength of 200 psi) or "flowable fill" material (a mixture of sand/cement/fly ash). The fluidity and lift placement thickness of any such material should be controlled in order to prevent "floating" of any "submerged" structure. Alternatively, a gravel backfill be used, subject to approval by the Geotechnical Engineer and the County or DSA official.

3. Utility Trenches

- a. Utility trench backfill should be governed by the provisions of this report relating to minimum compaction standards. In general, on-site service lines may be backfilled with recompacted fill compacted to 90 percent of the maximum density. Backfill of offsite service lines will be subject to the specifications of the jurisdictional agency or this report, whichever are greater.
- b. Utility trenches running parallel to footings should be located at least 5 feet outside the footing line, and above a 2-horizontal versus 1-vertical projection downward from a point 9 inches above the bottom of the footing. If the utility line is located below the 2:1 projection, the trench will need to be backfilled with slurry to a level above the 2:1 projection.
- c. Compacted fills should be utilized for backfill. Clean sand backfill should be avoided under structures because it provides a conduit for water to migrate under foundations.
- d. Backfill operations should be observed and tested by the Geotechnical Engineer to monitor compliance with these recommendations.
- e. Although not anticipated, shoring and/or sloping of shallow trenches may be required because of the potential presence of caving sandy soils. Trenches deeper than 5 feet should be sloped or shored in accordance with OSHA guidelines.
- f. Rocks greater than 6 inches in diameter should not be placed in trench zones (from 12 inches below pavement subgrade or ground surface to 12 inches above top of pipe or box); rocks greater than 2.5 inches in diameter should not be

placed in pipe zones (from 12 inches above top of pipe or box to 6 inches below bottom of pipe or box exterior).

- g. Jetting should not be utilized for compaction in utility trenches.
- h. If any utilities are to be located in public rights-of-way, construction will need to comply with the requirements of the public agency.

4. Excavations

- a. Excavations within the depth of the recommended remedial grading will typically encounter relatively silt-sand mixture. This material should be easily excavated with conventional earthmoving equipment.
- b. Temporary unshored, unsurcharged, open excavations above the groundwater level may be cut vertically to a maximum height of no more than 4 feet. Excavations extending higher than 4 vertical feet should be sloped back above the 4-foot vertical cut to at least a 1.5-horizontal versus 1-vertical slope or flatter. If excavations dry out, sloughing will occur. No excavation should be made within a 1:1-line projected outward from the toe of any existing footing or structure.
- c. During the time excavations are open, no heavy grading equipment or other surcharge loads (i.e., excavation spoils) should be allowed within a horizontal distance from the top of any slope equal to the depth of the excavation (both distances measured from the top of the excavation slope).
- d. Shoring of trenches may be required due to the potential presence of caving sand.
- e. All open cuts should be in compliance with applicable Occupational Safety Health Administration (OSHA) regulations (California Construction Safety Orders, Title 8) and should be monitored for evidence of incipient instability. Standard construction techniques should be sufficient for temporary site excavations. Project safety is the responsibility of the Contractor and the Owner. Earth Systems will not be responsible for project safety.

5. Temporary Shoring

Earth Systems anticipates that excavations for the recommended remedial grading may be approximately 10 feet deep where the proposed ramp system notched into the existing slopes. If laying back the slope is not feasible, temporary shoring would be necessary. The shoring may consist of cantilevered or braced/tieback sheet piling

or cantilevered or braced soldier beam and lagging systems or similar temporary shoring system. For the purposes of this report, the temporary shoring is assumed to be a cantilevered soldier beam and lagging shoring system.

- a. Cantilevered temporary shoring for construction of the recommended remedial grading should be designed to resist active lateral earth pressures. A temporary shoring system retaining a level ground surface should be designed for 44 pcf of equivalent fluid weight per foot of depth measured below the top of the retained ground surface behind the shoring. For shoring retaining a ground surface with an inclination of no steeper than 4H:1V should be designed for 54 pcf of equivalent fluid weight.
- b. The lateral earth pressure to be resisted by the temporary shoring should be increased to allow for surcharge loads. The surcharge considered should include the loads from any other structures or vehicle traffic within a distance at least equal to the height of the shoring. Surcharge effects for cantilevered shoring may be computed assuming active earth pressure conditions using a pressure coefficient of 0.35.
- c. Lateral (horizontal) loads may be resisted by passive resistance of soil against the piers. An equivalent fluid weight (EFW) of 355 psf per foot of penetration in firm, native soil above the groundwater table may be used for lateral load design. The maximum passive pressure used for design should not exceed 4,500 psf. For soldier piers spaced at least 3 diameters apart, an effective width of 2.5 times the actual pile diameter may be used for passive pressure calculations.
- d. Axial loads on the soldier piers may be resisted by skin friction on the piers below the depth of the excavation (assumed to be approximately 10 feet below existing grade). Drilled piers for the shoring system should extend at least 15 feet below the deepest excavation level and be a minimum of 2 feet in diameter. The skin friction on the proposed soldier piers above and below the lowered groundwater level due to construction dewatering may be taken as 200 psf. The skin friction should be multiplied by the perimeter surface area of the grouted soldier pile.
- e. To limit sloughing, exposed soil between soldier piers should be supported by lagging and voids that are backfilled. Backfill of voids behind lagging should consist of concrete slurry (1- to 2-sack of cement per CY) placed so that no voids remain between the lagging and the excavation face of soil or bedrock. Alternatively, exposed soil and bedrock supported through the use of reinforced

gunite or other approved material designed to minimize soil movement. Because of arching, the lateral earth pressure on lagging may be taken as 60 percent of the overall lateral earth pressure on the shoring system (need not include seismic pressure). All timber lagging to be left in the ground should be pressure treated in accordance with Standard Specifications for Public Works Construction, Section 204-2. Elimination of lagging from portions of the excavation may be considered depending upon the stability of the material as observed after installation of soldier piers and during excavation. Lagging may be eliminated only upon approval of the Geotechnical Engineer and the City of Ventura.

- f. It is recommended that Earth Systems review the plans and specifications for the proposed shoring system and that an Earth Systems representative observe and monitor the installation of the shoring.

B. Structural Design

1. Conventional Spread Footings

Because the estimated total differential settlement is less than the differential settlement threshold given in Table 12.13-3 of ASCE 7-16 for the type of structures proposed, and lateral spreading is not anticipated, conventional spread footings may be used to support the proposed structures.

- a. All footings for the proposed improvements should bear into firm, recompacted fill as recommended elsewhere in this report. Foundation excavations should be observed by a representative of this firm after excavation, but prior to placing of reinforcing steel or concrete, to verify bearing conditions.
- b. Footings should have a minimum embedment depth of 18 inches and may be designed based on an allowable bearing value 2,200 psf. This value includes a factor of safety of 3.0.
- c. Allowable bearing values are net (weight of footing and soil surcharge may be neglected) and are applicable for dead plus reasonable live loads. A one-third increase in the bearing values is permitted for use with the alternative load combinations given in Section 1605.2 of the 2022 CBC.
- d. Continuous foundations on slopes should be stepped to maintain horizontal bottoms along all portions of the foundation.
- e. Isolated pad foundations should be restrained laterally in both directions by means of grade beams, structural slab, or other approved method.

2. Drilled Piers

Drilled piers may be used to support the proposed light standards and goal posts. Foundation piers should be designed as friction piles. Piers may consist of drilled, reinforced cast-in-place concrete caissons (cast-in-drilled-hole "CIDH" piles). Piers may be drilled or hand-dug. Steel reinforcing may consist of "rebar cages" or structural steel sections

- a. As a minimum, the new piers should be at least 36 inches in diameter and have a minimum length of 10 feet into the firm native soil. The Geotechnical Engineer should be consulted during pier installation to determine compliance with the geotechnical recommendations.
- b. For vertical capacity, the piers may be proportioned using an allowable skin friction (adhesion) value of 200 pounds per square foot (psf) should be used. For axial uplift loads, a skin friction (adhesion) value equal to two-thirds of the downward capacity should be used. No vertical load carrying capacity is provided for the portions of the piers in artificial fill soil. The load capacities should be based upon skin friction with no end bearing. These allowable capacities include a safety factor of 2.0 and may be increased by one-third when considering transient loads such as wind or seismic forces.
- c. Reduction in axial capacity due to group effects should be considered for piers spaced at 3 diameters on-center or closer, and a reduction in lateral capacity due to group effects should be considered for piers spaced at 6 to 8 diameters on-center or closer.
- d. These allowable skin friction values provided above are based upon available subsurface field data and on Earth Systems' experience on similar projects. The compressive and tensile strength of new pier designs should be checked to verify the structural capacity of the piers. Reinforcement of piers should be specified by the Project Structural Engineer. The specific method of pier installation will affect the performance of the piers. Earth Systems recommends a meeting with the design team and contractor to verify that the specific method of pier installation can provide the anticipated load supporting capacity.
- e. When seismic loads are considered, a down-drag (negative skin friction) force does not need to be applied because liquefaction is not anticipated to happen within the depths of the piers.

- f. Lateral (horizontal) loads may be resisted by passive resistance of the soil against the piers. An equivalent fluid weight (EFW) of 300 psf per foot of penetration in the native soil above the groundwater level may be used for lateral load design. Below the groundwater level, an equivalent fluid weight (EFW) of 150 psf per foot of penetration in artificial fill and alluvium may be used. The resisting pressures provided are ultimate values. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended). In passive pressure calculations, the upper one foot of soil should be subtracted from the depth of penetration unless confined by pavement or slab.
- g. For piers spaced at least three diameters apart, an effective width of 2.5 times the actual pier diameter may be used for passive pressure calculations.
- h. It is the Project Structural Engineer's responsibility to design the reinforcement for the piers to sustain the imposed axial and lateral loading.

3. Drilled Pier Installation

The following recommendations are based upon Earth Systems' analyses of the geotechnical conditions at the project site and our understanding of the project. The project civil and structural engineers may require additional installation criteria based on other factors (type of pile, structural design, method of construction, etc.).

- a. The geotechnical engineers, or their representatives, should be present during excavation and installation of all piers to observe subsurface conditions, and to document penetration into load supporting materials (i.e., hydraulically placed artificial fill).
- b. Since the piers are designed to rely on intimate frictional contact with the soil, any casing (if used) should be removed during placement of concrete. Slick or smeared zones on the sidewalls of the boreholes should be removed and, bentonite slurry and similar stabilizing fluids should not be used in boreholes without allowing for a reduction in pile load capacity.
- c. The design mix for the concrete to be used in the pier construction should be established and approved by the structural engineer prior to the time of construction. Compression tests should be performed on samples of the concrete in accordance with applicable codes or requirements of the structural engineer. Inspection by qualified personnel should be provided during the concrete batching and during placement of pier steel and concrete.

- d. Piers located within three pier diameters of each other should be drilled and filled alternately so that concrete is permitted to set before drilling an adjacent pier. The time for initial set of the concrete will depend on the design mix and should be determined in the field at the time of construction. No fewer than 4 hours should be allowed for the concrete to set before drilling for an adjacent pier. No pier hole should be left open overnight. Since the exact pier installation process is not known at this time, it is important for Earth Systems to be consulted relative to recommendations for placement criteria to aid in maintaining the integrity of the piers during placement.
- e. The bottoms of pier excavations should be relatively clean of loose soils and debris prior to placement of concrete. Any water encountered should be pumped from the boreholes prior to the placement of concrete, or placement of concrete should be by use of a tremie or pump line such that the water is displaced during the concrete placement. The volume of concrete placed should be measured to compare with the design volume.
- f. Installed piers should not be more than two percent (2%) from the plumb position.

4. Slabs-on-Grade

- a. Concrete slabs-on-grade should be supported by firm recompacted fill as recommended elsewhere in this report.
- b. The information that follows regarding design criteria for slabs is generally the same as that given in Table 1809.7 for the "Low" expansion range. Actual slab designs should be provided by the Project Structural Engineer, but the reinforcement and slab thicknesses recommended should not be less than the criteria set forth in Table 1809.7 for the appropriate expansion range, or as recommended below, whichever is more stringent.
- c. Slabs bearing on soil in the "Low" expansion range should be underlaid with a minimum of 4 inches of clean sand. Areas where floor wetness would be undesirable should be underlaid with a vapor retarder (as specified by the Project Architect or Civil Engineer) to reduce moisture transmission from the subgrade soils to the slab. The retarder should be placed as specified by the project Structural Engineer or Architect.
- d. Slabs bearing on soil in the "Low" expansion range should at a minimum be reinforced at mid-slab with No. 4 bars on 24-inch centers.

- e. Soils should be premoistened to 120 percent of optimum moisture content to a depth of 21 inches below lowest adjacent grade. Premoistening should be confirmed by testing.
- f. It is recommended that perimeter slabs (walkways, patios, etc.) be designed relatively independent of footing stems (i.e., free floating) so foundation adjustment will be less likely to cause cracking.

5. Frictional and Lateral Coefficients

- a. Resistance to lateral loading may be provided by soil friction acting on the base of foundations in compacted engineered fill. A coefficient of friction of 0.60 may be applied to dead load forces. This value does not include a safety factor.
- b. For foundations against compacted engineered fill, a passive resistance equal to 350 pcf of equivalent fluid weight may be included for resistance to lateral load. This value does not include a safety factor.
- c. A minimum safety factor of 1.5 should be used when designing for sliding or overturning.
- d. For retaining walls, passive resistance may be combined with frictional resistance without any reduction.

6. Settlement Considerations

- a. In the event of a strong seismic event, the soil underlying the site may undergo both liquefaction and dry sand seismically-induced settlements. Because both are the result of earthquake-induced vibrations, the settlements from both phenomena are additive. Assuming that the groundwater table rose to its historically shallowest depth of 37 feet, soils underlying the project site could undergo as much as 4.6 inches of combined liquefaction-induced and seismically-induced dry sand settlements (considering the upper 50 feet of the soil profile).
- b. Settlement due to hydro-consolidation (hydro-collapse) should be minimal and may be ignored.
- c. In addition to the settlements described above, the foundations may undergo elastic settlements. Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. We anticipate that conventional spread foundations underlain by compacted fill could undergo about one-half inch (1/2") of static settlement.

- d. Maximum static settlement of drilled piers embedded in firm, native soil is estimated to be about one-half inch (1/2").
- d. Differential settlement between adjacent foundation elements should be about one-half the maximum total settlement (i.e., static and seismic-induced combined) over a horizontal distance of 30 feet.
- e. The Project Structural Engineer will need to design the foundation system to accommodate the anticipated settlement values.

7. Retaining Walls

- a. On-site soils can be used for retaining wall backfill once they are retested for expansion index and cleaned of all organic material, rocks, debris, and irreducible material larger than 6 inches.
- b. Conventional cantilevered retaining walls backfilled with compacted on-site soils may be designed for active pressures developed from 37 pcf of equivalent fluid weight for well-drained, level backfill. Active pressures developed from 44 pcf of equivalent fluid weight may be used for well-drained backfill sloping at 4H:1V (horizontal to vertical).
- c. Restrained retaining walls backfilled with compacted on-site soils may be designed for at-rest pressures developed from 56 pcf of equivalent fluid weight for well-drained, level backfill. At-rest pressures developed from 67 pcf of equivalent fluid weight may be used for well-drained backfill sloping at 4H:1V (horizontal to vertical).
- d. The equivalent fluid weights listed above were based on the assumption that backfill soils will be compacted to 90 percent of the maximum dry density as determined by the ASTM D 1557 Test Method.
- e. Retaining walls that retain more than 6 feet of soil will need to be designed for a seismic loading force that is applied in addition to the static forces when seismic shaking occurs. Seismic increments of earth pressure can be determined using 23 and 36 pcf of additional equivalent fluid weight need to be considered for cantilever and restrained retaining walls, respectively. These equivalent fluid weights have been determined by a procedure presented by Al Atik and Sitar (2010). The seismic increment of pressure can be assumed to be distributed so that the centroid of pressure acts at 0.33H above the base of a retaining wall, where H is the wall height in feet. Because this seismic force is transient, and in

accordance with CBC Section 1807.2.3, a minimum safety factor of 1.1 may be used for sliding and overturning when seismic loads are included.

- f. The lateral earth pressure to be resisted by the retaining walls or similar structures should also be increased to allow for other applicable surcharge loads. The surcharges considered should include forces generated by structures or temporary loads that would influence the wall design.
- g. A system of backfill drainage should be incorporated into retaining wall designs. Backfill comprising the drainage system immediately behind retaining structures should be free-draining granular material with a filter fabric between it and the rest of the backfill soils. As an alternative, the backs of walls could be lined with geodrain systems. The backdrains should extend from the bottoms of the walls to about 18 inches from the finished backfill grade. Waterproofing may aid in reducing the potential for efflorescence on the faces of retaining walls.
- h. Compaction on the uphill sides of walls within a horizontal distance equal to one wall height should be performed by hand-operated or other lightweight compaction equipment. This is intended to reduce potential "locked-in" lateral pressures caused by compaction with heavy grading equipment.
- i. Water should not be allowed to pond near the tops of retaining walls. To accomplish this, final backfill site grades should be such that all water is diverted away from retaining walls

8. Preliminary Asphalt Paving Sections

- a. The following preliminary paving sections table summarizes thicknesses of asphalt and aggregate base required for different traffic indices (ranging from 4.0 to 7.0, with 0.5 intervals) and a design life of 20 years using the tested R-Value of 44. The design engineer should select the appropriate pavement section provided in the table below for the traffic index that he/she has determined appropriate for their application (i.e., pavement subjected to vehicular traffic and/or running track).

Traffic Index	Asphalt Thickness (inches)	Minimum Aggregate Base Thickness (inches)
4.0	3.0	4.0
4.5	3.0	4.0
5.0	3.0	4.0
5.5	3.0	4.5

6.0	3.5	4.5
6.5	4.0	5.0
7.0	4.0	6.0

- b. The subgrade soils in the upper one foot below the finished subgrade elevation should be properly moisture conditioned and compacted to achieve a minimum relative compaction of 95% of the ASTM D1557 maximum dry density.
- c. Aggregate base materials should be compacted as engineered fill to at least 95% compaction.
- d. Asphalt paving materials and placement methods should meet specifications stated in the 2024 "Greenbook" for asphalt concrete.
- e. The preliminary paving sections provided above have been designed for the type of traffic indicated and the assumed design life. If the pavement is placed before project construction is complete, construction loads, which could increase the Traffic Indices above those assumed above, should be taken into account.
- f. The subgrade R-value should be evaluated at or near the end of rough grading so that final pavement designs can be made.

9. Preliminary Concrete Paving Sections

The preliminary concrete pavement design recommendations are provided in this section based on different traffic categories as outlined in ACI 330R Table 3.3 (Traffic Categories A, B, and C). The design assumes a compressive strength of concrete of 3,250 at 28 days, a coefficient of subgrade reaction (k) of 180 pci, which corresponds approximately to an R-Value of 44, and a design life of 20 years. Utilizing the design methodology provided in ACI 330R-01, the following minimum unreinforced concrete pavement thicknesses are recommended:

ADTT	Granular Subbase Thickness	Modified k	Design Thickness (in)	Max Spacing
A (ADTT=1)	6 in.	196	5.0	12.5 ft.
	12 in.	237	4.5	10 ft.
A (ADTT=10)	6 in.	196	5.5	12.5 ft.
	12 in.	237	5.0	12.5 ft.
B (ADTT=25)	6 in.	196	6.0	15 ft.
	12 in.	237	6.0	15 ft.

B (ADTT=300)	6 in.	196	7.0	15 ft.
	12 in.	237	6.5	15 ft.
C (ADTT=100)	6 in.	196	7.0	15 ft.
	12 in.	237	6.5	15 ft.
C (ADTT=300)	6 in.	196	7.0	15 ft.
	12 in.	237	6.5	15 ft.
C (ADTT=700)	6 in.	196	7.5	15 ft.
	12 in.	237	7.0	15 ft.

- b. The preliminary paving section presented above is designed for the type of traffic indicated and the assumed design life. If truck traffic is expected to exceed the provided values or a longer design life is desired, this section should be reevaluated. If the pavement is placed before construction on the project is complete, construction loads should be taken into account. Traffic should not be allowed on the pavement until 28 days after concrete placement, or until the 28-day design strength is achieved.
- c. The preliminary paving section presented above is an unreinforced design. If rebars are incorporated into the paving section, extra crack control will be provided.
- d. Performance of concrete pavement depends on a uniform subgrade upon which to cast the concrete. Hence, the subgrade should be compacted to a minimum of 95% of the maximum dry density determined by ASTM D 1557. Bearing soil should be premoistened to optimum moisture before placing concrete.

ADDITIONAL SERVICES

This report is based on the assumption that an adequate program of monitoring and testing will be performed by Earth Systems during construction to check compliance with the recommendations given in this report. The recommended tests and observations include, but are not necessarily limited to the following:

- 1. Review of the building and grading plans during the design phase of the project.
- 2. Observation and testing during site preparation, grading, placing of engineered fill, and foundation construction.
- 3. Consultation as required during construction.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The analysis and recommendations submitted in this report are based in part upon the data obtained from the in-situ tests and borings drilled on the site. The nature and extent of variations between and beyond the borings may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

The scope of services did not include any environmental assessment or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statements in this report or on the soil boring logs regarding odors noted, unusual or suspicious items or conditions observed, are strictly for the information of the client.

Findings of this report are valid as of this date; however, changes in conditions of a property can occur with passage of time whether they be due to natural processes or works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside the control of this firm. Therefore, this report is subject to review and should not be relied upon after a period of one year.

In the event that any changes in the nature, design, or location of the structure(s) and other improvements are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

This report is issued with the understanding that it is the responsibility of the Owner, or of his representative to ensure that the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plan and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

As the Geotechnical Engineers for this project, Earth Systems has striven to provide services in accordance with generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This report was prepared for the

exclusive use of the Client for the purposes stated in this document for the referenced project only. No third party may use or rely on this report without express written authorization from Earth Systems for such use or reliance.

It is recommended that Earth Systems be provided with the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems is not accorded with the privilege of making this recommended review, it can assume no responsibility for misinterpretation of the recommendations.

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APPENDIX A

Vicinity Map

Regional Fault Map

Regional Geologic Map 1 (Dibblee)

Regional Geologic Map 2 (USGS/CGS [SCAMP])

Seismic Hazard Zones Map

Historical High Groundwater Map

Field Study

Site Plan

Fault Location and Trench Log

Logs of Borings

Logs of CPT Sounding and Interpretation

Boring Log Symbols

Unified Soil Classification System



*Taken from USGS Topo Map,
Ventura Quadrangel,
Ventura County, California, 2022.

