



GEOTECHNICAL & ENVIRONMENTAL ENGINEERING — CONSTRUCTION TESTING & INSPECTION

March 15, 2024

TES No. 240005.001

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Project: Proposed Sports Complex
Clovis Community College
10309 N. Willow Avenue
Clovis, California

Subject: Geotechnical Investigation and Geologic-Seismic Hazards Evaluation Report

Dear Ms. Robertson:

The attached report presents the results of a geotechnical investigation and geologic-seismic hazards evaluation for the proposed sports complex to be located in Clovis, California. This report describes the investigation, findings, conclusions, and recommendations for use in project design and construction.

TECHNICON Engineering Services, Inc. (TECHNICON) appreciates the opportunity to provide geotechnical engineering services to State Center Community College District during the design phase of this project. We trust this information meets your current needs. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully submitted,

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**GEOTECHNICAL INVESTIGATION AND GEOLOGIC-
SEISMIC HAZARDS EVALUATION REPORT
PROPOSED SPORTS COMPLEX
CLOVIS COMMUNITY COLLEGE
10309 N. WILLOW AVENUE
CLOVIS, CALIFORNIA**

Prepared for:

State Center Community College District
1171 Fulton Street, Room 617
Fresno, California 93721

March 15, 2024

TES No. 240005.001



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GEOTECHNICAL & ENVIRONMENTAL ENGINEERING — CONSTRUCTION TESTING & INSPECTION

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**GEOTECHNICAL INVESTIGATION AND GEOLOGIC-SEISMIC
HAZARDS EVALUATION REPORT
PROPOSED AGTEC INNOVATION CENTER
MERCED COLLEGE
3600 M STREET
MERCED, CALIFORNIA**

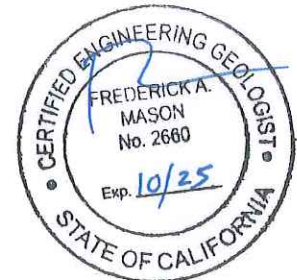
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**GEOTECHNICAL INVESTIGATION AND GEOLOGIC-SEISMIC
HAZARDS EVALUATION REPORT
PROPOSED SPORTS COMPLEX
CLOVIS COMMUNITY COLLEGE
10309 N. WILLOW AVENUE
CLOVIS, CALIFORNIA**

1 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation for the proposed sports complex to be constructed at the existing Clovis Community College in Clovis, California. The purpose of the investigation was to explore and evaluate the subsurface conditions at the site to develop geotechnical recommendations for project design and construction.

The Vicinity Map, presented on Figure 1, shows the general location of the project and the Site Map, presented on Figure 2, shows the proposed improvements and the boring locations for this investigation.

A geologic-seismic hazards evaluation was prepared concurrently with the geotechnical investigation and is incorporated into Sections 3 through 5 of this report. References reviewed during preparation of the geologic and seismic hazards section of this report are listed in Section 10, "References".

1.2 LOCATION

The project is located in northeastern Fresno County, northwest of E. Behymer Avenue and N. Willow Avenue in Clovis, California. Based on the Friant, California 7 ½-minute quadrangle topographic map, the site lies within the southwest quarter of Section 13, R20E and T12S. The elevation of the site is approximately 381 feet above the Mean Sea Level. Based on the USGS 7½-minute topographic map, the site coordinates are approximately:

Latitude: 36.8825° N
Longitude: 119.7338° W

1.3 PROPOSED CONSTRUCTION

The project involves the design and construction of a sports complex that includes a new athletic track, home and visitor bleachers, a single-story building, and sports lighting. The proposed

building is anticipated to consist of a wood-framed structure utilizing spread footings and a concrete slab-on-grade floor. The proposed bleachers are anticipated to be premanufactured and supported on concrete slabs-on-grade. The sports lighting will be supported on pier/pole foundations. Maximum wall and column loading are anticipated to be less than 3 kips per foot and 30 kips, respectively. Appurtenant improvements is estimated to include asphalt and Portland cement concrete pavements, underground utilities, concrete flatwork, and landscaping. Cut and fill elevations are anticipated to be minor, less than 1 to 2 feet to achieve a level pad grade and positive site drainage.