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

N



VICINITY MAP

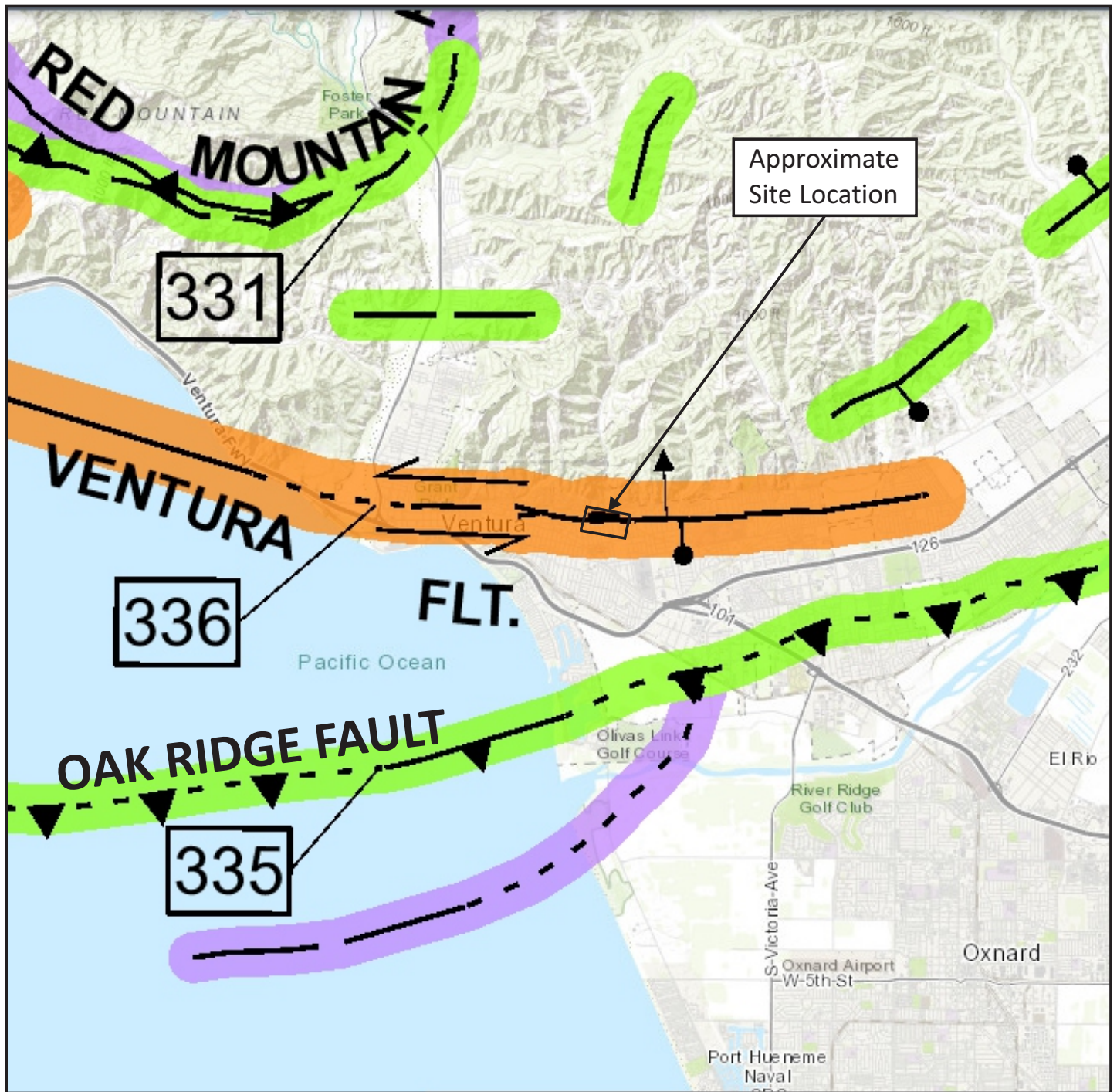
Ventura High School Track and Field
Ventura, California



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August 2025

305811-006



*Taken from Jennings and Bryant, Geologic Data Map No.6, 2010

Approximate Scale:
1 Inch = 2 Mile



REGIONAL FAULT MAP

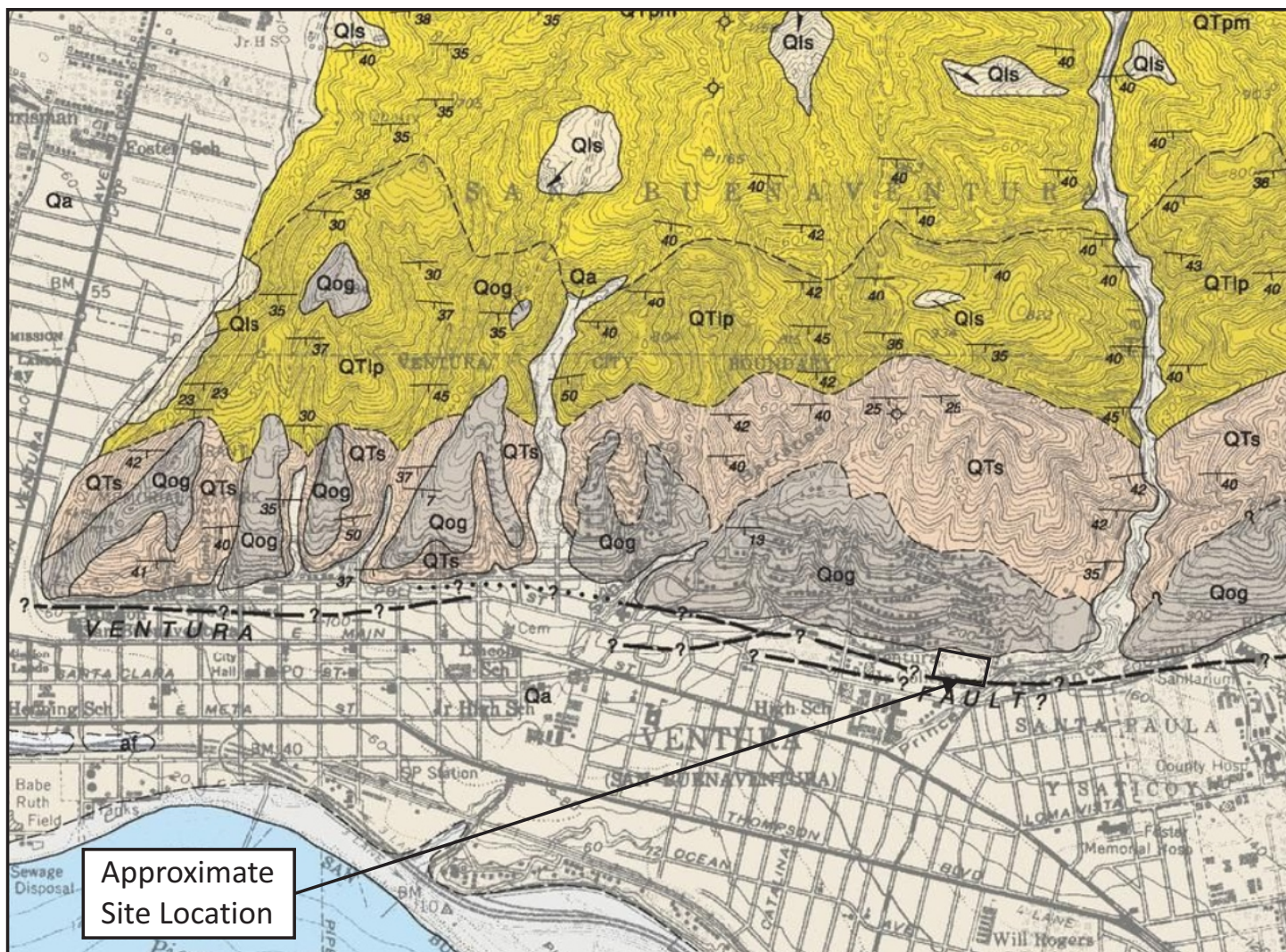
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Ventura, California



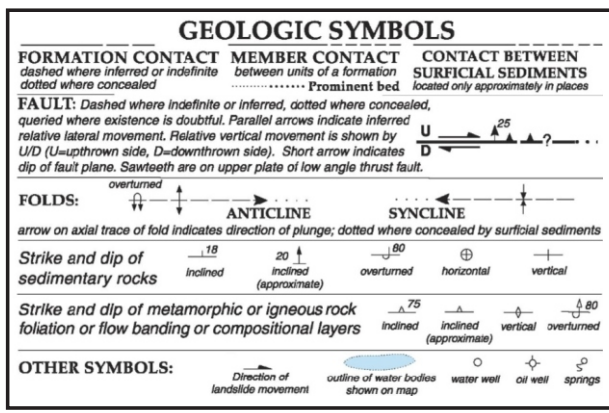
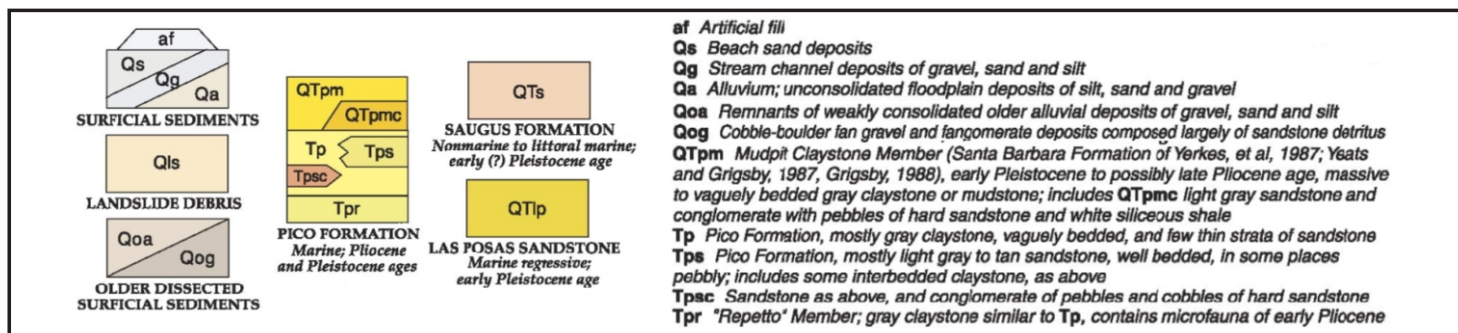
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*Taken from Dibblee, T.W., and Ehrenspeck, H.E., ed., 1988, Geologic map of the Ventura and Pitas Point quadrangles, Ventura County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-21



Approximate Scale: 1" = 2,000'



REGIONAL GEOLOGIC MAP 1

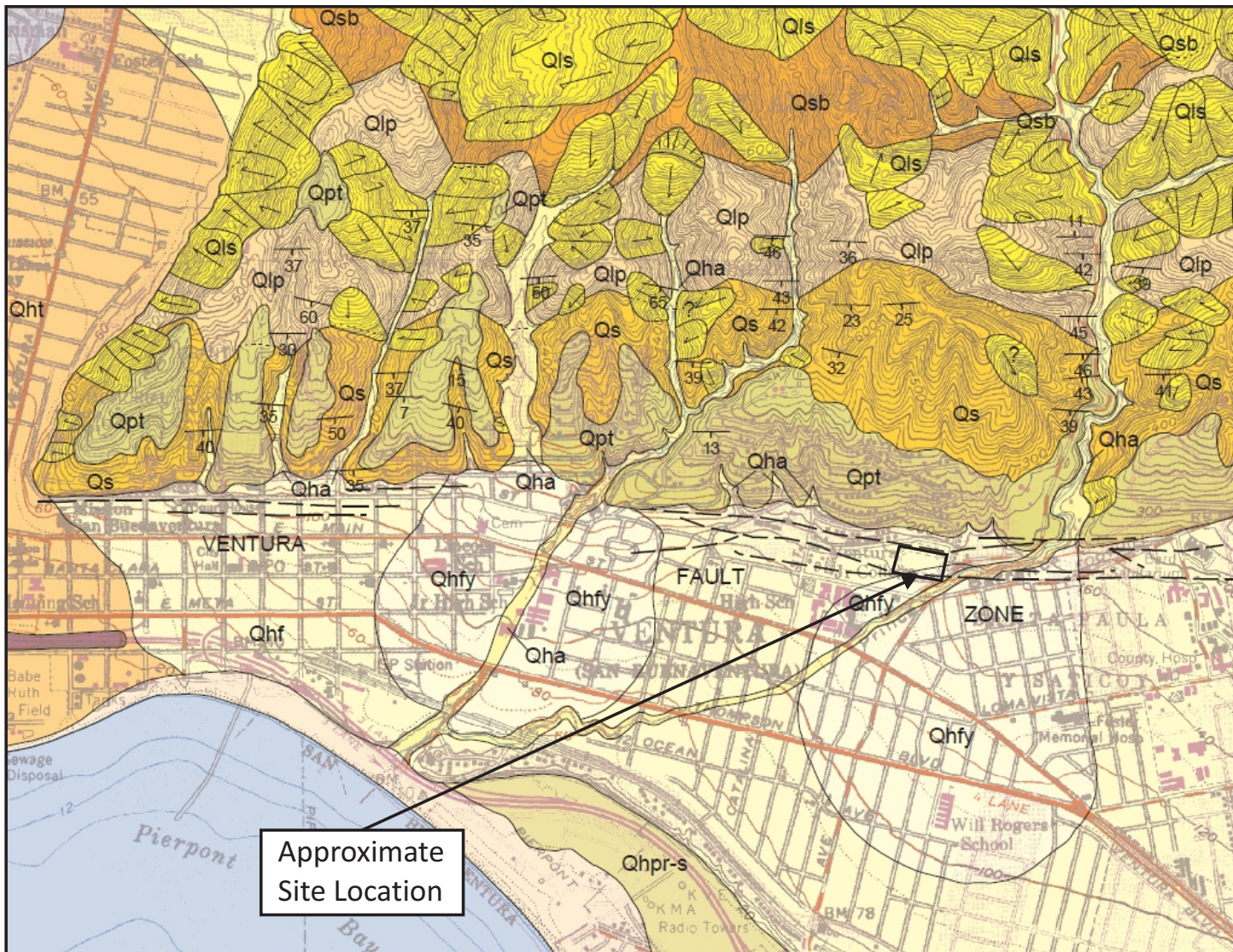
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*Taken from USGS, SCAMP Geologic Map of the Ventura 7.5' Quadrangle, Ventura County, California, 2003.

Qhfy	Latest Holocene alluvial fan deposits, deposited by streams emanating from mountain canyons onto alluvial calley floors; deposits originate as debris flows, hyperconcentrated mudflows, or braided stream flows; composed of moderately to poorly sorted, and moderately to poorly bedded sandy clay with some gravel.	Qls	Holocene to Pleistocene landslide deposits, include numerous active landslides; composed of weathered broken up rocks; extremely susceptible to renewed landsliding.
Qha	Undivided Holocene alluvial and colluvial deposits on the floors of valleys, includes active stream deposits in hill slope areas; composed of unconsolidated sandy clay with some gravel.	Qhpr-s	Holocene paralic deposits of the Sea Cliff marine terrace 1800 to 5800 years old (Lajoie, and others, 1982); composed of semi-consolidated sandy clay with some gravel.
Qht	Holocene stream terrace deposits, deposited in point bar and overbank settings; composed of unconsolidated clayey sand and sandy clay with gravel.	Qs	Pleistocene Saugus Formation; weakly consolidated alluvial deposits composed of sandstone and siliceous shale gravel and cobbles in sandy matrix; highly susceptible to landsliding.
Qhf	Holocene alluvial fan deposits; deposited by streams emanating from mountain canyons onto alluvial valley floors; deposits originate as debris flows, hyperconcentrated mudflows, or braided stream flows; composed of moderately to poorly sorted, and moderately to poorly bedded, sandy clay with some gravel.	Qlp	Pleistocene Las Posas Sandstone; weakly indurated sand, with some gravelly sand units; highly susceptible to landsliding.
		Qsb	Pleistocene Santa Barbara claystone; locally contains Monterey Formation shale fragments; highly susceptible to landsliding.



Approximate Scale: 1" = 2,000'

0 2,000' 4,000'



REGIONAL GEOLOGIC MAP 2

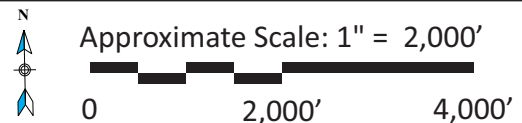
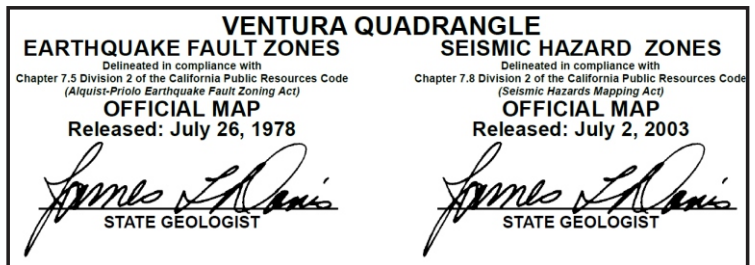
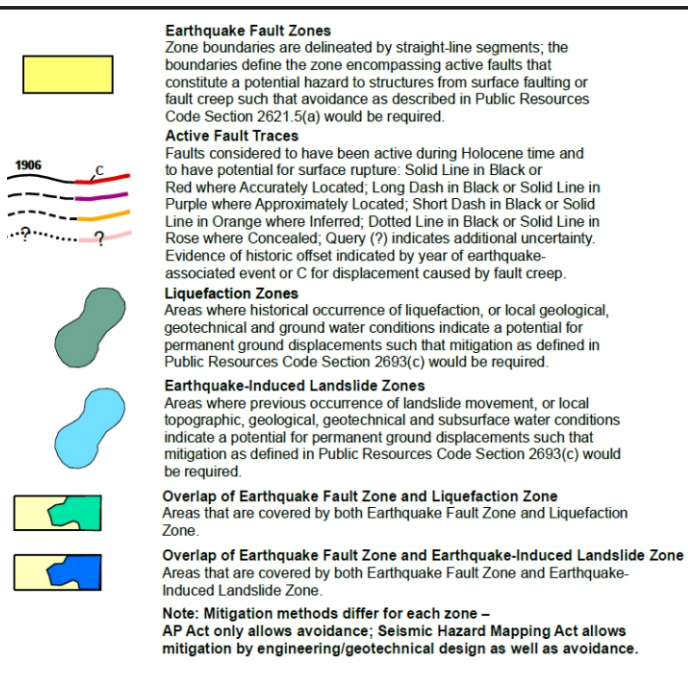
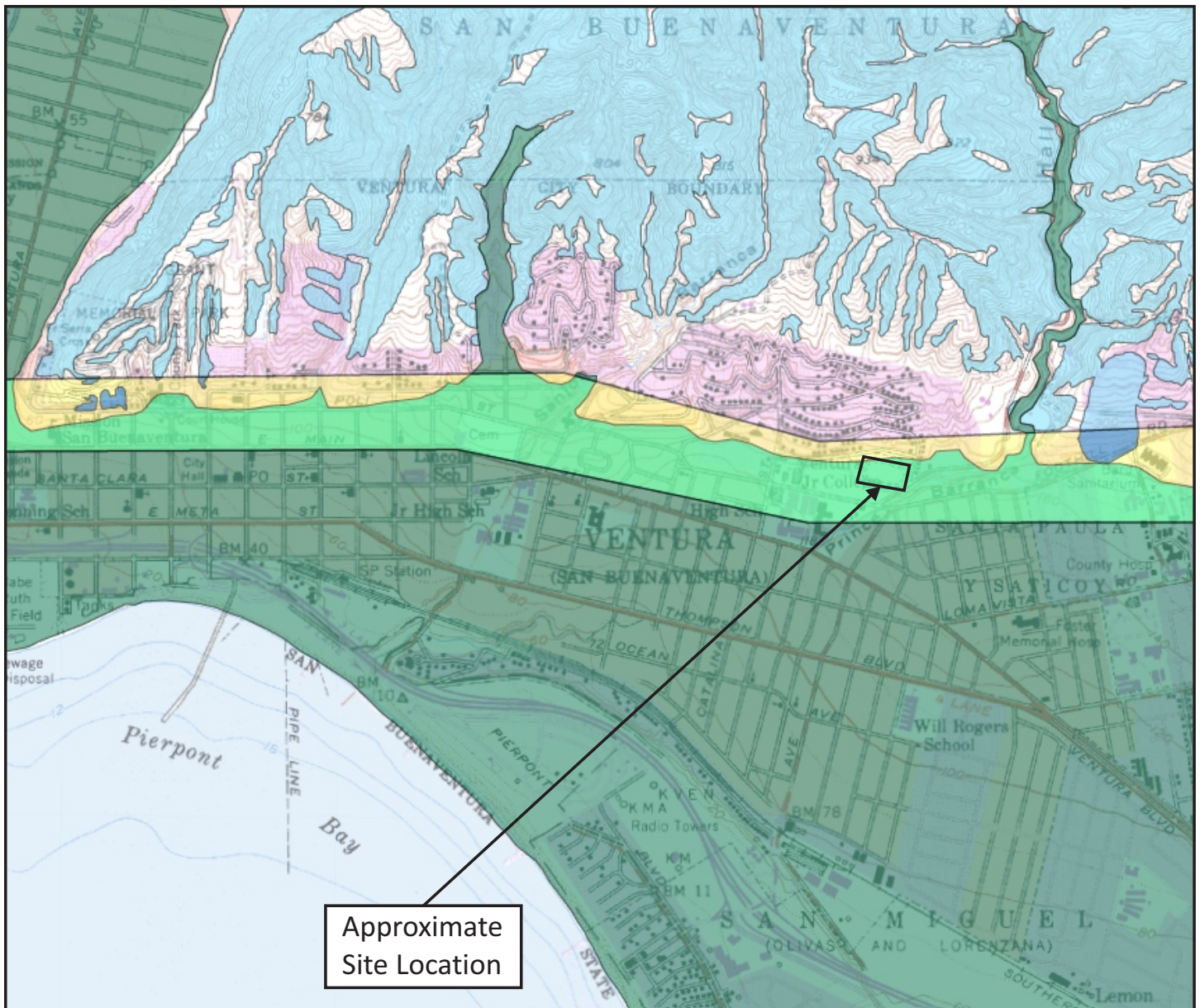
Ventura High School Track and Field
Ventura, California



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August 2025

305811-006



SEISMIC HAZARD ZONES MAP

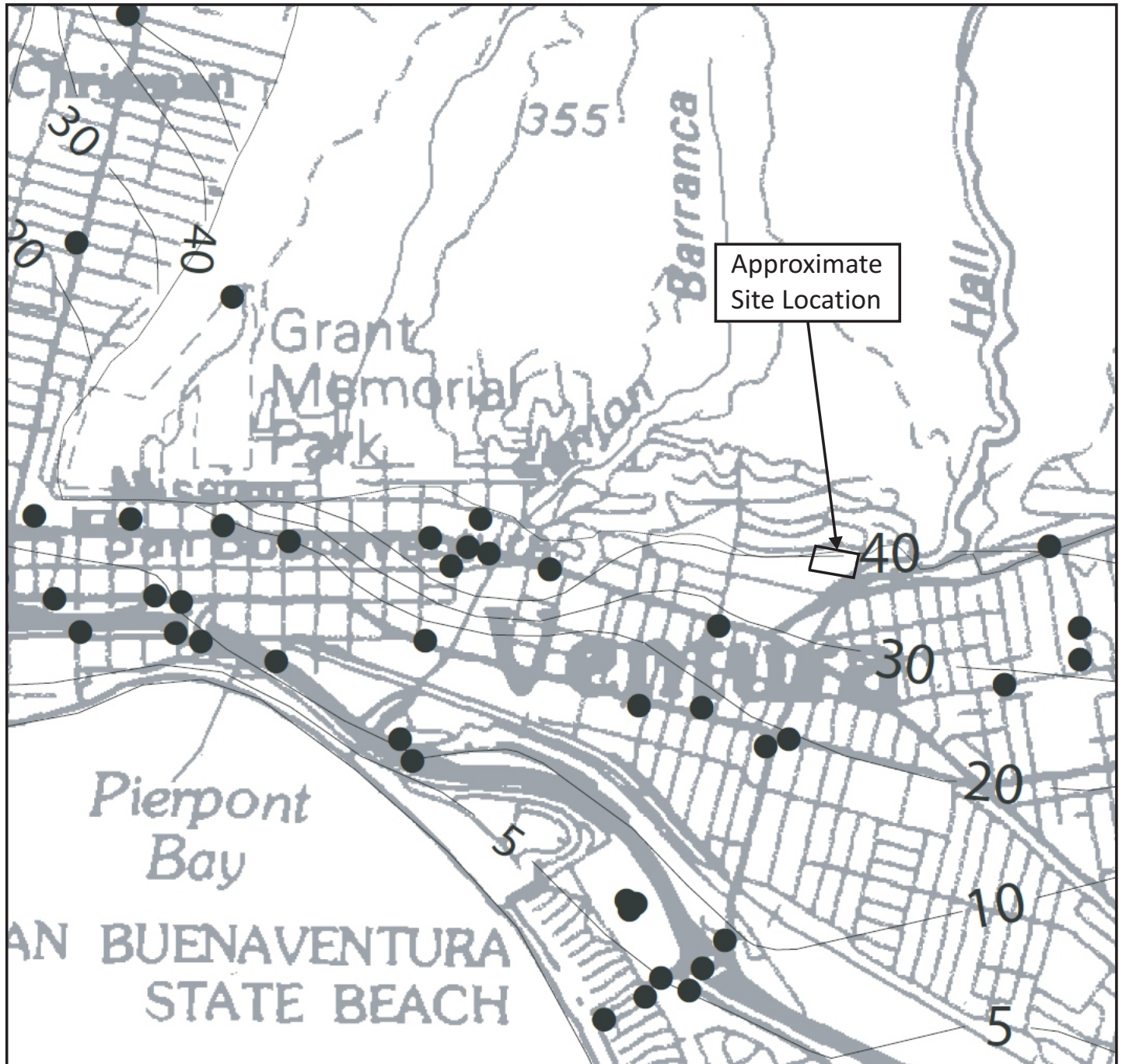
Ventura High School Track and Field
Ventura, California



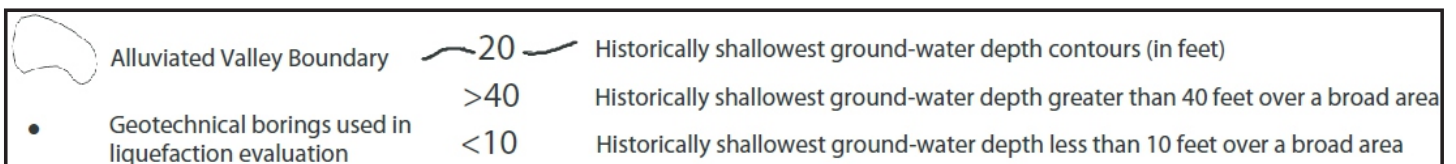
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*Taken from CGS, Seismic Hazard Zone Report For The Ventura 7.5-Minute Quadrangle, Ventura County, California, 2003.



Approximate Scale: 1" = 2,000'

0 2,000' 4,000'



HISTORICAL HIGH GROUNDWATER MAP

Ventura High School Track and Field
Ventura, California



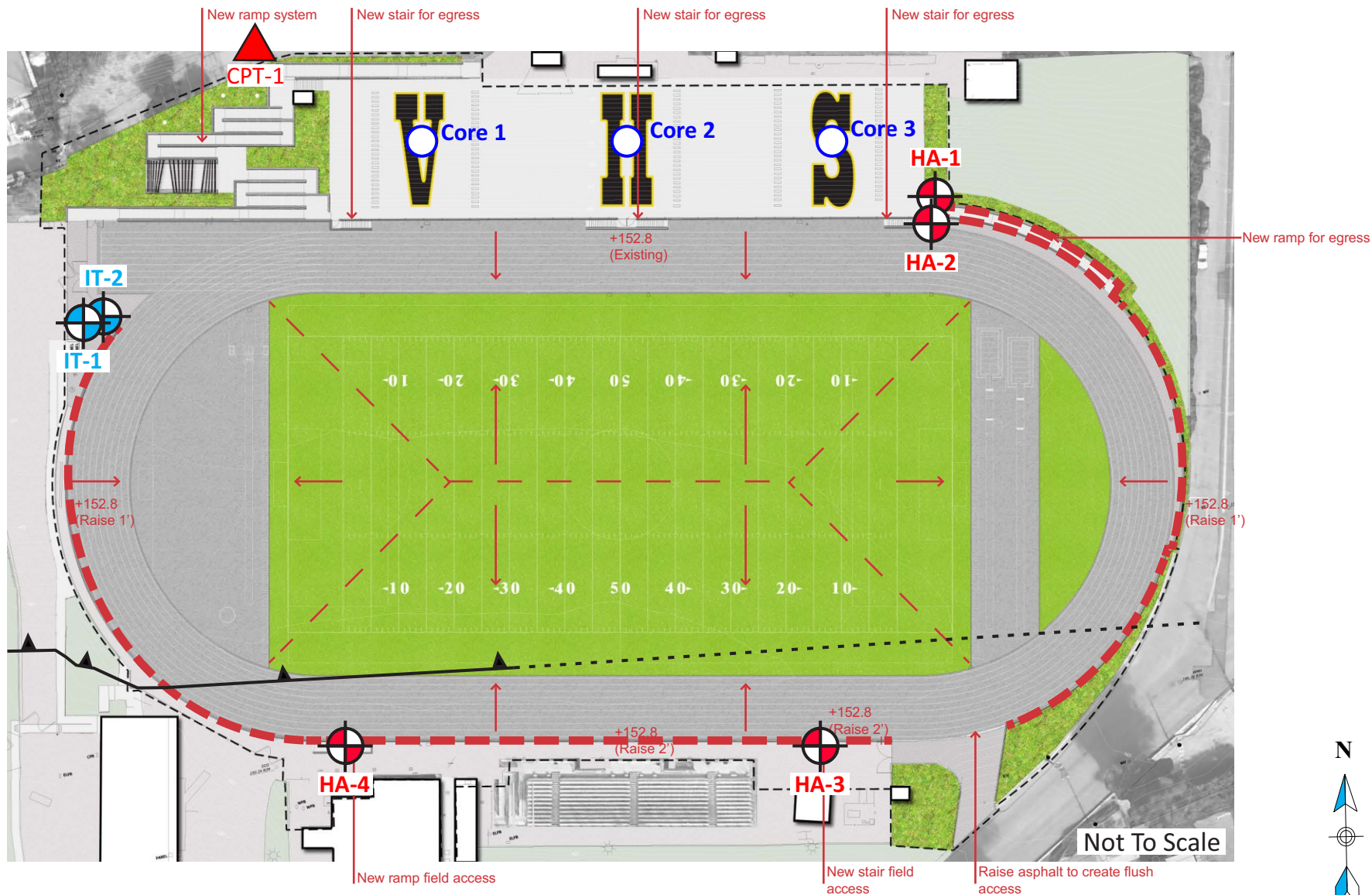
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FIELD STUDY

- A. On March 24, 2025, a total of six (6) borings (HA-1 through HA-4 and IT-1 and IT-2) were hand-augered to depths ranging from 2 to 14 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory analyses, and in which to perform infiltration tests. The approximate locations of the borings were determined in the field by pacing and sighting, see Site Plan in this Appendix.
- B. Samples were obtained within the borings with a Modified California (M.C.) ring sampler (ASTM D 3550 with a shoe similar to ASTM D 1586). The M.C. sampler has a 3-inch outside diameter, and a 2.42-inch inside diameter when used with brass ring liners (as it was during this study). The samples were obtained by driving the sampler with a hand-operated, lightweight, sliding hammer. Due to the sampling method employed, blow counts were not recorded.
- C. Bulk soil samples were collected from the cuttings of the soils encountered in all exploratory borings (HA-1 through HA-4) between depths of zero and 5 feet below the ground surface.
- D. On March 25, 2025, a Cone Penetrometer Test (CPT) sounding was performed to obtain information pertaining to the soil profile. The CPT sounding was performed using equipment owned and operated by Kehoe Testing and Engineering. During advancement of the cone penetrometer, readings of sleeve friction (in tons per square foot), tip resistance (also in tons per square foot), and friction ratio (in percent) were recorded at 0.05-meter intervals as per ASTM D 5778 and ASTM D 3441. The approximate location of the CPT was determined in the field by pacing and sighting and is shown on the Site Plan in this Appendix.
- E. The final logs of the borings represent interpretations of the contents of the field logs obtained during the subsurface study and the results of laboratory testing performed on the samples. The final logs, as well as the interpretations of the CPT sounding, are included in this Appendix.



HA-4



: Approximate Hand Auger Location.

IT-2



: Approximate Infiltration Test Location.



: Fault Surface Intercept (Based on 2001 Study) - Dashed Where Approximated.

CPT-1



: Approximate CPT Sounding Location.

Core 3



: Approximate Concrete Core Location.

SITE PLAN

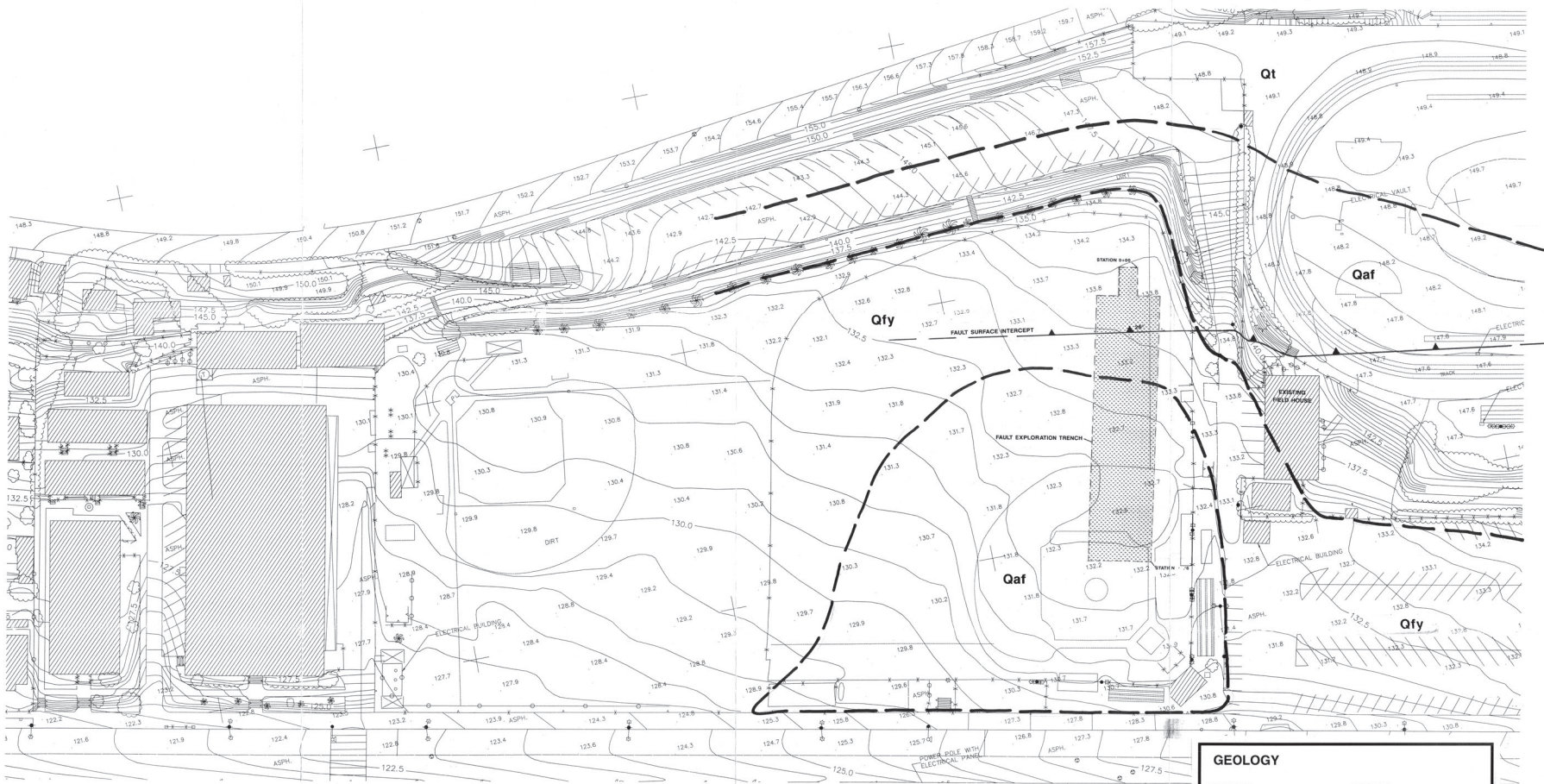
Ventura High School Track & Field
Ventura, California



Earth Systems

August 2025

305811-006



MASTER SITE PLAN - ATHLETIC FIELDS

SCALE: 1"=30'



CLASSROOM BUILDING

VENTURA HIGH SCHOOL

GEOLOGY

ROCK UNITS

- Qaf** - Artificial Fill
- Qfy** - Younger Alluvial Fan
- Qt** - Terrace Deposit

SYMBOLS

- - - - - Approximate Formal Contact
- - - - - Fault Surface Intercept. Dashed Where Approximated
- - - - - Limit Of Fault Exploration Trench



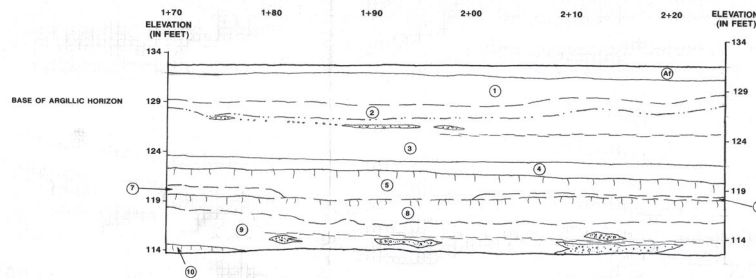
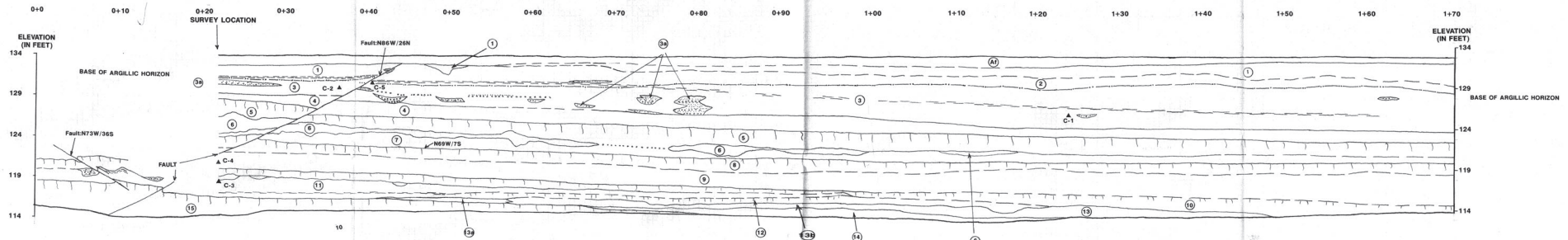
Earth Systems
Southern California
1791-A Miller Street, Ventura, California 93003
PH: (805) 645-6727 FAX: (805) 645-1325

KEY TO SYMBOLS
VENTURA HIGH SCHOOL
FIELD HOUSE
August 2001 25th Fl. VT-2001-001

MARCH 24, 2001

RASMUSSEN & ASSOCIATES
Architecture
Planning
Interiors

FIELD HOUSE FAULT TRENCH
TREND:N14E
EAST TRENCH WALL



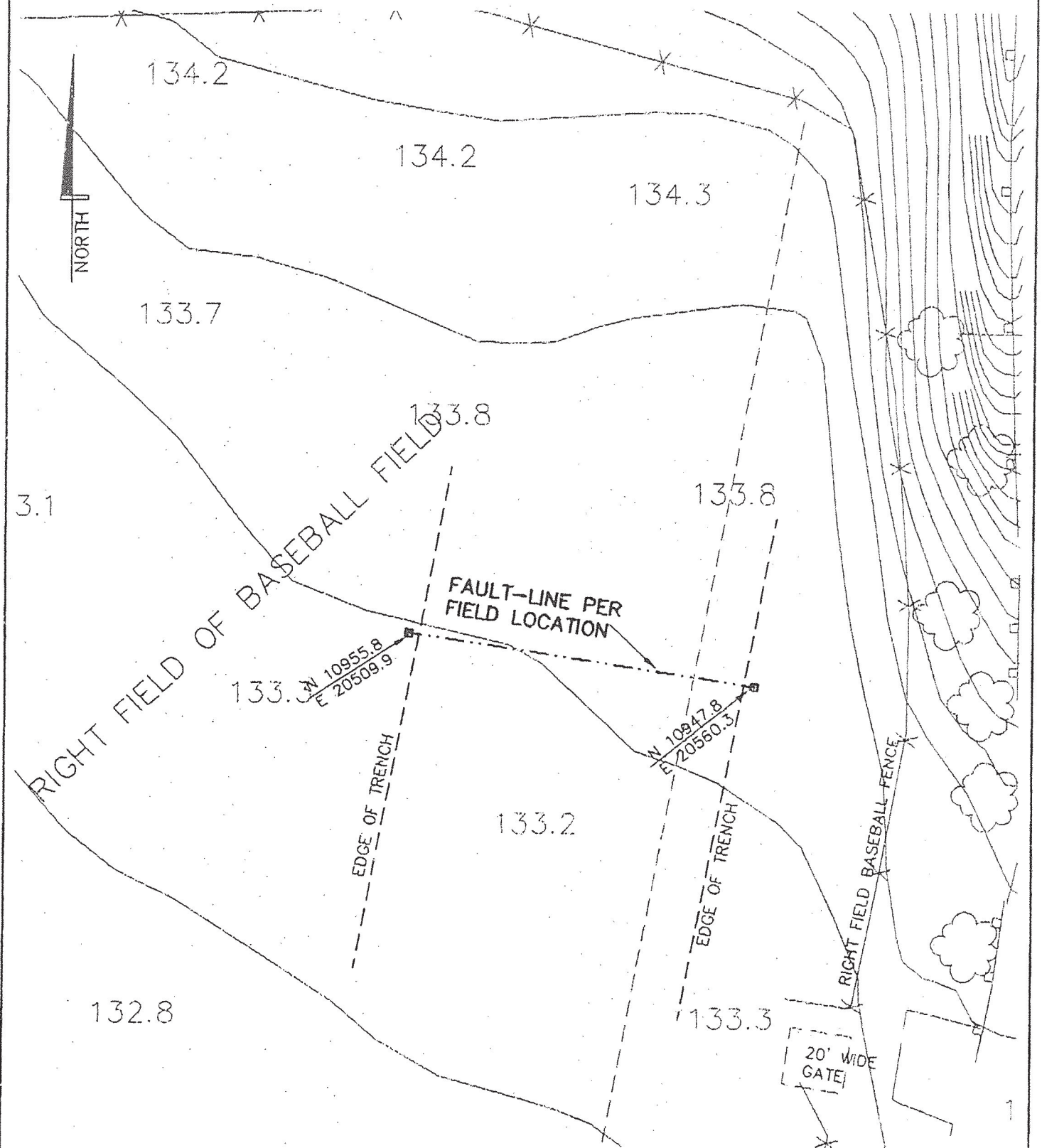
DESCRIPTIONS

Engineering Unit	Symbol	Description
Artificial Fill	(A1)	Brown clayey silty sand; firm, dry, w/ fine gravel clasts to 1" diameter, many fine rootlets, occasional construction debris, distinct planar contact to unit 1.
Soil (A-horizon)	(1)	Light gray-brown clayey silt to silty clay; common to many vesicular pores 1/32" to 1/8" diameter, moderately firm to firm, dry, with occasional fine gravel clasts, common rootlets, moderate induration, undulatory gradational contact to unit 2, common to many root casts and krotovina burrows.
MH	(2)	Slightly red-brown, clayey silt; weak argillic development, slight to moderate induration; massive to thinly bedded, locally contains thinly bedded, discontinuous clay horizons common to many root casts and krotovina, low to medium dense.
SM	(3)	Light brown slightly clayey silt; same as unit 2 except no argillic development, slight to moderate induration; massive to thinly bedded, locally contains thinly bedded, discontinuous clay horizons, common to many root casts and krotovina, low to medium dense.
GP - SW	(3a)	Light brown to gray-brown slightly clayey, very coarse sandy gravel to gravely very coarse sand; loose to slightly indurated, locally exhibits normal grading, dry, sub-angular to sub-rounded clasts, poorly graded clasts, erosional basal contacts, inset lenses and discontinuous gravel logs that locally exhibit normal graded beds.
SM	(4)	Light brown to gray-brown fine-silty sand; locally clayey, laminated to bedded, moderately well-graded, medium dense, moist, locally very coarse sand lenses and stringers with silty clay interbeds, slight erosional contact to 5.
SM	(5)	Dark brown slightly gravely silty-sand, slightly moist to moist, moderate to well-indurated, low-med dense silty-sand matrix, contains few to many krotovina, relict A-horizon (paleosol), few sub-rounded, gravel clasts to 1" diameter, undulatory gradational to abrupt contact to 6.

Engineering Unit	Symbol	Description
GW	(6)	Gray-brown silty to very coarse sandy gravel with occasional boulder clasts to 12" diameter; sub-rounded gravel clasts, matrix-supported, locally clasts supported gravel pods, loose to slightly indurated, few fine rootlets, locally discontinuous erosional basal contact to unit 7.
SM-SW	(7)	Light gray silty-sand; poorly graded, moderately induration, massive, few fine rootlets, many root/animal burrows locally clayey along base and locally bedded, planar erosional contact to 8 with many burrows at base into unit 8.
CL-MH	(8)	Brown silty clay to locally clayey silt; scattered subangular to subrounded gravel clasts, relict A-horizon (paleosol), many animal and root burrows, firm to stiff, well-indurated, few fine rootlets massive, low dense, slightly undulatory gradational contact to unit 9.
CL-MH	(9)	Light gray to gray-brown silty clay to clayey silt; massive to laminated, locally thinly bedded, with thin clay interbeds, occasional subangular-subrounded fine gravel clasts, planar erosional contact to unit 10.
CL-MH	(10)	Brown to light gray-brown silty clay to clayey silt; scattered subangular to sub-rounded fine gravel clasts, common to locally many root/animal burrows, firm-stiff, low-medium dense, moderate-well induration, gradational contact to unit 11.
CL-MH	(11)	Light brown to gray-brown silty clay to clayey silt; low-medium dense, loose to locally firm where clayey, few root/animal burrows, laminated to thinly bedded basal gray-green clay unit, conformably overlies units 12 and 15, contains inset sandy to fine gravel lenses.
CL-MH	(12)	Same as 11 except silty clay without gravel clasts, discontinuous, stiff.
GW	(13a)	Light gray silty very coarse sandy fine gravel; locally clayey, massive to bedded, subrounded to rounded clasts, some of which exhibit oxidized staining, locally exhibits normal grading, moderate induration, dense to medium dense, planar erosional contact to unit 14, laterally grades to unit 13b.
GW-GP	(13b)	Same as above except clayey gravel with clasts to 2" diameter, locally thin, occasional sub-angular clasts, massive with locally coarse gravel stringers.
CL-CH	(14)	Gray-green clay, massive, moist, low-medium dense firm-stiff, conformable contact to unit 15.
CL-SC	(15)	Dark brown silty-clay, stiff, scattered fine sub-angular gravel clasts, few-many root/animal burrows, massive, locally mottled red-brown.