1.4 PURPOSE AND SCOPE OF SERVICES

The purpose of the investigation and evaluation was to explore the site subsurface conditions and evaluate pertinent geologic and seismic data to develop recommendations and opinions to aid in project design, approval, and construction. The scope of services consisted of field exploration, laboratory testing, design analysis, and preparation of this written report as described in **TECHNICON** proposal, dated January 4, 2024 (TES No. GP23-252). This Geotechnical Investigation and Geologic-Seismic Hazards Evaluation Report includes the following:

- ☐ A description of the proposed project, including a vicinity map showing the location of the site and a site plan showing the exploration locations;
- ☐ A description of the site surface and subsurface conditions encountered during the field investigation, including boring logs;
- ☐ A summary of the field exploration and laboratory testing program;
- ☐ Comments on regional and site engineering geology and seismology;
- ☐ Determination of peak horizontal ground surface acceleration utilizing the mapped spectral acceleration parameters of the 2022 California Building Code (CBC);
- ☐ Discussion of geologic hazards affecting the site and project, including liquefaction, seismically induced settlement, landslides, flooding, etc;
- ☐ Site preparation and earthwork, including the use of on-site soils for engineered fill and recommended import fill specifications;
- ☐ Spread footing design, including bearing capacity of foundation soil for sustained loading and total combined loading, embedment depths and anticipated total settlements;
- ☐ Resistance of lateral loads, including passive pressure and coefficient of friction;
- ☐ Design of pier foundations including axial and lateral capacity;

- ☐ Design factors for earth retaining structures;
- ☐ Design of concrete slabs-on-grade for buildings, including modulus of subgrade reaction;
- ☐ Recommendations for asphalt concrete and Portland cement concrete pavement design;
- ☐ Comments on the corrosion potential of on-site soil to buried metal and concrete;
- ☐ Comments to aid in the design of on-site drainage.

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

The field exploration, conducted on January 12 and 15, 2024 consisted of drilling ten (10) exploratory test borings, and a site reconnaissance by a staff engineer. The test borings were drilled with a SIMCO 2800 truck-mounted drill rig using 4-inch diameter solid flight auger and extended to depths of 16.5, 21.5 and 51.5 feet below existing ground surface (bgs). Additionally, three (3) locations were drilled to a depth of 5 feet bgs for R-value sample collection. The approximate locations of the test borings and R-values are indicated on the Site Map, Figure 2.

The soils encountered in the borings were visually classified in the field and a continuous log was recorded. Relatively undisturbed samples were collected from the test borings at selected depths by driving a 2.5-inch I.D. split barrel sampler containing brass liners into the undisturbed soil with a 140-pound automatic hammer free falling a distance of 30 inches. In addition, samples of the subsurface soils were obtained using a 1.4-inch I.D. standard penetrometer, driven 18 inches in accordance with ASTM D1586 test procedures. The sampler was used without liners. Resistance to sampler penetration was noted as the number of blows per foot over the last 12 inches of sampler penetration on the boring logs. The blow counts listed in the boring logs have not been corrected for the effects of overburden pressure, rod length, sampler size, boring diameter, or hammer efficiency. Bulk samples were also retained from auger cuttings of the near surface soils at selected test boring locations.

2.2 FIELD AND LABORATORY TESTING

Penetration rates, determined in general accordance with ASTM D1586, were used to aid in evaluating the consistency, compression, and strength characteristics of the foundation soils.

Laboratory tests were performed on selected near surface samples to evaluate their physical characteristics. The following laboratory tests were used to develop the design geotechnical parameters:

- ☐ Unit weight (ASTM D2937)
- ☐ Moisture Content (ASTM D2216)
- ☐ Sieve Analysis (ASTM C136)

- ☐ Expansion Index (ASTM D3080)
- ☐ Direct Shear (ASTM D3080)
- ☐ Soluble Sulfate and Soluble Chloride Contents (California Test Method No. 417 & 422)
- ☐ pH and Minimum Resistivity (California Test Method No. 643)
- ☐ Collapse Potential (ASTM D5333)
- ☐ Resistance Value (Caltrans Test Method No. 301)

The dry density and moisture content test results are shown on the boring logs in Appendix A. The soluble sulfate, soluble chloride, pH, and minimum resistivity are discussed in Section 7.7, "Corrosion Potential". The remaining test results are provided in Appendix B.

3 SITE AND GEOLOGIC CONDITIONS

3.1 REGIONAL GEOLOGY

The site lies within the central east portion of the San Joaquin Valley, within the Great Valley geomorphic province of California (CGS, 2002). The Central Valley is between the Sierra Nevada geomorphic province to the east, and the Coastal Ranges geomorphic province to the west. The thick sequence of sediments that form the valley floor were eroded from these adjacent mountain regions and have been accumulating since the Jurassic period, about 160 million years.