SCALE 1" = 5' (HORIZONTAL = VERTICAL)

EXHIBIT "B"

F
J
CONSULTING
AND

(805) 501-4075

fjs@fjslandconsulting.com

FJS Land Consulting
594 Stoney Peak Ct.
Simi Valley, CA 93065

Fax (805) 583-3710

VENTURA HIGH SCHOOL
Field Fault Line Location
CITY OF VENTURA
VUSD

DRAWN BY: FJS

CHECKED BY: FJS

DATE: 7/3/2001

SCALE: 1" = 20'

SHT 2 OF 2

C:\FJS\PDATA\00-061\AUTOCAD\EXHIBITS\FAULT-LINE-BASEBALL.DWG

Logs of Borings



BORING NO: HA-2									DRILLING DATE: March 24, 2025	
PROJECT NAME: Ventura High School Track & Field									DRILL RIG:	
PROJECT NUMBER: 305811-006									DRILLING METHOD: Hand Auger	
BORING LOCATION: Per Plan									LOGGED BY: V. Wallace	
Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS	
	Bulk	SPT	Mod. Calif.							
0									2" Asphaltic Concrete; No Base	
						SM	112.2	10	ARTIFICIAL FILL: Yellowish brown silty sand, trace gravel, medium dense, moist	
5							108.1	9.2	ALLUVIUM: Yellowish brown silty sand, trace gravel, medium dense, moist	
						SM	148.5	9.7		
10							110.5	4.4		
									Total Depth: 10.5 Feet	
15									No Groundwater Encountered	
20										
25										
30										
35										

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

BORING NO: HA-3									DRILLING DATE: March 24, 2025	
PROJECT NAME: Ventura High School Track & Field									DRILL RIG:	
PROJECT NUMBER: 305811-006									DRILLING METHOD: Hand Auger	
BORING LOCATION: Per Plan									LOGGED BY: V. Wallace	
Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS	
	Bulk	SPT	Mod. Calif.							
0									2" Asphaltic Concrete; No Base	
						SM	116.9	9.8	ARTIFICIAL FILL: Yellowish brown silty sand, medium dense, moist	
5						ML	102.4	11.1	ARTIFICIAL FILL: Yellowish brown sandy silt, little cobbles, stiff, moist	
						SM	147.1	11.1	ARTIFICIAL FILL: Yellowish brown silty fine sand, medium dense, moist	
10						SM	111.1	5.9	ARTIFICIAL FILL: Yellowish brown silty fine sand, medium dense, moist	
						SM			ALLUVIUM: Yellowish brown silty fine sand, medium dense, moist	
15						SM			ALLUVIUM: Yellowish brown gravelly fine to coarse sand, little silt, medium dense, moist	
									Total Depth: 14.0 Feet; Refusal	
20									No Groundwater Encountered	
25										
30										
35										

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

BORING NO: HA-4								DRILLING DATE: March 24, 2025	
PROJECT NAME: Ventura High School Track & Field								DRILL RIG:	
PROJECT NUMBER: 305811-006								DRILLING METHOD: Hand Auger	
BORING LOCATION: Per Plan								LOGGED BY: V. Wallace	
Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
	Bulk	SPT	Mod. Calif.						
0									2" Asphaltic Concrete; No Base
5						ML	116.3	7.7	ARTIFICIAL FILL: Yellowish brown sandy silt, some cobbles, little gravel, stiff, moist
10									Total Depth: 5.5 Feet; Refusal No Groundwater Encountered
15									
20									
25									
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Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

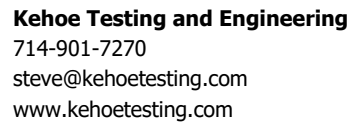
BORING NO: IT-1 PROJECT NAME: Ventura High School Track & Field PROJECT NUMBER: 305811-006 BORING LOCATION: Per Plan								DRILLING DATE: March 24, 2025 DRILL RIG: DRILLING METHOD: Hand Auger LOGGED BY: V. Wallace	
Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
	Bulk	SPT	Mod. Calif.						
0						GM			2" Asphaltic Concrete; No Base ARTIFICIAL FILL: Yellowish brown sandy gravel, some cobbles, little silt, medium dense, moist
5									Total Depth: 3.0 Feet; Refusal No Groundwater Encountered
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Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

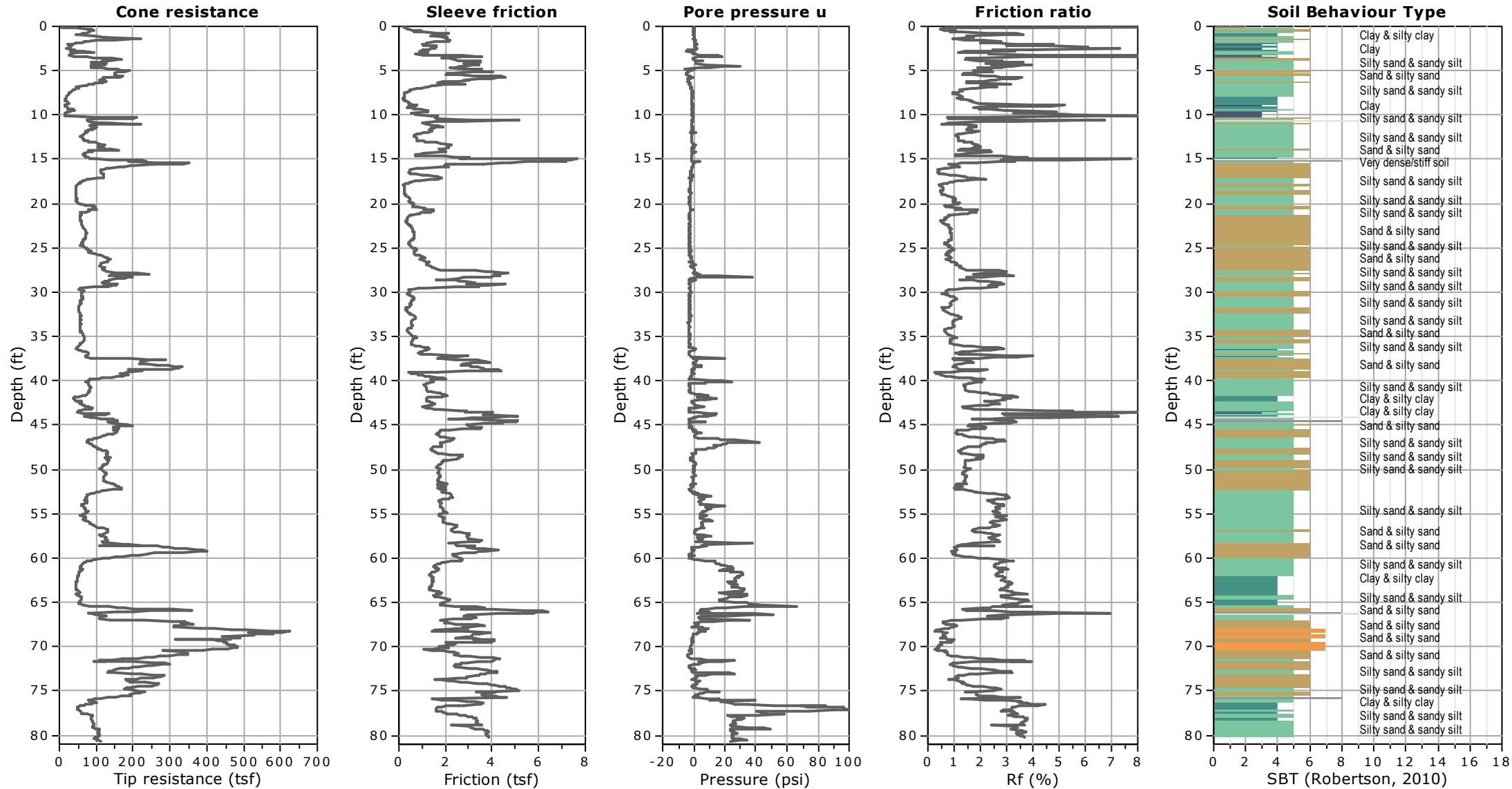
BORING NO: IT-2 PROJECT NAME: Ventura High School Track & Field PROJECT NUMBER: 305811-006 BORING LOCATION: Per Plan								DRILLING DATE: March 24, 2025 DRILL RIG: DRILLING METHOD: Hand Auger LOGGED BY: V. Wallace	
Vertical Depth	Sample Type			PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
Bulk	SPT	Mod. Calif.							
0						GM			2" Asphaltic Concrete; No Base ARTIFICIAL FILL: Yellowish brown sandy gravel, some cobbles, little silt, medium dense, moist
5									Total Depth: 2.0 Feet; Refusal No Groundwater Encountered
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Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Log of CPT Sounding and Interpretation



Total depth: 80.61 ft, Date: 3/25/2025



Project: Ventura HS Track and Field

Project No: 305811-006

Date: 03/25/25

CPT SOUNDING: CPT-1				Plot: 1		Density: 1		SPT N		Program developed 2003 by Shelton L. Stringer, GE, Earth Systems Southwest											
Est. GWT (feet): 50.0				Shift SBT: 0		Dr correlation: 0		Baldi		Qc/N: 0		Jefferies & Davies				Phi Correlation: 4		SPT N			
Base	Base	Avg	Avg				Est.	Qc		Total						Clean	Clean	Rel.	Nk:		
Depth	Depth	Tip	Friction	Soil		Density or	Density	to	SPT	po					Norm.	Sand	Sand	Dens.	Phi		
meters	feet	Qc, tsf	Ratio, %	Classification	USCS	Consistency	(pcf)	N	N(60)	tsf	tsf	F	n	Cq	Qc1n	lc	Qc1n	N1(60)	N1(60)		
																		Dr (%)	(deg.)		
																		Su (tsf)	OCR		
0.15	0.5	82.33	0.90	Sand to Silty Sand	SP/SM	medium dense	100	5.5	15	0.013	0.013	0.90	0.54	1.70	132.3	1.79	145.4	25	29	88	35
0.30	1.0	72.46	2.57	Sandy Silt to Clayey Silt ML		medium dense	110	4.8	15	0.039	0.039	2.57	0.65	1.70	116.4	2.15	181.1	26	36	83	35
0.46	1.5	131.16	1.84	Silty Sand to Sandy Silt SM/ML		dense	110	5.3	25	0.066	0.066	1.84	0.57	1.70	210.8	1.88	246.1	42	49	100	39
0.61	2.0	45.79	3.03	Sandy Silt to Clayey Silt ML		medium dense	110	4.4	10	0.094	0.094	3.04	0.71	1.70	73.6	2.34	153.0	18	31	64	32
0.76	2.5	31.74	4.80	Silty Clay to Clay	CL	medium dense	110	3.9	8	0.121	0.121	4.82	0.79	1.70	51.0	2.59	167.7	14	34	49	31
0.91	3.0	56.13	2.15	Sandy Silt to Clayey Silt ML		medium dense	110	4.8	12	0.149	0.149	2.15	0.66	1.70	90.2	2.17	143.9	20	29	73	33
1.07	3.5	74.89	4.72	Overconsolidated Soil ??		medium dense	110	4.4	17	0.176	0.176	4.73	0.71	1.70	120.3	2.35	255.1	29	51	85	36
1.22	4.0	123.94	2.67	Silty Sand to Sandy Silt SM/ML		dense	110	5.0	25	0.204	0.204	2.68	0.61	1.70	199.1	2.02	264.6	42	53	100	39
1.37	4.5	98.42	3.10	Sandy Silt to Clayey Silt ML		dense	110	4.8	20	0.231	0.231	3.11	0.65	1.70	158.1	2.13	240.0	35	48	96	37
1.52	5.0	157.92	2.03	Silty Sand to Sandy Silt SM/ML		very dense	110	5.3	30	0.259	0.259	2.04	0.57	1.70	253.7	1.86	293.4	50	59	100	41
1.68	5.5	153.06	2.07	Silty Sand to Sandy Silt SM/ML		dense	110	5.3	29	0.286	0.286	2.08	0.57	1.70	245.9	1.88	287.6	49	58	100	40
1.83	6.0	122.93	2.32	Silty Sand to Sandy Silt SM/ML		dense	110	5.1	24	0.314	0.314	2.33	0.60	1.70	197.5	1.97	250.5	41	50	100	39
1.98	6.5	79.21	2.38	Silty Sand to Sandy Silt SM/ML		medium dense	110	4.9	16	0.341	0.341	2.39	0.64	1.70	127.3	2.10	185.7	28	37	87	35
2.13	7.0	43.32	1.79	Silty Sand to Sandy Silt SM/ML		medium dense	110	4.7	9	0.369	0.369	1.81	0.67	1.70	69.6	2.20	115.6	15	23	62	32
2.29	7.5	27.16	1.06	Silty Sand to Sandy Silt SM/ML		loose	110	4.7	6	0.396	0.396	1.08	0.67	1.70	43.6	2.22	74.7	9	15	42	30
2.44	8.0	17.61	1.29	Sandy Silt to Clayey Silt ML		loose	110	4.3	4	0.424	0.424	1.32	0.74	1.70	28.3	2.42	68.1	6	14	24	29
2.59	8.5	20.45	1.82	Sandy Silt to Clayey Silt ML		loose	110	4.2	5	0.451	0.451	1.86	0.75	1.70	32.9	2.46	84.1	7	17	31	29
2.74	9.0	17.66	3.55	Silty Clay to Clay	CL	very stiff	110	3.7	5	0.479	0.479	3.65	0.82	1.70	28.4	2.69	5				
2.90	9.5	33.47	3.19	Clayey Silt to Silty Clay ML/CL		medium dense	110	4.2	8	0.506	0.506	3.24	0.75	1.70	53.8	2.45	136.8	11	27	51	30
3.05	10.0	35.58	3.85	Clayey Silt to Silty Clay ML/CL		medium dense	110	4.1	9	0.534	0.534	3.91	0.76	1.68	56.5	2.50	155.5	12	31	53	31
3.20	10.5	122.23	2.95	Sandy Silt to Clayey Silt ML		dense	110	4.9	25	0.561	0.561	2.97	0.64	1.50	173.0	2.09	249.6	33	50	100	37
3.35	11.0	128.54	1.17	Sand to Silty Sand	SP/SM	dense	100	5.5	23	0.588	0.588	1.17	0.55	1.38	167.6	1.79	184.6	31	37	98	36
3.51	11.5	93.21	1.62	Silty Sand to Sandy Silt SM/ML		medium dense	110	5.1	18	0.614	0.614	1.63	0.61	1.39	122.6	1.99	157.9	23	32	85	34
3.66	12.0	71.56	1.55	Silty Sand to Sandy Silt SM/ML		medium dense	110	5.0	14	0.641	0.641	1.56	0.63	1.37	92.7	2.06	129.0	18	26	74	33
3.81	12.5	61.76	1.14	Silty Sand to Sandy Silt SM/ML		medium dense	110	5.0	12	0.669	0.669	1.15	0.62	1.33	77.6	2.03	104.6	15	21	66	32
3.96	13.0	79.31	1.37	Silty Sand to Sandy Silt SM/ML		medium dense	110	5.1	16	0.696	0.696	1.38	0.61	1.29	96.9	2.01	127.5	19	26	76	33
4.11	13.5	117.41	1.81	Silty Sand to Sandy Silt SM/ML		medium dense	110	5.1	23	0.724	0.724	1.82	0.60	1.26	139.6	1.99	179.1	27	36	91	35
4.27	14.0	121.77	1.65	Sand to Silty Sand	SP/SM	medium dense	100	5.2	24	0.750	0.750	1.66	0.59	1.23	141.2	1.95	175.6	27	35	91	35
4.42	14.5	68.53	1.44	Silty Sand to Sandy Silt SM/ML		medium dense	110	4.9	14	0.776	0.776	1.45	0.64	1.22	78.9	2.09	113.9	16	23	67	32
4.57	15.0	122.59	3.39	Sandy Silt to Clayey Silt ML		medium dense	110	4.7	26	0.804	0.804	3.41	0.67	1.20	139.2	2.20	231.4	29	46	91	36
4.72	15.5	265.22	1.93	Sand to Silty Sand	SP/SM	very dense	100	5.4	49	0.830	0.830	1.94	0.55	1.14	286.7	1.82	320.6	53	64	100	41
4.88	16.0	147.79	0.78	Sand	SP	medium dense	100	5.7	26	0.855	0.855	0.79	0.52	1.12	156.0	1.70	161.4	28	32	95	36
5.03	16.5	115.76	0.54	Sand	SP	medium dense	100	5.7	20	0.880	0.880	0.54	0.51	1.10	120.3	1.69	123.7	22	25	84	34
5.18	17.0	106.43	1.57	Sand to Silty Sand	SP/SM	medium dense	100	5.1	21	0.905	0.905	1.59	0.61	1.10	110.7	2.01	145.8	22	29	81	34
5.33	17.5	60.26	1.06	Silty Sand to Sandy Silt SM/ML		medium dense	110	4.9	12	0.931	0.931	1.08	0.64	1.08	61.8	2.09	89.3	13	18	57	31
5.49	18.0	45.90	0.50	Sand to Silty Sand	SP/SM	loose	100	5.0	9	0.958	0.958	0.51	0.62	1.06	46.1	2.03	61.9	9	12	45	30
5.64	18.5	45.00	0.52	Sand to Silty Sand	SP/SM	loose	100	5.0	9	0.983	0.983	0.53	0.63	1.05	44.6	2.05	61.3	9	12	43	30
5.79	19.0	43.80	0.64	Silty Sand to Sandy Silt SM/ML		loose	110	4.9	9	1.009	1.009	0.66	0.64	1.03	42.7	2.11	63.0	9	13	42	30
5.94	19.5	45.29	1.02	Silty Sand to Sandy Silt SM/ML		loose	110	4.7	10	1.036	1.036	1.04	0.67	1.01	43.4	2.21	73.6	10	15	42	30
6.10	20.0	58.11	0.98	Sand to Silty Sand	SP/SM	medium dense	100	4.9	12	1.063	1.063	0.99	0.64	1.00	54.8	2.12	81.4	12	16	52	31
6.25	20.5	94.93	1.00	Sand to Silty Sand	SP/SM	medium dense	100	5.2	18	1.088	1.088	1.02	0.60	0.98	88.3	1.96	110.1	18	22	72	32
6.40	21.0	70.18	1.64	Silty Sand to Sandy Silt SM/ML		medium dense	110	4.7	15	1.114	1.114	1.67	0.67	0.97	64.1	2.20	107.1	14	21	58	31
6.55	21.5	56.29	0.73	Sand to Silty Sand	SP/SM	medium dense	100	4.9	11	1.140	1.140	0.75	0.63	0.95	50.7	2.08	71.7	11	14	49	30
6.71	22.0	59.27	0.57	Sand to Silty Sand	SP/SM	medium dense	100	5.1	12	1.165	1.165	0.58	0.61	0.94	52.8	2.00	68.9	11	14	50	30
6.86	22.5	62.27	0.76	Sand to Silty Sand	SP/SM	medium dense	100	5.0	13	1.190	1.190	0.77	0.63	0.93	54.7	2.06	75.4	11	15	52	30
7.01	23.0	68.98	0.87	Sand to Silty Sand	SP/SM	medium dense	100	5.0	14	1.215	1.215	0.88	0.63	0.92	59.8	2.06	82.5	13	17	55	31
7.16	23.5	72.48	0.89	Sand to Silty Sand	SP/SM	medium dense	100	5.0	15	1.240	1.240	0.90	0.62	0.91	62.0	2.05	84.9	13	17	57	31
7.32	24.0	67.79	0.88	Sand to Silty Sand	SP/SM	medium dense	100	4.9	14	1.265	1.265	0.89	0.63	0.89	57.2	2.07	80.7	12	16	54	31
7.47	24.5	62.47	0.70	Sand to Silty Sand	SP/SM	medium dense	100	5.0	13	1.290	1.290	0.72	0.63	0.88	52.1	2.06	72.0	11	14	50	30
7.62	25.0	67.94	0.88	Sand to Silty Sand	SP/SM	medium dense	100	4.9	14	1.315	1.315	0.90	0.64	0.87	55.9	2.08	79.7	12	16	53	

Project: Ventura HS Track and Field

Project No: 305811-006

Date: 03/25/25

CPT SOUNDING: CPT-1				Plot: 1	Density: 1	SPT N	Program developed 2003 by Shelton L. Stringer, GE, Earth Systems Southwest														
Est. GWT (feet): 50.0				Shift SBT: 0	Dr correlation: 0	Baldi	Qc/N: 0	Jefferies & Davies							Phi Correlation: 4					SPT N	
Base	Base	Avg	Avg			Est.	Qc	Total								Clean	Clean	Rel.		Nk:	
Depth	Depth	Tip	Friction	Soil	Density or	Density	to	SPT	po	p'o	F	n	Cq	Norm.	Sand	Sand	Dens.	Phi	Su		
meters	feet	Qc, tsf	Ratio, %	Classification	Consistency	(pcf)	N	N(60)	tsf	tsf				Qc1n	Qc1n	N1(60)	N1(60)	Dr (%)	(deg.)	(tsf)	
														2.6						OCR	
13.11	43.0	79.61	1.59	Silty Sand to Sandy Silt	SM/ML	medium dense	110	4.5	18	2.254	2.254	1.63	0.71	0.58	44.0	2.32	88.9	12	18	43	31
13.26	43.5	59.36	5.22	Clay	CL/CH	hard	110	3.5	17	2.281	2.281	5.43	0.85	0.52	29.1	2.80	17			3.36	7.5
13.41	44.0	92.44	4.83	Overconsolidated Soil	??	hard	110	3.9	24	2.309	2.309	4.95	0.80	0.54	46.8	2.63	24			5.30	11.7
13.56	44.5	138.91	2.85	Silty Sand to Sandy Silt	SM/ML	medium dense	110	4.5	31	2.336	2.336	2.89	0.71	0.57	75.0	2.32	150.5	20	30	65	33
13.72	45.0	165.50	2.11	Silty Sand to Sandy Silt	SM/ML	medium dense	110	4.8	35	2.364	2.364	2.14	0.66	0.59	92.1	2.16	145.2	23	29	73	34
13.87	45.5	158.59	1.67	Sand to Silty Sand	SP/SM	medium dense	100	4.9	32	2.390	2.390	1.70	0.64	0.59	88.8	2.10	129.5	21	26	72	34
14.02	46.0	144.94	1.17	Sand to Silty Sand	SP/SM	medium dense	100	5.0	29	2.415	2.415	1.19	0.62	0.60	82.1	2.03	109.6	19	22	69	33
14.17	46.5	102.41	2.08	Silty Sand to Sandy Silt	SM/ML	medium dense	110	4.4	23	2.441	2.441	2.13	0.71	0.55	53.4	2.33	109.7	15	22	51	32
14.33	47.0	81.01	2.37	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.2	19	2.470	2.470	2.44	0.75	0.53	40.5	2.46	104.4	12	21	39	31
14.48	47.5	102.32	1.49	Sand to Silty Sand	SP/SM	medium dense	120	4.6	22	2.500	2.500	1.53	0.68	0.56	53.7	2.24	94.4	14	19	51	31
14.63	48.0	127.38	1.30	Sand to Silty Sand	SP/SM	medium dense	120	4.9	26	2.530	2.530	1.33	0.65	0.57	68.5	2.12	101.7	16	20	61	32
14.78	48.5	128.46	2.00	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.6	28	2.560	2.560	2.04	0.69	0.54	66.2	2.25	119.0	17	24	60	32
14.94	49.0	130.95	1.53	Sand to Silty Sand	SP/SM	medium dense	120	4.8	27	2.590	2.590	1.56	0.66	0.55	68.5	2.16	108.0	17	22	61	32
15.09	49.5	120.73	1.44	Sand to Silty Sand	SP/SM	medium dense	120	4.7	25	2.620	2.620	1.47	0.67	0.55	62.4	2.17	100.2	16	20	57	32
15.24	50.0	116.50	1.49	Sand to Silty Sand	SP/SM	medium dense	120	4.7	25	2.650	2.650	1.52	0.67	0.54	59.4	2.20	99.0	15	20	55	32
15.39	50.5	124.77	1.38	Sand to Silty Sand	SP/SM	medium dense	120	4.8	26	2.680	2.664	1.41	0.66	0.54	64.1	2.15	100.2	16	20	58	32
15.54	51.0	119.68	1.42	Sand to Silty Sand	SP/SM	medium dense	120	4.7	25	2.710	2.679	1.45	0.67	0.54	60.9	2.18	98.6	15	20	56	32
15.70	51.5	136.57	1.25	Sand to Silty Sand	SP/SM	medium dense	120	4.9	28	2.740	2.693	1.27	0.64	0.55	70.9	2.09	102.2	17	20	63	32
15.85	52.0	159.26	1.13	Sand	SP	medium dense	120	5.1	31	2.770	2.708	1.15	0.61	0.56	84.5	2.01	110.5	19	22	70	33
16.00	52.5	113.52	1.85	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.5	25	2.800	2.722	1.89	0.70	0.52	55.5	2.28	105.4	15	21	52	32
16.15	53.0	77.04	2.92	Sandy Silt to Clayey Silt	ML	medium dense	120	4.0	19	2.830	2.736	3.03	0.79	0.47	34.5	2.57	109.5	12	22	33	31
16.31	53.5	70.84	2.59	Sandy Silt to Clayey Silt	ML	medium dense	120	4.0	18	2.860	2.751	2.70	0.78	0.47	31.6	2.57	99.6	11	20	29	30
16.46	54.0	65.89	2.73	Sandy Silt to Clayey Silt	ML	hard	120	3.9	17	2.890	2.765	2.86	0.80	0.46	28.9	2.62	17			3.71	6.8
16.61	54.5	69.74	2.64	Sandy Silt to Clayey Silt	ML	medium dense	120	3.9	18	2.920	2.780	2.75	0.79	0.47	30.7	2.59	99.6	11	20	28	30
16.76	55.0	70.96	2.64	Sandy Silt to Clayey Silt	ML	medium dense	120	3.9	18	2.950	2.794	2.75	0.79	0.47	31.2	2.58	100.2	11	20	28	30
16.92	55.5	64.65	2.87	Sandy Silt to Clayey Silt	ML	hard	120	3.8	17	2.980	2.808	3.01	0.81	0.45	27.8	2.64	17			3.64	6.6
17.07	56.0	77.98	2.55	Sandy Silt to Clayey Silt	ML	medium dense	120	4.0	19	3.010	2.823	2.65	0.77	0.47	34.5	2.54	102.1	12	20	33	30
17.22	56.5	100.46	2.41	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.3	24	3.040	2.837	2.48	0.74	0.48	45.7	2.43	110.7	14	22	44	31
17.37	57.0	130.89	1.82	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.6	28	3.070	2.852	1.86	0.68	0.51	62.7	2.24	111.1	17	22	57	32
17.53	57.5	119.48	2.50	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.3	27	3.100	2.866	2.56	0.73	0.49	54.8	2.38	121.7	16	24	52	32
17.68	58.0	125.11	2.52	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.4	29	3.130	2.880	2.58	0.72	0.49	57.4	2.36	124.8	17	25	54	32
17.83	58.5	210.21	1.33	Sand	SP	medium dense	120	5.1	41	3.160	2.895	1.35	0.60	0.54	108.1	1.97	136.8	24	27	80	34
17.98	59.0	365.34	1.02	Sand	SP	dense	120	5.7	64	3.190	2.909	1.03	0.52	0.59	204.0	1.69	210.7	38	42	100	38
18.14	59.5	249.57	1.07	Sand	SP	medium dense	120	5.4	46	3.220	2.924	1.08	0.56	0.56	132.8	1.84	151.0	27	30	89	35
18.29	60.0	125.37	2.21	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.4	28	3.250	2.938	2.27	0.71	0.48	57.3	2.33	116.8	16	23	54	32
18.44	60.5	68.57	2.83	Sandy Silt to Clayey Silt	ML	hard	120	3.8	18	3.280	2.952	2.97	0.80	0.44	28.4	2.63	18			3.86	6.6
18.59	61.0	57.83	2.74	Sandy Silt to Clayey Silt	ML	hard	120	3.7	16	3.310	2.967	2.90	0.82	0.43	23.4	2.69	16			3.23	5.5
18.75	61.5	55.18	2.76	Sandy Silt to Clayey Silt	ML	hard	120	3.7	15	3.340	2.981	2.94	0.83	0.42	22.1	2.72	15			3.07	5.2
18.90	62.0	50.35	2.75	Sandy Silt to Clayey Silt	ML	hard	120	3.6	14	3.370	2.996	2.95	0.84	0.42	19.8	2.75	14			2.79	4.7
19.05	62.5	49.07	2.94	Sandy Silt to Clayey Silt	ML	hard	120	3.5	14	3.400	3.010	3.15	0.85	0.41	19.0	2.78	14			2.71	4.6
19.20	63.0	44.85	3.12	Sandy Silt to Clayey Silt	ML	hard	120	3.4	13	3.430	3.024	3.38	0.87	0.40	17.0	2.84	13			2.46	4.1
19.35	63.5	46.49	2.96	Sandy Silt to Clayey Silt	ML	hard	120	3.5	13	3.460	3.039	3.20	0.86	0.40	17.7	2.81	13			2.56	4.2
19.51	64.0	50.58	3.37	Clayey Silt to Silty Clay	ML/CL	hard	120	3.5	15	3.490	3.053	3.62	0.86	0.40	19.2	2.82	15			2.80	4.6
19.66	64.5	58.58	3.09	Sandy Silt to Clayey Silt	ML	hard	120	3.6	16	3.520	3.068	3.29	0.84	0.41	22.7	2.74	16			3.27	5.4
19.81	65.0	53.86	3.59	Clayey Silt to Silty Clay	ML/CL	hard	120	3.5	15	3.550	3.082	3.85	0.86	0.40	20.3	2.82	15			2.99	4.9
19.96	65.5	109.07	2.96	Sandy Silt to Clayey Silt	ML	medium dense	120	4.1	26	3.580	3.096	3.06	0.76	0.44	45.6	2.49	123.5	15	25	44	32
20.12	66.0	218.73	2.59	Silty Sand to Sandy Silt	SM/ML	medium dense	120	4.7	47	3.610	3.111	2.64	0.67	0.48	100.1	2.20	167.5	26	34	77	35
20.27	66.5	105.37	3.50	Sandy Silt to Clayey Silt	ML	medium dense	120	4.0	26	3.640	3.125	3.62	0.78	0.43	42.8	2.56	131.8	15	26	42	32
20.42	67.0	163.59	1.64	Sand to Silty Sand	SP/SM	medium dense	120	4.8	34	3.670	3.140	1.67	0.66	0.49	75.6	2.15	117.2	19	23	65	33
20.57	67.5	304.22	0.78	Gravelly Sand to Sand	SW	dense	120	5.8	59	3.700	3.154	0.79	0.51	0.58	185.0	1.64	184.3	33	37	100	37
20.73	68.0	442.96	0.60	Gravelly Sand to Sand	SW	dense	120	6.1	73	3.730	3.168	0.60	0.50	0.58	241.9	1.48	241.9	41	48	100	39
20.88	68.5	566.34	0.61	Gravelly Sand to Sand	SW	very dense	120	6.2	91	3.760	3.183	0.62	0.50	0.58	308.6	1.41	308.6	51	62	100	



CPT No : CPT-1

Project Name: Ventura HS Track and Field

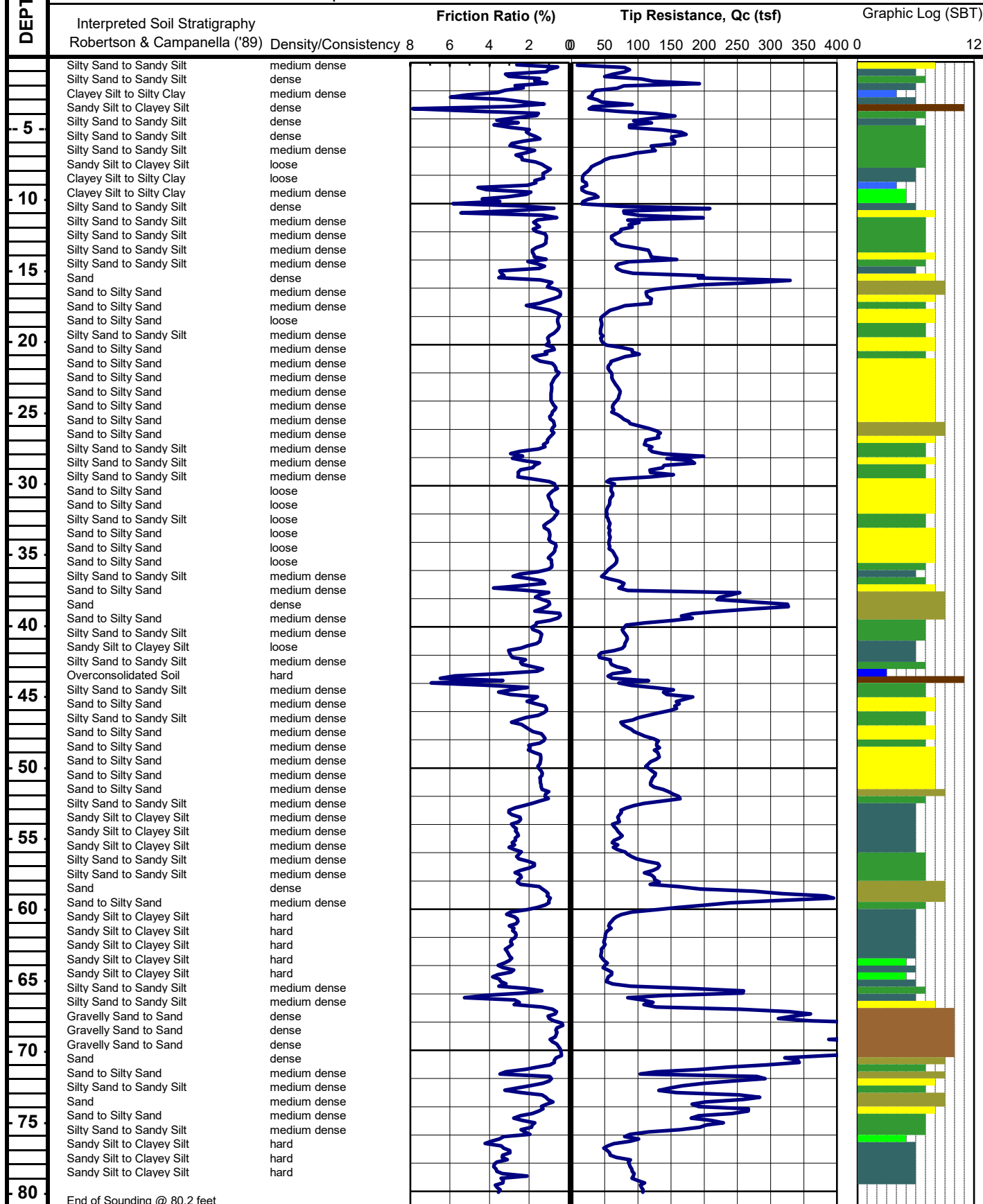
Project No.: 305811-006

Location: See Site Exploration Plan

Cone Penetrometer: **Kehoe Testing**

Truck Mounted Electric Cone
with 23-ton reaction weight

Date: 3/25/2025



End of Sounding @ 80.2 feet

APPENDIX B

Laboratory Testing
Tabulated Laboratory Test Results
Individual Laboratory Test Results

LABORATORY TESTING

- A. Samples were reviewed along with field logs to determine which would be analyzed further. Those chosen for laboratory analyses were considered representative of soils that would be exposed and/or used during grading, and those deemed to be within the influence of proposed construction. The test results are presented in graphic and tabular form in this Appendix.
- B. In-situ moisture content and dry unit weight for the ring samples were determined in general accordance with ASTM D 2937.
- C. Maximum density tests were performed to estimate the moisture-density relationship of typical soil materials. The tests were performed in accordance with ASTM D 1557.
- D. The relative strength characteristics of soils were determined from the results of direct shear tests on two remolded samples. Test specimens were placed in contact with water at least 24 hours before testing and then sheared under normal loads ranging from 1 to 3 ksf in general accordance with ASTM D 3080.
- E. Settlement characteristics were developed from the results of two one-dimensional consolidation/hydrocollapse test performed in general accordance with ASTM D 2435. The sample was incrementally loaded to its approximate overburden pressure and then flooded with water. After monitoring for collapse, the sample was incrementally loaded up to 4 ksf. The sample was allowed to consolidate under each load increment. Rebound was measured under reverse alternate loading. Compression was measured by dial gauges accurate to 0.0001 inch. Results of the consolidation tests are presented in this Appendix in the form of percent consolidation versus log of pressure curves.
- F. The gradation characteristics of certain samples were evaluated by hydrometer (in accordance with ASTM D 7928) and sieve analysis procedures. The samples were soaked in water until individual soil particles were separated, then washed on the No. 200 mesh sieve, oven dried, weighed to calculate the percent passing the No. 200 sieve, and mechanically sieved.
- G. A Resistance ("R") Value test was conducted on a bulk sample secured during field study. The test was performed in accordance with California Method 301. Three specimens at different moisture contents were tested, and the R-Value at 300 psi exudation pressure was determined from the plotted results.

TABULATED LABORATORY TEST RESULTS

REMOLDED SAMPLES

BORING AND DEPTH	HA-1@0'-5'	HA-4@0'-5'
USCS	ML	ML
MAXIMUM DRY DENSITY (pcf)	124, 132*	126.5, 135*
OPTIMUM MOISTURE (%)	10, 8.5*	9.5, 8*
PEAK COHESION (psf)	120	200
PEAK FRICTION ANGLE	30°	28°
ULTIMATE COHESION (psf)	50	120
ULTIMATE FRICTION ANGLE	31°	27°

***Corrected for Oversized Material (ASTM D4718)**

RELATIVELY UNDISTURBED SAMPLES

BORING AND DEPTH	HA-2@2.5'	HA-3@2.5'
USCS	SM	SM
FINE CONTENT (<#200)	48%	43%

Resistance Value

BORING HA-2 @0'-5'	44
--------------------	----

Individual Laboratory Test Results

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Ventura High School Track & Field

Sample ID: H A 1 @ 0-5'

Procedure Used: A

Prep. Method: Moist

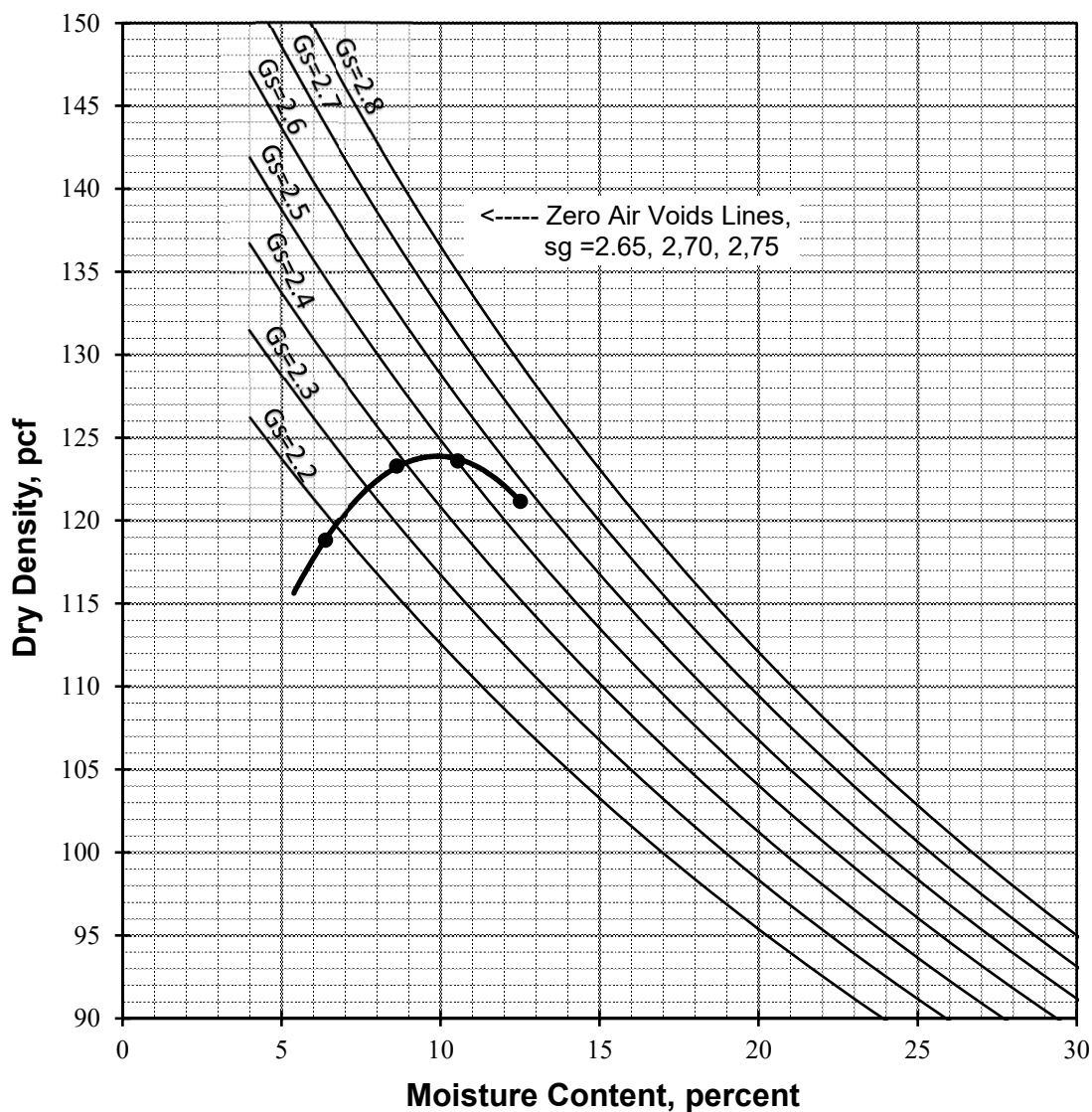
Date: 4/10/2025

Rammer Type: Automatic

Description: Dark Yellowish Brown Sandy Silt with Gravel

Maximum Density: 124 pcf
Optimum Moisture: 10%

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	27.0



MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Ventura High School Track & Field

Procedure Used: A

Sample ID: H A 1 @ 0-5'

Prep. Method: Moist

Date: 4/10/2025

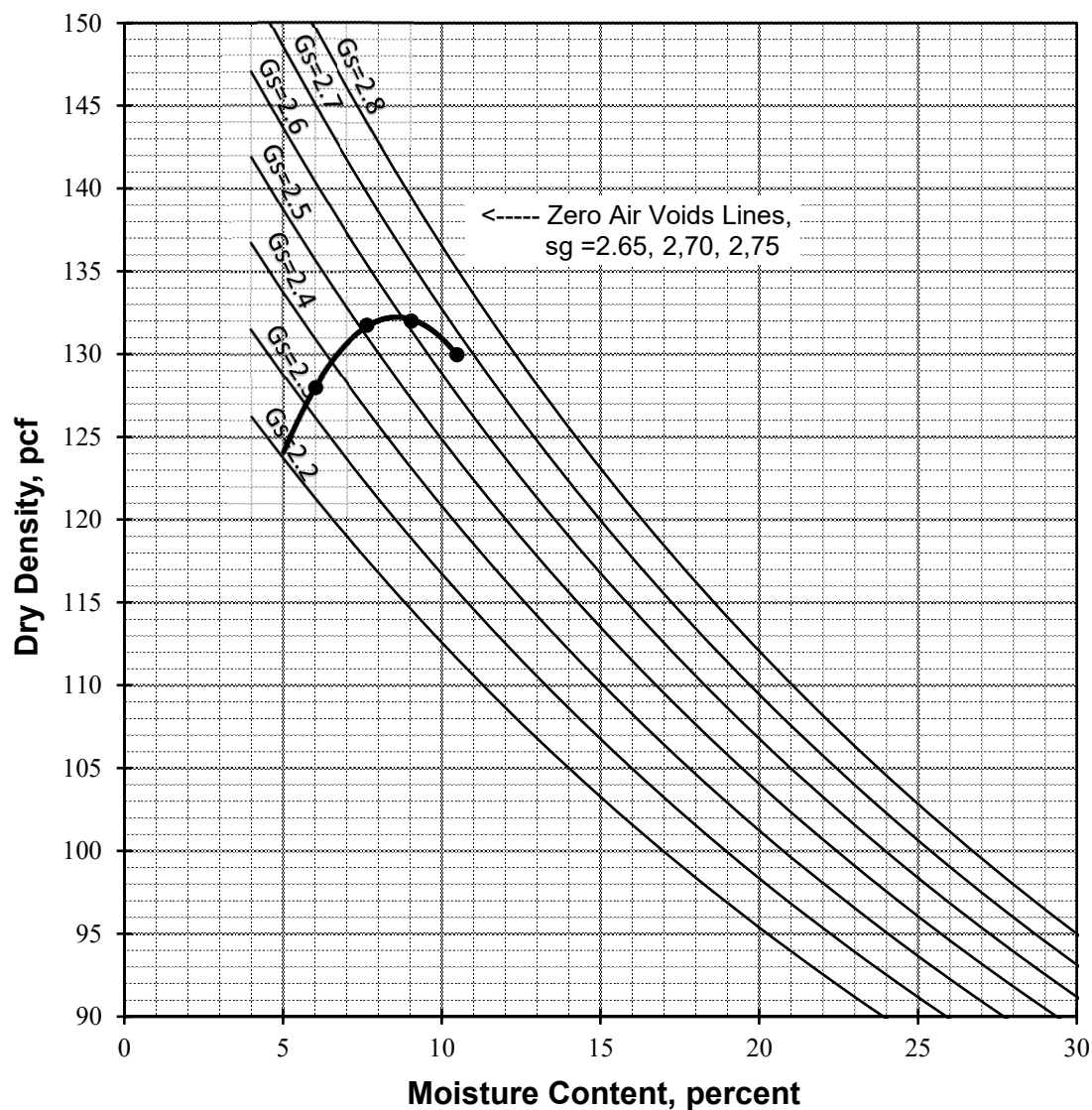
Rammer Type: Automatic

Description: Dark Yellowish Brown Sandy Silt with Gravel

Maximum Density: 132 pcf**Optimum Moisture: 8.5%**

Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	27.0



MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Ventura High School Track & Field

Sample ID: H A 4 @ 0-5'