The regional bedrock forms an asymmetrical trough, which is deepest near the western margin. The surficial sediments filling the trough include deposits of alluvial fans, flood plains, marshes, and lakes (Croft, 1972). The regional geologic map is presented on Figure 3.

3.2 AREA AND SITE GEOLOGY

The geology at the site is mapped as Pleistocene aged nonmarine deposits (Qc), described as older alluvium of consolidated and dissected fan deposits comprised of sand, gravel, and cobbles. The soil subgrade characteristics encountered during the field investigation (i.e. soil type, blow count, etc.) are representative of these sediments. Figure 4 presents a site-specific geologic map of the project.

3.3 SURFACE CONDITIONS

At the time of investigation, the project site consisted of a vacant lot that supported a moderate growth of seasonal grasses and weeds. The site is generally bounded by the Clovis Community College to the north, parking lots to the east, E. Behymer to the south, and a water treatment facility to the west. The overall site topography is relatively flat and approximately 1 foot above the elevation of E. Behymer Street.

3.4 EARTH MATERIALS

The subsurface soils consist of Pleistocene aged nonmarine deposits (QC). The earth material encountered by the subsurface exploration consisted of clayey sand and sandy silt in the upper 8 to 15 feet and underlain by laterally discontinuous layers of clayey sand, sandy clay, sandy silt, and poorly graded sand extending to the maximum depth explored, 51.5 feet bgs. The granular

soils generally had a relative density of medium dense to very dense and the fine grained soils had a consistency of very stiff to hard.

The above is a general description of the earth material profile. A more detailed representation of the stratigraphy at the specific exploration locations is provided on the boring logs in Appendix A and the cross sections on Figures 5 and 6.

3.5 GROUNDWATER CONDITIONS

Groundwater was not encountered within depth of exploration, 51.5 bgs. The California Department of Water Resources "Sustainable Groundwater Management Agency Data Viewer" Spring 2023, indicates the current groundwater depth in the area is greater than 100 feet bgs. Research utilizing the California Department of Water Resources (DWR) website shows the nearest well with recorded data to be approximately 0.35 miles to the southeast (Well No. 12S21E19D001M). Based on the groundwater elevation data collected at this well, the most recent groundwater elevation data collected in late 2011 indicates a water surface level 158 feet bgs. Further review indicates this well had a historic high groundwater measurement in 1961 of 46 feet.

Groundwater conditions at the site could change in the future due to variations in rainfall, groundwater withdrawal, construction activities, or other factors not apparent at the time our test borings were made. However, groundwater is not anticipated to impact construction.

4 FAULTING AND SEISMICITY

4.1 HISTORICAL SEISMICITY

The project site is in a region traditionally characterized by moderate seismic activity. Seismic activity of the site was researched using information obtained from the U.S. Geologic Survey (USGS) and California Geologic Survey (CGS) websites, a catalog by the Advanced National Seismic System (ANSS) and Caltrans Acceleration Response Spectra (ARS).

Some of the historical earthquake events that caused significant shaking at the site are listed in Table 4.1-1.

TABLE 4.1-1
SIGNIFICANT REGIONAL EARTHQUAKE EVENTS

Earthquake Name	Year	Distance from Site (km)	Magnitude (Mw)
Great Fort Tejon	1857	88	7.9
Coalinga	1983	121	6.4
Owens Valley	1872	148	6.5
Ridgecrest	2019	228	7.1

Epicenters of significant earthquakes ($M \geq 5.5$) within the vicinity of the site are shown on Figure 7. Data for earthquakes that occurred from 1800 to 2022 have been obtained from the Significant California Earthquakes website (CGS, 2019) and a composite catalog by the ANSS. The ANSS catalog is a worldwide earthquake catalog which is created by merging the master earthquake catalogs from contributing ANSS member networks and then removing duplicate events, or non-unique solutions from the same event. The ANSS network includes the Northern and Southern California Seismic Networks, the Pacific Northwest Seismic Network, the University of Nevada, Reno Seismic Network, the University of Utah Seismographic Stations, and the United States National Earthquake Information Service. The earthquake database also consists of earthquake records between 1800 and 1900 from Seeburger and Bolt (1976) and Topozada et al. (1978 and 1981).

4.2 FAULTS LOCAL TO THE PROPOSED SITE

The site is not located in an Alquist-Priolo Earthquake Fault Zone as established by the Alquist-Priolo Fault Zoning Act (Section 2622 of Chapter 7.5, Division 2 of the California Public Resources Code).