Procedure Used: A

Prep. Method: Moist

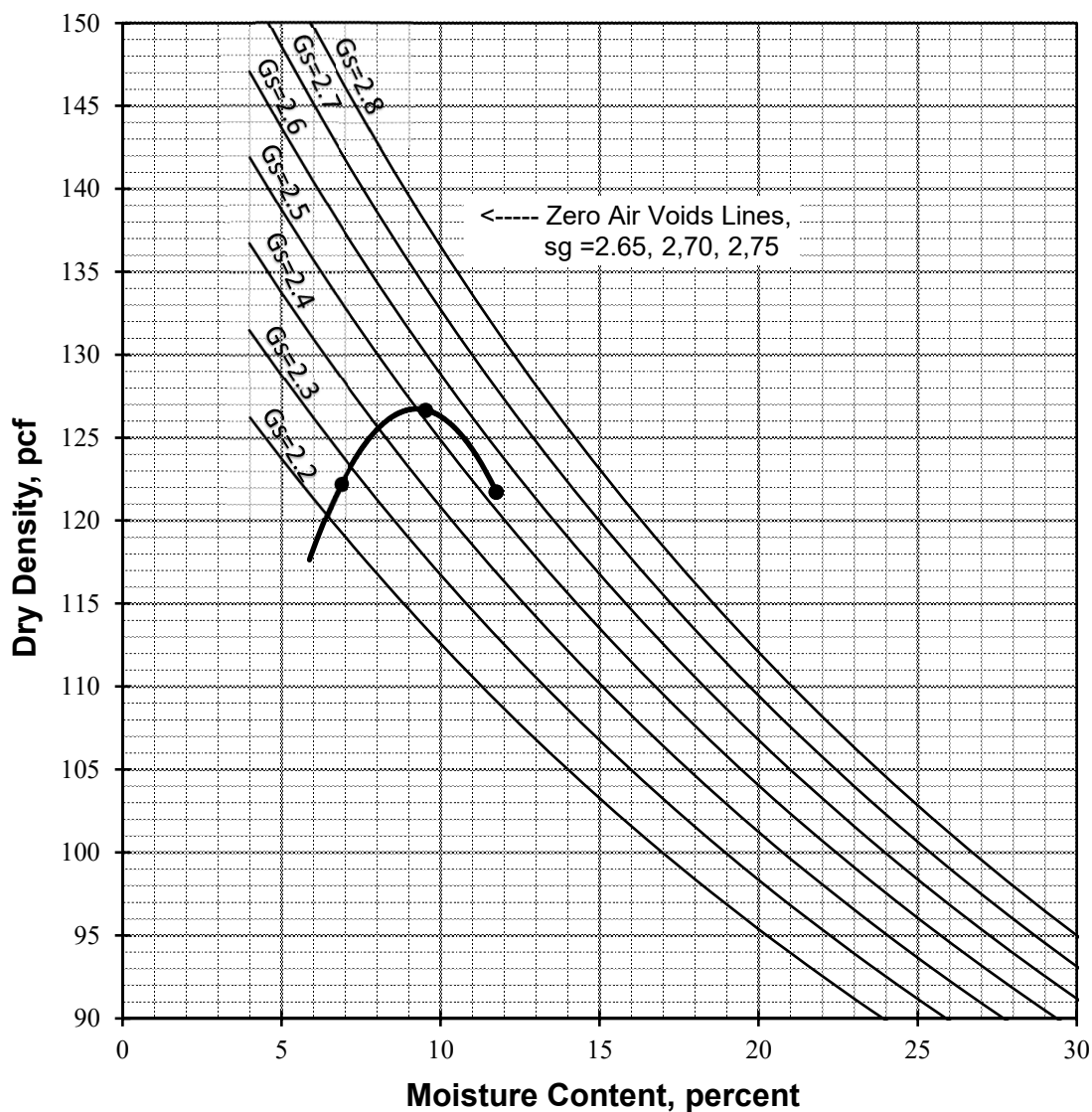
Date: 4/10/2025

Rammer Type: Automatic

Description: Dark Yellowish Brown Sandy Silt with Gravel

Maximum Density: 126.5 pcf**Optimum Moisture: 9.5%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	26.7



MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Ventura High School Track & Field

Procedure Used: A

Sample ID: H A 4 @ 0-5'

Prep. Method: Moist

Date: 4/10/2025

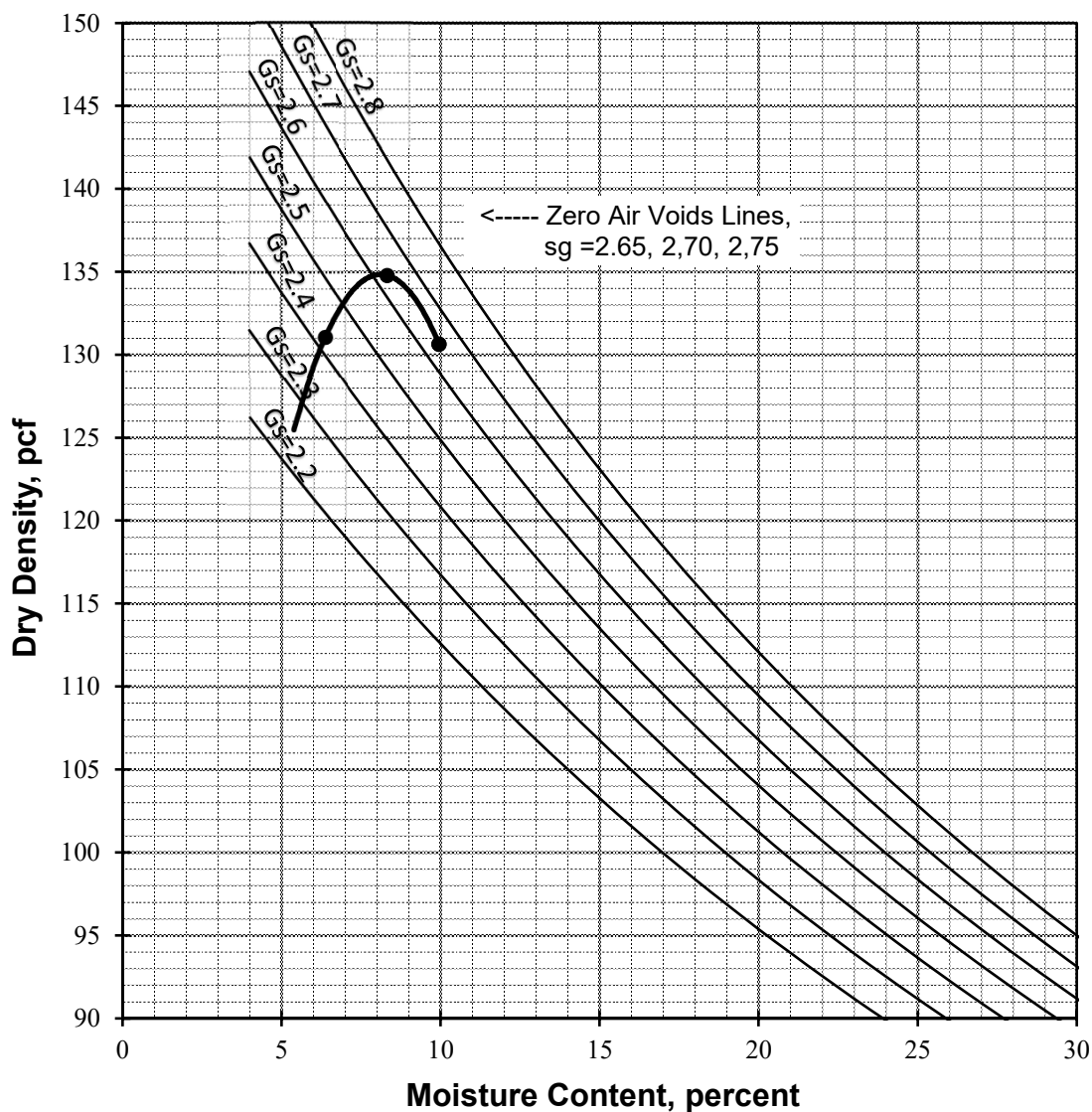
Rammer Type: Automatic

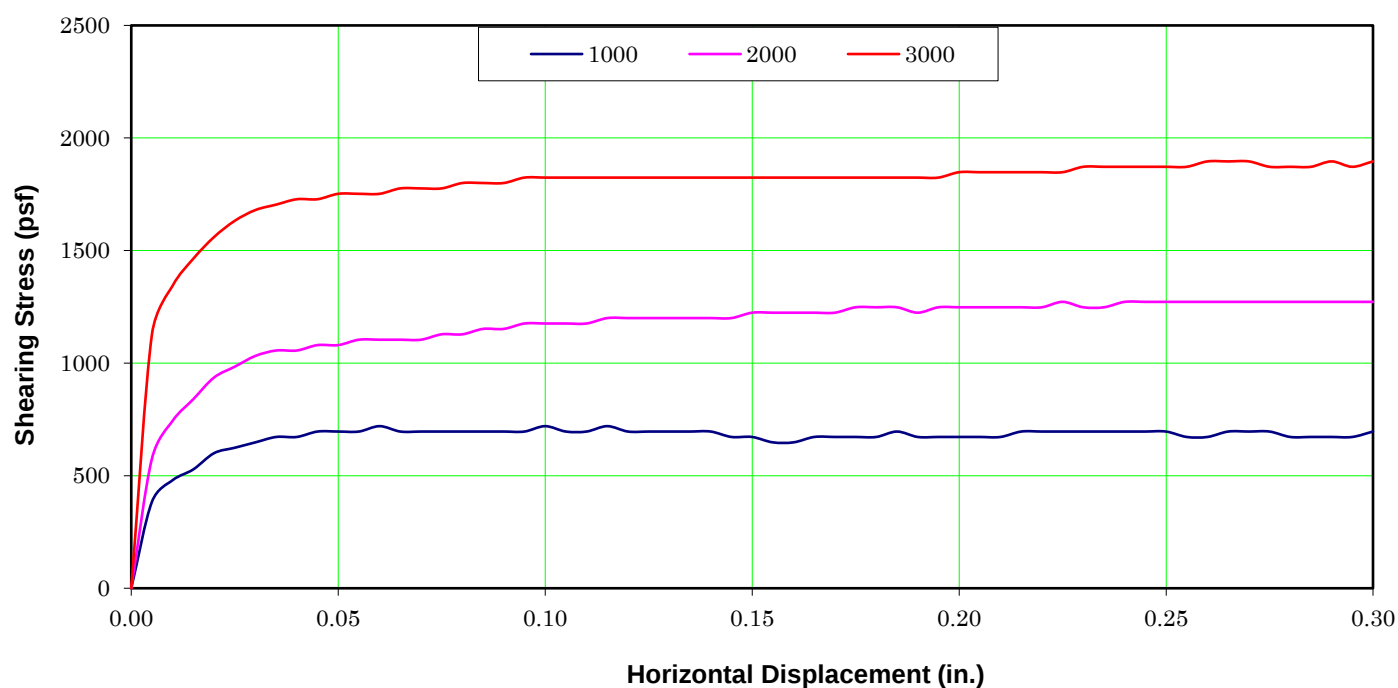
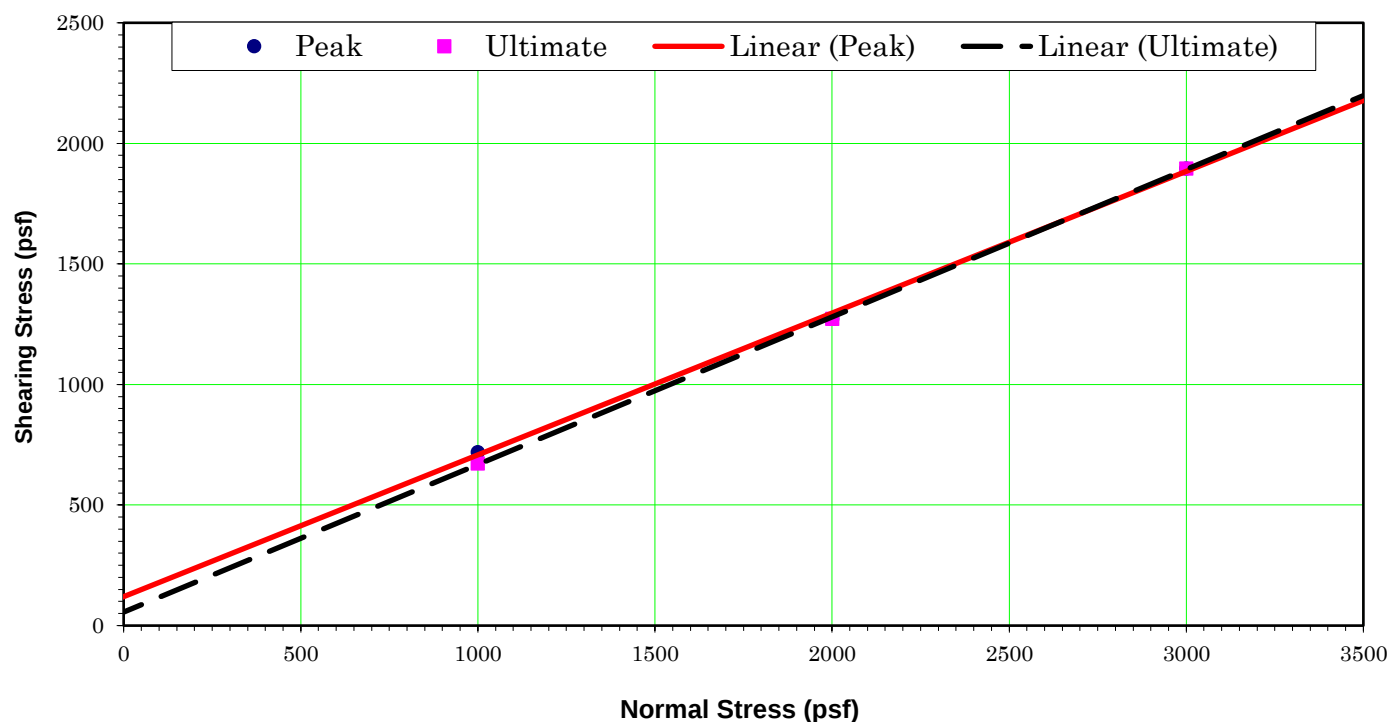
Description: Dark Yellowish Brown Sandy Silt with Gravel

Maximum Density: 135 pcf**Optimum Moisture: 8%**

Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	26.7





DIRECT SHEAR DATA*

Sample Location: HA-1 @ 0-5
Sample Description: ML
Dry Density (pcf): 110.8
Initial % Moisture: 9.9
Average Degree of Saturation: 100.0
Shear Rate (in/min): 0.005 in/min

Note: Sample specimens were initially saturated before testing by partially immersing the specimens in water for about 24 hours while the specimens were restrained from swelling. Initial saturation was not performed in the direct shear machine, and is not prescribed by ASTM D 3080.

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	720	1272	1896
Ultimate stress (psf)	672	1272	1896

	Peak	Ultimate
ϕ Angle of Friction (degrees):	30	31
c Cohesive Strength (psf):	120	50
Test Type:	Peak & Ultimate	

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

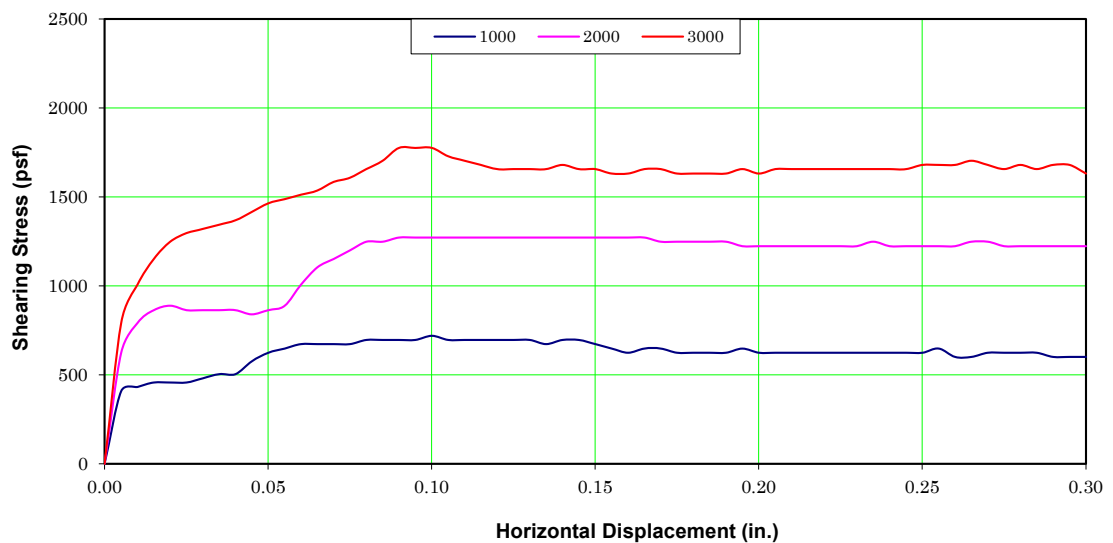
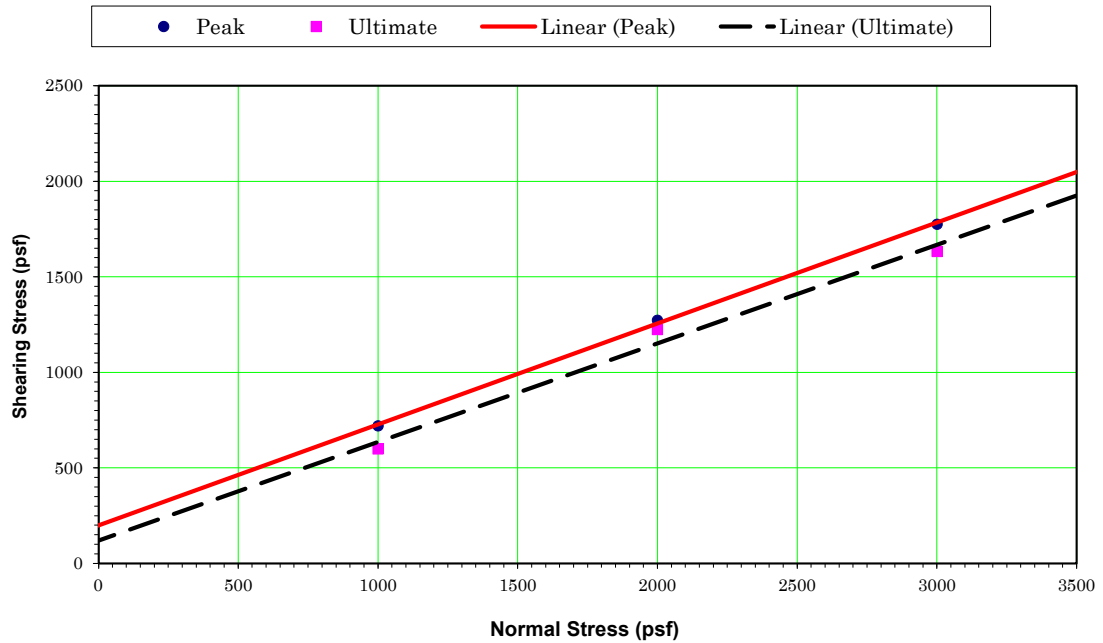
Ventura High School Track and Field



Earth Systems

4/22/2025

305811-006



DIRECT SHEAR DATA*

Sample Location: HA-4 @ 0-5
 Sample Description: ML
 Dry Density (pcf): 113.1
 Initial % Moisture: 9.3
 Average Degree of Saturation: 92.9
 Shear Rate (in/min): 0.005 in/min

Note: Sample specimens were initially saturated before testing by partially immersing the specimens in water for about 24 hours while the specimens were restrained from swelling. Initial saturation was not performed in the direct shear machine, and is not prescribed by ASTM D 3080.

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	720	1272	1776
Ultimate stress (psf)	600	1224	1632

	Peak	Ultimate
ϕ Angle of Friction (degrees):	28	27
c Cohesive Strength (psf):	200	120
Test Type: Peak & Ultimate		

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Ventura High School Track and Field



Earth Systems

4/22/2025

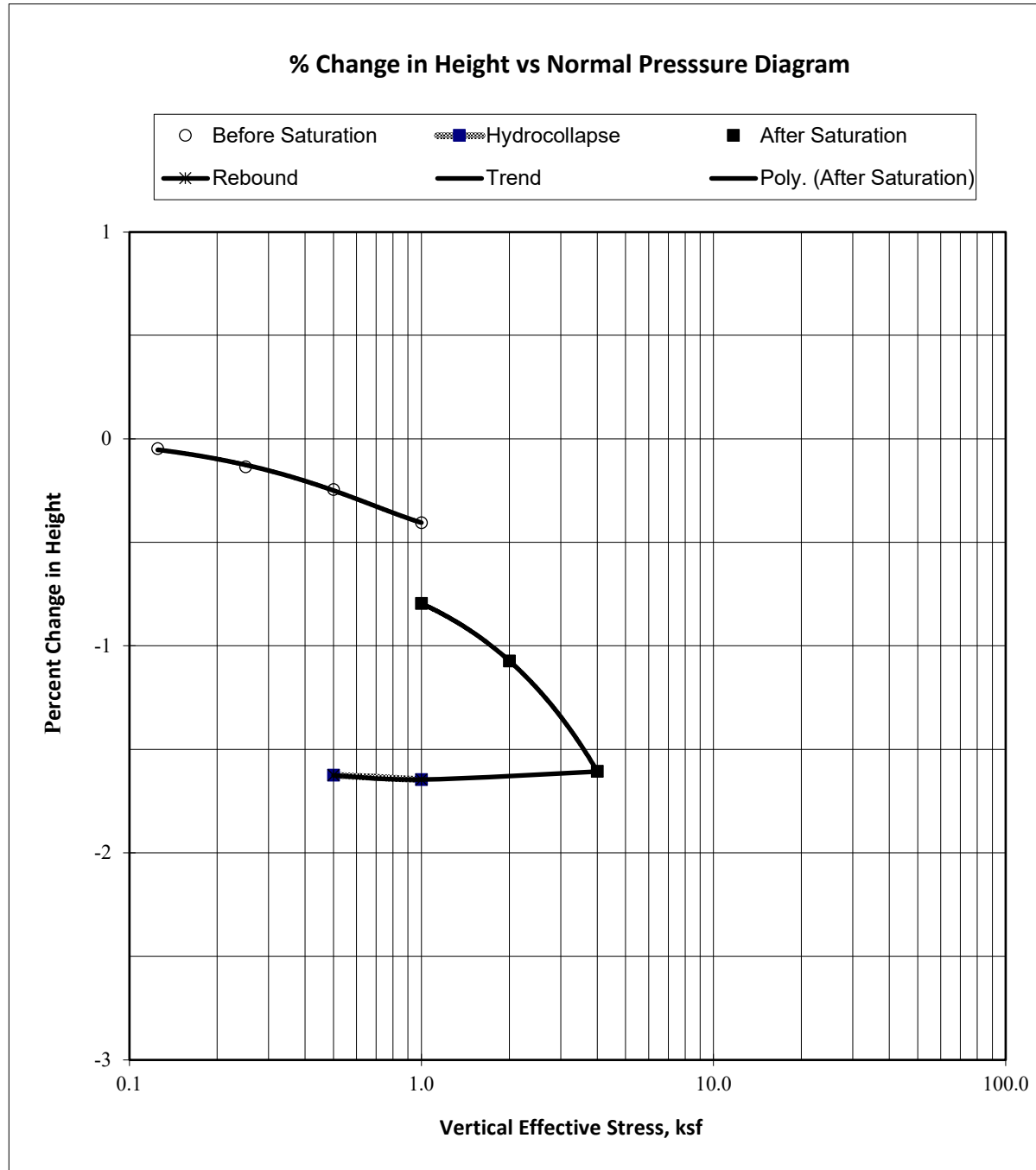
305811-006

CONSOLIDATION TEST

ASTM D 2435-90 & D5333

Ventura High School Track and Field
HA-2 @ 7.5
SM
Ring Sample

Initial Dry Density: 110.4 pcf
Initial Moisture, %: 15.8%
Specific Gravity: 2.67 (assume)
Initial Void Ratio: 0.511

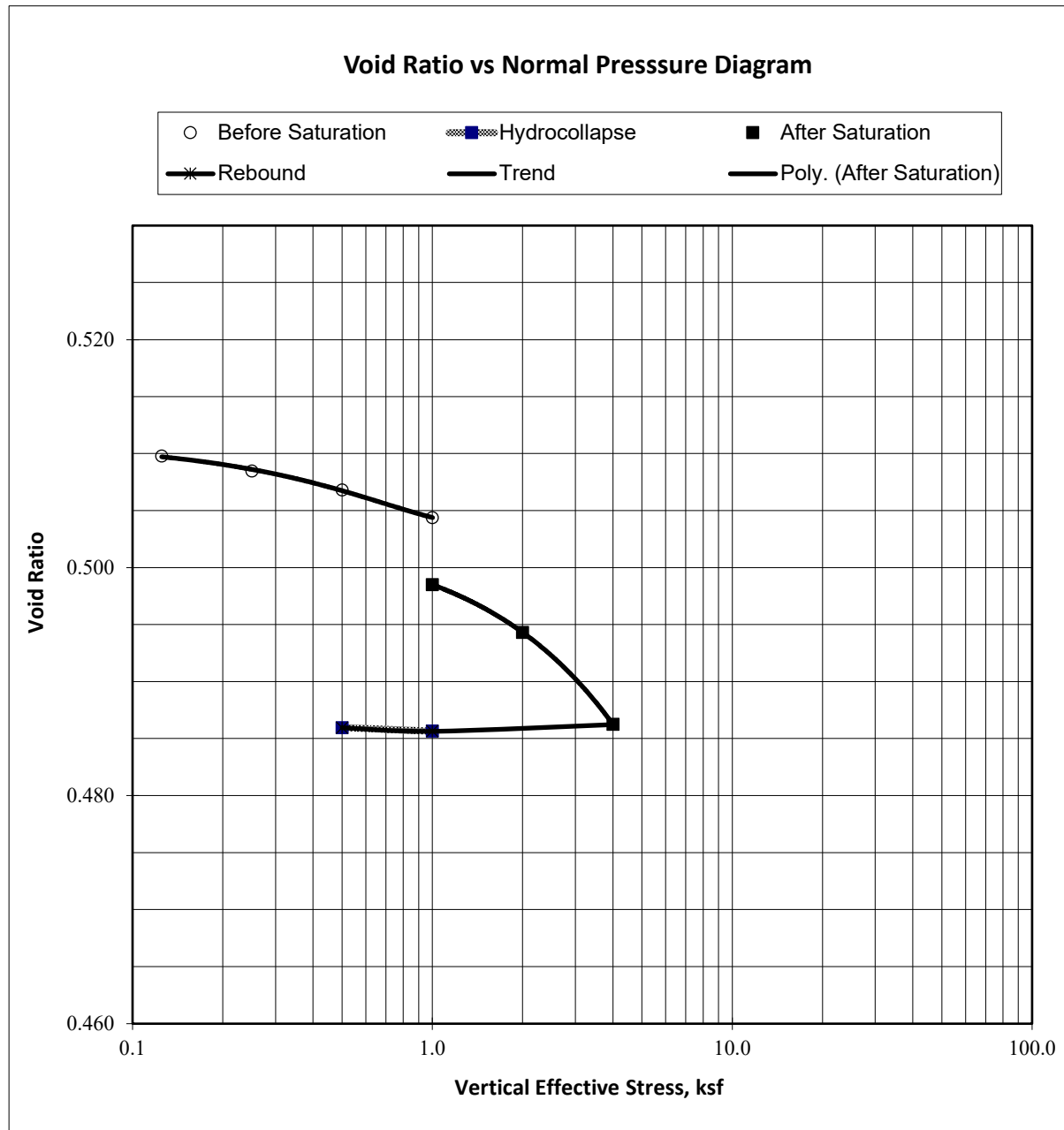


CONSOLIDATION TEST

ASTM D 2435-90

Ventura High School Track and Field
HA-2 @ 7.5
SM
Ring Sample

Initial Dry Density: 110.4
Initial Moisture, %: 15.8
Specific Gravity: 2.67 (assume)
Initial Void Ratio: 0.511

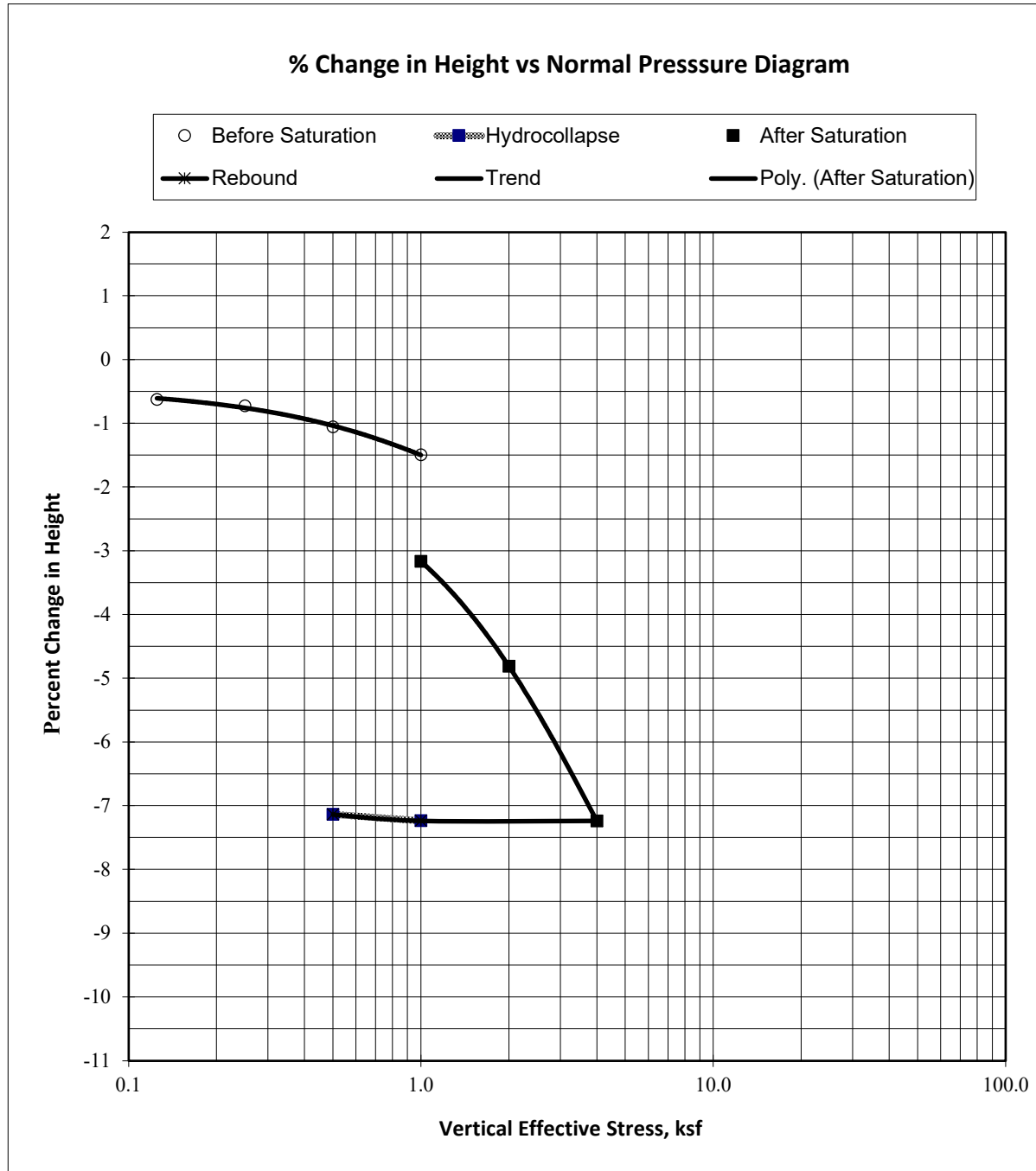


CONSOLIDATION TEST

ASTM D 2435-90 & D5333

Ventura High School Track and Field
HA-3 @ 7.5
SM
Ring Sample

Initial Dry Density: 109.2 pcf
Initial Moisture, %: 16.8%
Specific Gravity: 2.67 (assume
Initial Void Ratio: 0.527

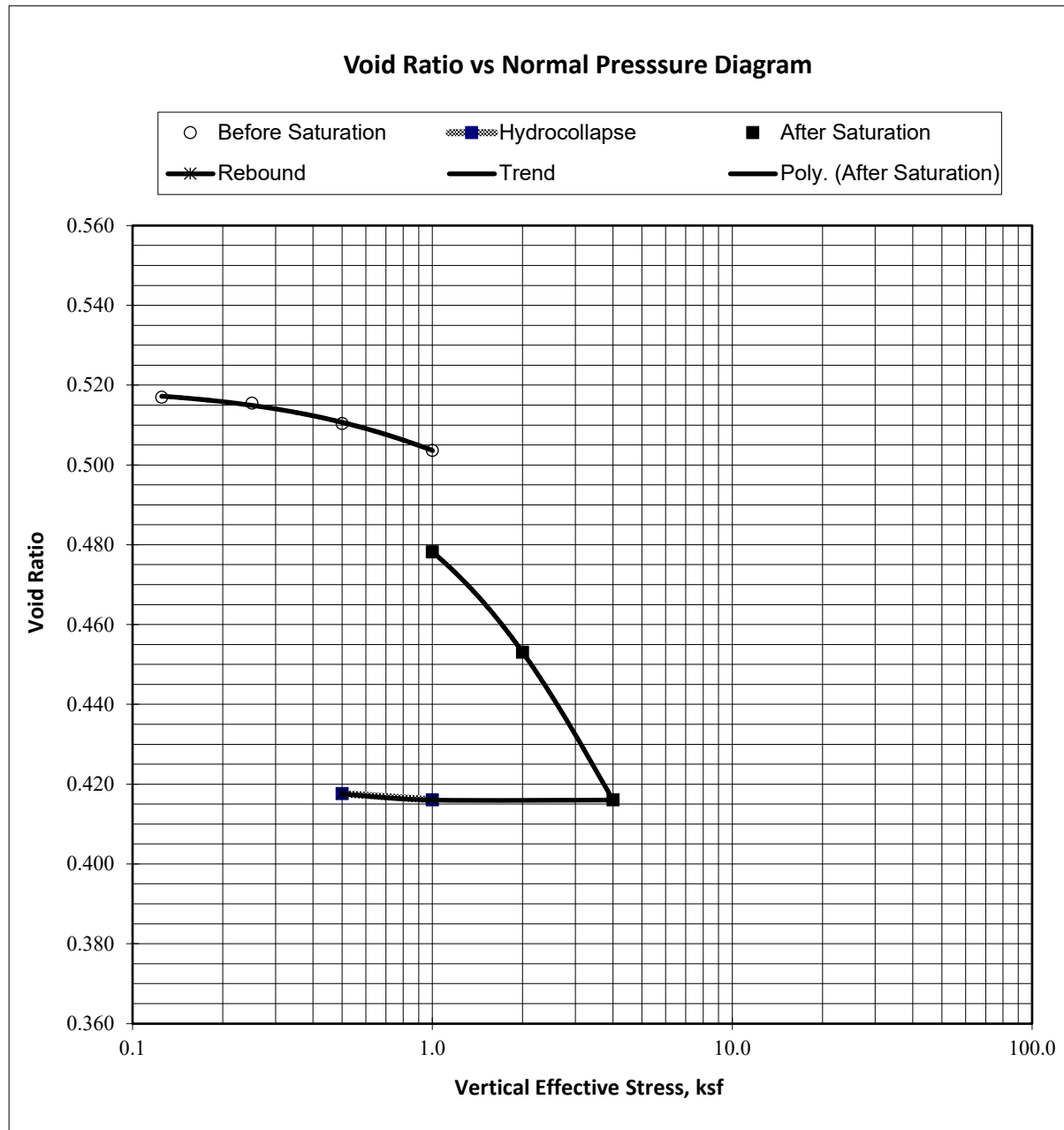


CONSOLIDATION TEST

ASTM D 2435-90

Ventura High School Track and Field
HA-3 @ 7.5
SM
Ring Sample

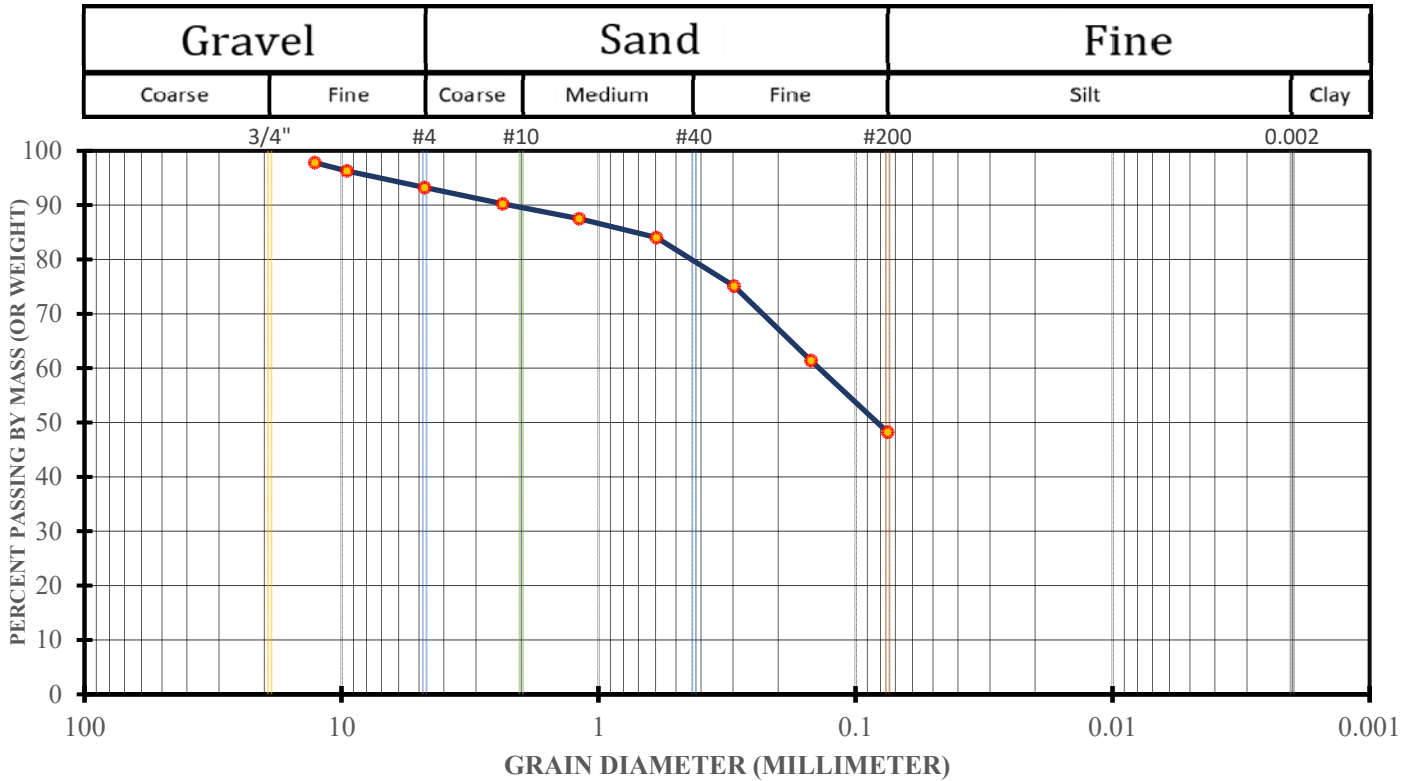
Initial Dry Density: 109.2
Initial Moisture, %: 16.8
Specific Gravity: 2.67 (assume)
Initial Void Ratio: 0.527



Sieve Analysis Results

Sample ID: HA-2 @ 2.5
Sample Type: Rings

Date: 4/11/2025
Technician: -



Sieve #	Size (mm)	Passing Weight (%)
1/2"	12.70	97.8
3/8"	9.52	96.3
#4	4.75	93.2
#8	2.36	90.2
#16	1.19	87.5
#30	0.60	84.0
#50	0.297	75.1
#100	0.149	61.4
#200	0.075	48.2
-	-	-
-	-	-
-	-	-
-	-	-
-	-	-
-	-	-
-	-	-
-	-	-
-	-	-

Class Percentage	
Gravel (<#4)	6.8%
Sand (#4<d<#200)	45.0%
Fine (<#200)	48.2%

Particle Size Distribution	
D60	0.1385
D30	#N/A
D10	#N/A

Coefficient of Uniformity (Cu)	
$C_u = D_{60} / D_{10}$	#N/A

Coefficient of Curvature (Cc)	
$C_c = (D_{30})^2 / D_{60} \times D_{10}$	#N/A

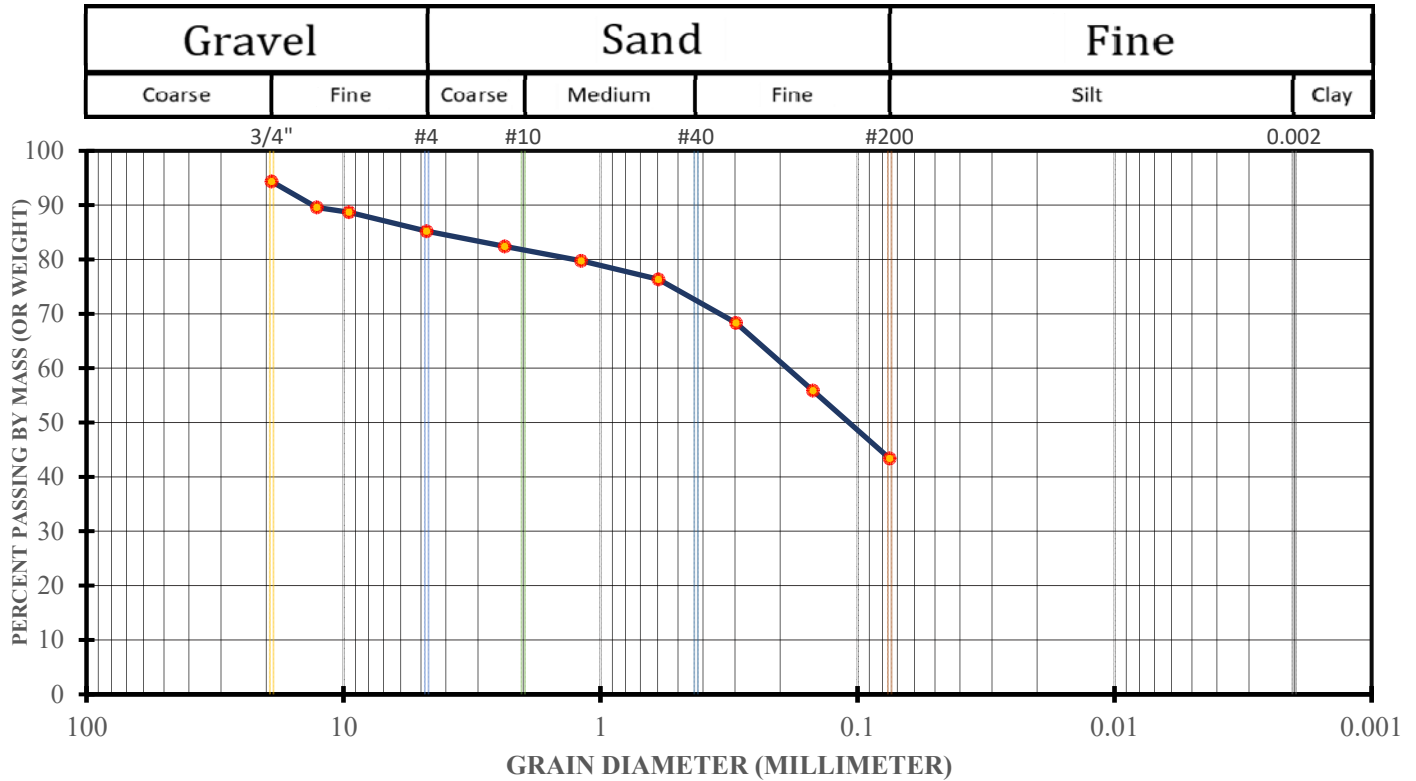
This Program is Designed By: Amir Nikouei Nahali - Earth Systems - Ventura

Job Name	Ventura High School Track and Field	 EARTH SYSTEMS
Job Number	305811-006	
Date	April 2025	

Sieve Analysis Results

Sample ID: HA-3 @ 2.5
Sample Type: Rings

Date: 4/11/2025
Technician: -



Sieve #	Size (mm)	Passing Weight (%)
3/4"	19.05	94.4
1/2"	12.70	89.6
3/8"	9.52	88.7
#4	4.75	85.2
#8	2.36	82.4
#16	1.19	79.8
#30	0.595	76.3
#50	0.297	68.3
#100	0.149	55.9
#200	0.075	43.4
-	-	-
-	-	-
-	-	-
-	-	-
-	-	-
-	-	-
-	-	-
-	-	-

Class Percentage	
Gravel (<#4)	14.8%
Sand (#4<d<#200)	41.8%
Fine (<#200)	43.4%

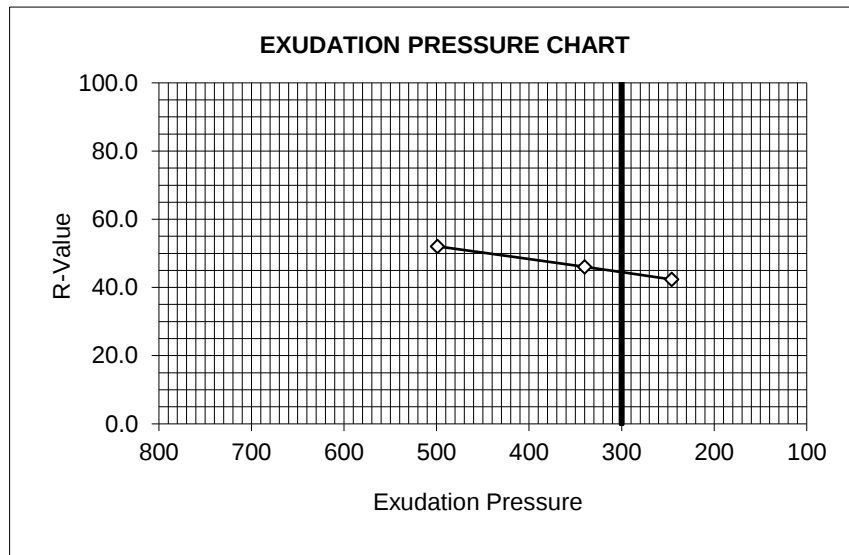
Particle Size Distribution	
D60	0.1872
D30	#N/A
D10	#N/A

Coefficient of Uniformity (Cu)	
$C_u = D_{60} / D_{10}$	#N/A

Coefficient of Curvature (Cc)	
$C_c = (D_{30})^2 / D_{60} \times D_{10}$	#N/A

This Program is Designed By: Amir Nikouei Nahali - Earth Systems - Ventura

Job Name	Ventura High School Track and Field	 EARTH SYSTEMS
Job Number	305811-006	
Date	April 2025	



JOB NAME: Ventura HS Track & Field

SAMPLE I. D.: HA-2 0-5'

SOIL DESCRIPTION: Sandy Silt

SPECIMEN NUMBER	D	E	F
EXUDATION PRESSURE	499	340	246
RESISTANCE VALUE	52.1	46.1	42.3
EXPANSION DIAL(0.0001")	0	0	0
EXPANSION PRESSURE (PSF)	0.0	0.0	0.0
% MOISTURE AT TEST	9.9	10.5	11.1
DRY DENSITY AT TEST	121.9	121.7	109.3

R-VALUE @ 300 PSI EXUDATION 44

**Based on Traffic Index = 8.00 & Gravel Factor = 1.34*



Earth Systems Pacific

5917 Olivas Park Drive, Suite F
Ventura, CA 93003
(805) 642-6727 Phone

115 W. Canon Perdido Street
Santa Barbara, CA 93101
(805) 642-6727 Phone

COMPRESSION TEST REPORT

Client: Ventura Unified School District LEA #: 6 DSA FILE #:
Attn: Ryan Hughes EXP. Date: 11/1/2026 DSA APPL. #:
Address: 359 South Victoria Avenue Lab Facility: Earth Systems Pacific, Ventura
Ventura, CA 93003 Job #: 305811-006 Report #: 25-4-17

Project Name: Ventura HS Track & Field Structure: Bleachers
Location in Structure: Bleachers Report Date: 4/3/25
Sampled By: Steven Bacon, ICC Certified Special Inspector No.: 9837632 Sample Date: 3/26/25

SAMPLING INFORMATION

Specified Strength psi @ days

Material: ☐ Concrete ☐ Grout ☐ Mortar ☐ Prisms ☒ Cores ☐ Other

	Actual	Spec.	Pass/ Fail?
Slump (inches)	--	--	--
Percent Air (%) ☐ Press. ☐ Volum.	--	--	--
Unit Weight (pcf)	--	--	--
Air Temperature (°F)	--	--	--
Mix Temperature (°F)	--	--	--

Mix Number: Load # of
Concrete Supplier:
Truck #: Ticket #:
Time Batched: Time Sampled:
Set #: 1 of 1 yds of total yds
Sampled from: ☐ Chute ☐ Hose ☐ Other

Days to be Tested: Min. °F Max. °F Number of Samples: 3

TESTING INFORMATION

Date Samples Received Initial Curing Final Curing

Identification	Core-1	Core-2	Core-3					
Date Tested	3/26/25	3/26/25	3/26/25					
Age in Days								
Diameter/Width (in.)	3.74	3.74	3.74					
Height (in.)	7.49	7.48	7.50					
Cross Sect. Area (in. ²)	10.98	10.98	10.98					
Maximum Load (lbs.)	67,500	80,000	74,000					
Correction Factor	6,150	7290	6740					
Compr. Strength (psi)	6,148	7,286	6,740					
Fracture Type	3	3	3					

ASTM Methods, as Applicable: C31, C39, C140, C143, C172, C173, C231, C617, C1019, C1064, C1231, C1552

28 Day Average: psi

REMARKS
(Environmental
Conditions, ETC):

Tested by: Steve DeBolt

☐ ADDITIONAL COMMENTS ATTACHED.

The Material ☐ WAS ☐ WAS NOT
SAMPLED AND TESTED IN ACCORDANCE WITH
THE REQUIREMENTS OF THE DSA APPROVED DOCUMENTS.

The Material Tested ☐ MET ☐ DID NOT MEET
THE REQUIREMENTS OF THE DSA APPROVED DOCUMENTS.

Copy to: Standard Distribution
DSA Box

The results contained herein apply only to the items
inspected or tested. This report shall not be reproduced
except in full without prior written approval of Earth Systems
Compression Test Report (rev. 7/22/22)



Anthony P. Mazzei 4/7/25
Signature Date
Anthony P. Mazzei/Sr. Vice President, Managing Principal
Print Name/Title

APPENDIX C

OSHDP Seismic Parameters

US Seismic Design Maps

Fault Parameters



Ventura High School Track & Field

Latitude, Longitude: 34.28054, -119.26443



Date	4/2/2025, 1:50:18 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S _S	1.99	MCE _R ground motion. (for 0.2 second period)
S ₁	0.749	MCE _R ground motion. (for 1.0s period)
S _{MS}	1.99	Site-modified spectral acceleration value
S _{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S _{DS}	1.327	Numeric seismic design value at 0.2 second SA
S _{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F _a	1	Site amplification factor at 0.2 second
F _v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.874	MCE _G peak ground acceleration
F _{PGA}	1.1	Site amplification factor at PGA
PGA _M	0.961	Site modified peak ground acceleration
T _L	8	Long-period transition period in seconds
SsRT	1.99	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.256	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.581	Factored deterministic acceleration value. (0.2 second)
S1RT	0.749	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.848	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.811	Factored deterministic acceleration value. (1.0 second)
PGA _d	1.035	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA _{UH}	0.874	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C _{RS}	0.882	Mapped value of the risk coefficient at short periods
C _{R1}	0.883	Mapped value of the risk coefficient at a period of 1 s
C _V	1.498	Vertical coefficient

Table E-5 - General Procedure Seismic Design Values

2022 California Building Code (CBC) (ASCE 7-16) Seismic Design Parameters

(Values presented should only be used by a Structural Engineer to determine if the exception in 11.4.8 (ASCE 7-16) can be used)

Seismic Design Category	D	<u>CBC Reference</u>	<u>ASCE 7-16 Reference</u>	
Site Class	D	Table 1613.5.6	Table 11.6-1	
Latitude:	34.281	Table 1613.5.2	Table 20.3-1	
Longitude:	-119.264			
<u>Maximum Considered Earthquake (MCE) Ground Motion</u>				
Short Period Spectral Reponse	S_s	1.990 g	Figure 1613.5	Figure 22-1
1 second Spectral Response	S₁	0.749 g	Figure 1613.5	Figure 22-2
Site Coefficient	F _a	1.000	Table 1613.5.3(1)	Table 11.4-1
Site Coefficient	F _v	1.700	Table 1613.5.3(2)	Table 11-4.2
	S _{MS}	1.990 g	= F _a *S _s	
	S _{M1}	1.273 g	= F _v *S ₁	
<u>Design Earthquake Ground Motion</u>				
Short Period Spectral Reponse	S_{DS}	1.327 g	= 2/3*S _{MS}	
1 second Spectral Response	S_{D1}	0.849 g	= 2/3*S _{M1}	

Site Specific Evaluation May Be Required Due to Site Class = D or E and $S1 > 0.2$. The Presented SDS and SD1 are NOT Valid Unless the Exception of ASCE7-16, Section 11.4.8 Applies

To	0.13 sec	$= 0.2 * S_{D1} / S_{DS}$
Ts (11.4.8 ASCE 7-16 Exception Assumed)	0.64 sec	$= S_{D1} / S_{DS}$
Risk Category	II	Table 1604.5
Seismic Importance Factor	1.00	
F_{PGA}	1.10	
PGA_M	0.961	
Vertical Coefficient (C_v)	1.50	Table 11.9-1

Table 11.5-1 Design

Period T (sec)	S_a (g)
0.00	0.531
0.05	0.842
0.13	1.327
0.64	1.327
0.80	1.061
1.00	0.849
1.20	0.707
1.40	0.606
1.60	0.531
1.80	0.472
2.00	0.424
2.20	0.386
2.40	0.354
2.60	0.326
2.80	0.303
3.00	0.283

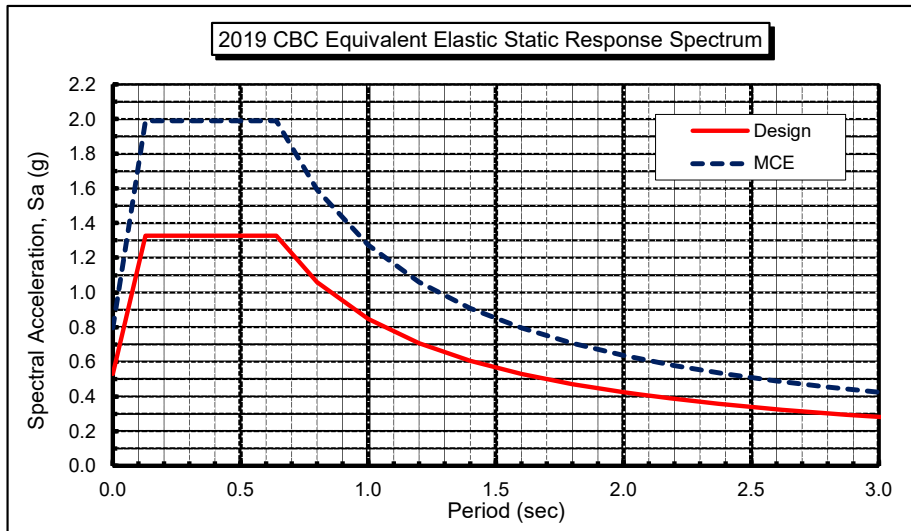


Table E-1
Fault Parameters

Fault Section Name	Distance		Upper	Lower	Avg	Avg	Avg	Trace	Fault Type	Mean	Slip Rate
			Seis. Depth	Seis. Depth	Dip Angle	Dip Direction	Rake	Length		Return Interval	
	(miles)	(km)	(km)	(km)	(deg.)	(deg.)	(deg.)	(km)		(years)	(mm/yr)
Ventura-Pitas Point FM3.1, 3.2	0.0	0.0	1.0	15.0	64	353	60	59	B	7.1	1
Oak Ridge (Offshore) FM3.2	2.1	3.3	0.0	7.9	32	180	90	38	B	6.9	3
Oak Ridge (Onshore) FM3.1, 3.2	3.3	5.3	1.0	19.4	65	159	90	50	B	7.2	4
Red Mountain FM3.1, 3.2	4.6	7.4	0.0	14.1	56	2	90	101	B	7.4	2
Sisar FM3.1, 3.2	8.2	13.2	0.0	17.4	29	168	na	20	B'	7.0	
North Channel FM3.2	9.5	15.3	1.1	4.5	26	10	90	51	B	6.7	1
Simi-Santa Rosa FM3.1, 3.2	10.4	16.7	1.0	12.1	60	346	30	39	B	6.8	1
Mission Ridge-Arroyo Parida-Santa Ana FM3.1, 3	10.4	16.7	0.0	7.6	70	176	90	69	B	6.8	0.4
San Cayetano FM3.1, 3.2	13.1	21.2	0.0	16.0	42	3	90	42	B	7.2	6
Pitas Point (Lower)-Montalvo, FM3.1	14.3	23.0	0.4	12.7	16	359	90	30	B	7.3	2.5
Malibu Coast (Extension), alt 1, FM3.1	14.6	23.4	0.0	7.8	74	4	30	35	B'	6.5	
Malibu Coast (Extension), alt 2 FM3.2	14.6	23.4	0.0	16.6	74	4	30	35	B'	6.9	
Santa Ynez (East) FM3.1, 3.2	15.9	25.6	0.0	13.3	70	172	0	68	B	7.2	2
Channel Islands Western Deep Ramp FM3.1, 3.2	16.5	26.6	4.8	12.5	21	204	90	62	B'	7.3	
Channel Islands Thrust FM3.1, 3.2	17.6	28.3	5.0	12.3	20	354	90	59	B	7.3	1.5
Pitas Point (Upper) FM3.2	18.6	29.9	1.4	10.0	42	15	90	35	B	6.8	1
Santa Cruz Island FM3.1, 3.2	20.6	33.1	0.0	13.3	90	188	30	69	B	7.1	1
Pine Mtn FM3.1, 3.2	21.8	35.1	0.0	16.3	45	5	na	62	B'	7.3	
Anacapa-Dume, alt 1, FM3.1	22.2	35.7	0.0	15.5	45	354	60	51	B	7.2	3
Anacapa-Dume, alt 2, FM3.2	22.2	35.7	1.2	11.4	41	352	60	65	B	7.2	3
Oak Ridge (Offshore), west extension FM3.2	23.6	38.0	0.0	3.1	67	195	na	28	B'	6.1	
Santa Cruz Catalina Ridge alt 1, FM3.1	24.1	38.7	0.0	11.0	90	na	na	114	B'	7.2	
Santa Cruz Catalina Ridge Alt2 FM3.2	24.1	38.7	0.0	11.0	90	38	na	137	B'	7.3	
Malibu Coast, alt 1, FM3.1	24.9	40.1	0.0	7.8	75	3	30	38	B	6.6	0.3
Malibu Coast, alt 2 FM3.2	24.9	40.1	0.0	16.6	74	3	30	38	B	6.9	0.3
Santa Ynez (West) FM3.1, 3.2	26.0	41.8	0.0	9.2	70	182	0	80	B	6.9	2
Big Pine (Central) FM3.1, 3.2	27.3	43.9	0.0	6.6	76	167	na	23	B'	6.3	
Ozena FM3.1, 3.2	27.6	44.4	0.0	13.9	33	na	na	41	B'	7.2	
Big Pine (West) FM3.1, 3.2	28.1	45.3	0.0	11.0	50	2	na	18	B'	6.5	
Santa Susana, alt 2 FM3.2	28.8	46.4	0.0	10.6	53	10	90	43	B'	6.8	
Santa Susana, alt 1, FM3.1	28.9	46.5	0.0	16.3	55	9	90	27	B	6.8	5
Pitas Point (Lower, West), FM 3.1	29.4	47.2	1.5	8.8	13	3	90	35	B	7.2	2.5
Del Valle FM3.1, 3.2	30.9	49.7	0.0	18.8	73	195	90	9	B'	6.3	
Holser, alt 1, FM 3.1	31.1	50.1	0.0	18.6	58	187	90	20	B	6.7	0.4
Holser, alt 2 FM3.2	31.1	50.1	0.0	18.5	58	182	90	17	B'	6.7	
Big Pine (East) FM3.1, 3.2	31.7	51.0	0.0	14.3	73	338	na	23	B'	6.6	
Northridge Hills FM3.1, 3.2	31.9	51.3	0.0	14.9	31	19	90	25	B'	7.0	
Northridge FM3.1, 3.2	32.9	52.9	7.4	16.8	35	201	90	33	B	6.8	1.5
Santa Monica Bay FM3.1, 3.2	34.4	55.4	2.3	18.0	20	44	na	17	B'	7.0	
San Pedro Basin FM3.1, 3.2	35.5	57.2	0.8	12.3	88	51	na	69	B'	7.0	

Reference: USGS OFR 2013-1165 (CGS SP 228)

Based on Site Coordinates of 34.28054 Latitude, -119.26443 Longitude

Mean Magnitude for Type A Faults based on 0.1 weight for unsegmented section, 0.9 weight for segmented model (weighted by probability of each scenario with section listed as given on Table 3 of Appendix G in OFR 2008-1437). Mean magnitude is average of Ellworths-B and Hanks & Bakun moment area relationship.

APPENDIX D

CPT-Based Liquefaction and Dry Sand Seismic Settlement Analyses

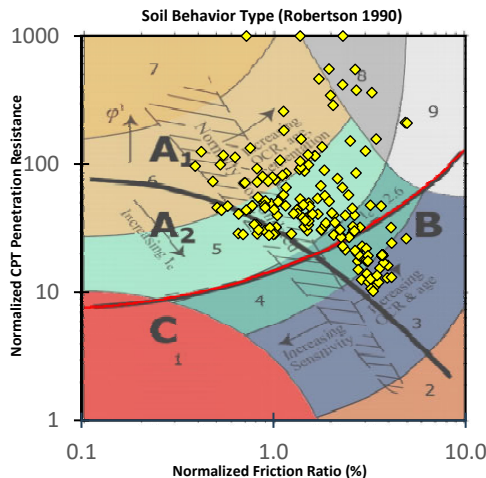
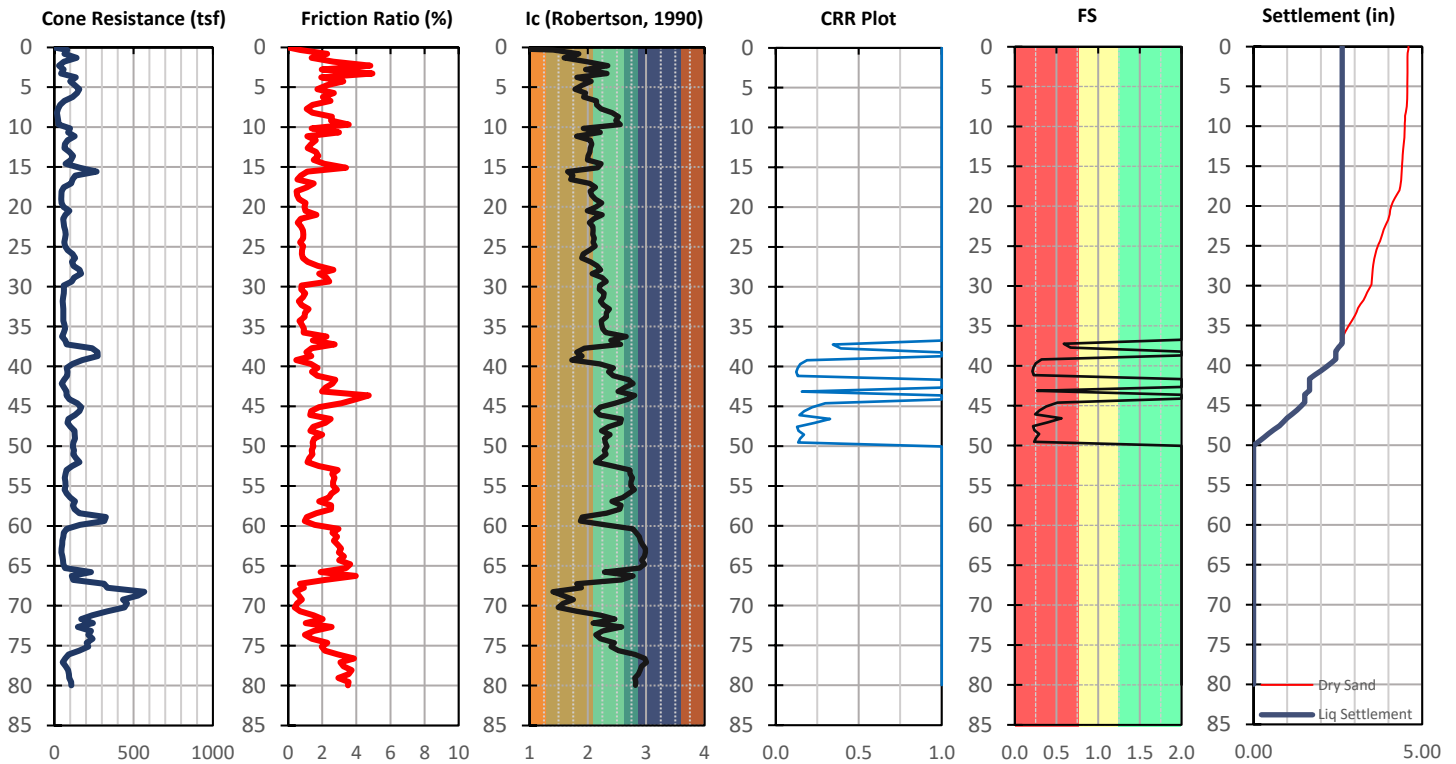
LIQUEFACTION ANALYSIS REPORT

Project Name: Ventura High School Track and Field
Job Number: 305811-006
Date: 4/21/2025

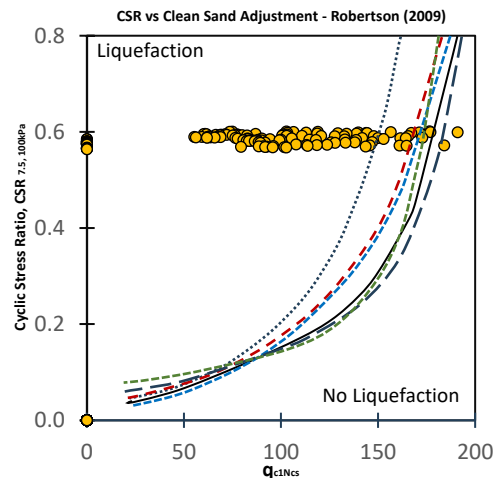


Input CPT File: CPT-1
Corrected Tip Resistance: Yes - Cone Area Ratio = 0.8
Thin Layer Correction: No
Analysis Method for CSR: Robertson (2009)
Analysis Method for CRR: Robertson (2009)
Analysis Method for Sett: Zhang et al. (2002)
Equivalent Clean Sand: Robertson (2009)
Unit Weight Based on: Robertson & Cabal (2010)
Fine Content Based on: Robertson & Wride (1998)
Ground Improvement: No
Dry Sand Settlement: Yes - Use 2/3 of PGAm
Weighting Factor for ev: No
Cyclic Softening in Clay: No
Average Results Interval: 3

Earthquake Magnitude (Mw): 7.4
Peak Ground Acceleration (PGAm): 0.96
Measured Ground Water Depth: 80 ft
Highest Ground Water Depth: 37 ft
Ground Improvement Depth: -
Depth Limit for Settlement Analysis: 50 ft
Depth Limit for ev Weighting Factor: -
Ic Cut-Off Value: 2.60
FS Cut-Off Value: 1.30
Total Thickness of Liquefiable Layers: 9.35 ft
Settlement in Dry Sand: 1.96 in
Settlement in Saturated Sand: 2.63 in
Settlement in Clay-Like Materials: -
Total Settlement: 4.59 in



- | | | |
|---------------------------|-----------------|-----------------------------------|
| 1. Sensitive Fine-Grained | 4. Silt-Mixture | 7. Gravely Sand to Sand |
| 2. Organic | 5. Sand-Mixture | 8. Very Stiff Sand to Clayey Sand |
| 3. Clay | 6. Sand | 9. Very Stiff Fine-Grained |



- | | |
|--------------------------------|---------------------------------|
| - - - Robertson & Wride (1998) | Juang et al. (2006) |
| — Idriss & Boulanger (2004) | — Idriss & Boulanger (2008) |
| - - - Moss et al. (2006) | - - - Idriss & Boulanger (2014) |

This Program is Designed by: Amir Nikouei Nahali

APPENDIX E

Infiltration Testing Results

INFILTRATION RATE BY THE BOREHOLE PERCOLATION TEST METHOD

This workbook calculates an adjusted infiltration rate from a borehole percolation test. The percolation rate is adjusted for sidewall area according to the Porchet method, and then re-adjusted for the effect of the gravel placed in annulus between the borehole wall and a pipe placed in the borehole by a method presented in Caltrans Test 750.

Project Name	Ventura High School Track & Field
Project Number	305811-006
Test Hole No.	IT-2
Tester	V. Wallace
Pre-Soak Date	3/24/25
Test Date	3/26/25

Test Hole Radius, r (inches)	2
Total Depth of Test Hole, D _T (feet)	2.0
Inside Diameter of Pipe, I (inches)	2.00
Outside Diameter of Pipe, O (inches)	2.36
Pipe Stick-Up (feet)	0.0
Porosity of Gravel, n	0.41
Porosity Correction Factor, C	0.52
Factor of Safety (FOS), F	N/A

[illegible]