The CGS Fault Activity Map of California (2010) was reviewed to determine if identified active faults are located on or near the subject site. According to the map, no identified active faults are located on or near the subject site. Locations of active and late Quaternary faults in the area with respect to the subject site are shown on Figure 8, Regional Fault Activity Map (obtained from the Fault Activity Map of California, Jennings, Bryant and Saucedo, 2010).

Based on review of published data and current understanding of the geologic framework and tectonic setting of the proposed improvements, the primary sources of seismic shaking at this site are listed in Table 4.2-1. The table also provides the fault type, distance from the site, and maximum moment magnitude (M_w). A major seismic event on these or other nearby faults may cause ground shaking at the site. Based on the deterministic ground acceleration, the San Andreas Fault, located west of the site, is considered the governing fault.

**TABLE 4.2-1
PRIMARY SOURCES OF SEISMIC SHAKING**

Fault Name	Fault Type	Distance from Site (miles)	Magnitude (M_w)
Great Valley	Thrust	47	6.6
Ortogonalita	Right Lateral/ Strike Slip	65	7.0
Round Valley	Normal	66	7.0
San Andreas	Right Lateral/ Strike Slip	74	6.7

4.3 SITE CLASS

Based on the field exploration, the site soil is classified as Site Class D as presented in ASCE 7-16 based on the average Standard Penetration Tests (N value) at the project site. Site Class D is defined as a stiff soil profile with shear wave velocities between 600 feet/sec and 1,200 feet/sec,

or Standard Penetration Resistance (N) between 15 to 50 blows/foot, or undrained shear strength (S_u) between 1,000 to 2,000 psf for the upper 100 feet.

4.4 GENERAL PROCEDURE SEISMIC DESIGN CRITERIA

In accordance with CBC 1613A.2 a general procedure ground motion analysis was performed. USGS seismic design mapped values were obtained for the project site utilizing a Site Class D, and site coordinates from the Structural Engineers Association of California (SEAOC) website (<http://seismicmaps.org>). The values obtained are provided in the table below.

TABLE 4.4-1
2022 CBC/ASCE 7-16 GENERAL PROCEDURE GROUND MOTION PARAMETERS

Seismic Item	Design Value	Seismic Item	Design Value
Site Class	D	Seismic Design Category	D
S_s	0.531	S_{MS}	0.730
S_1	0.213	S_{M1}	0.463
Site Coefficient, F_v	2.174*	S_{DS}	0.487
Site Coefficient, F_a	1.375	S_{D1}	0.309
T_s	0.634		

*This value of F_v should only be used for calculation of T_s . See Section 11.4.8 of ASCE 7-16

A probabilistic seismic hazards analysis (PSHA) procedure was performed using the USGS Unified Hazard Tool to estimate the earthquake magnitude. The program allows user input of the project site coordinates and produces the expected peak ground motions for selected probability of exceedance (e.g., return periods). Based on a probability of exceedance of 2 percent in 50 years, the USGS Unified Hazard Tool determined a peak ground acceleration of 0.332g and a weighted magnitude of $M_w = 6.15$.

4.5 SITE SPECIFIC SEISMIC DESIGN CRITERIA

In accordance with ASCE 7-16 11.4.8, since the project is in a site class D and the S_1 value is greater than 0.2 (0.213g) a site-specific ground motion hazard analysis was performed. The analysis followed the requirements of ASCE 7-16, Sections 21.2 through 21.5, as well as ASCE 7-16, Supplement No. 1 and No. 3, and 2022 CBC 1830A.6.

The following steps were utilized for determining the site-specific ground motion parameters: Seismic design parameters were obtained for the project site utilizing a Site Class D, and site coordinates from the Structural Engineers Association of California (SEAOC) website (<http://seismicmaps.org>). The USGS Unified Hazard Tool and the Risk-Targeted Ground Motion calculator was used to calculate the probabilistic ground motion response spectrum in accordance with ASCE 7-16 Section 21.2.1.2 Method 2. The 2014 NGA West2 – GMPEs worksheet from the Pacific Earthquake Engineering Research Center was then used to calculate deterministic spectral response acceleration as an 84th-percentile 5% damped spectral response acceleration in the maximum horizontal direction by using fault parameters and magnitude area relationships given by the USGS Unified Hazard Tool in accordance with ASCE 7-16 Section 21.2.2. Supplement No. 3 indicates that projects located in Site Class D should increase S_{M1} by 50 percent in Equation 11.4-2. This increase results in a 50 percent increase of S_{D1} in Equation 11.4-4. These increased values are to be used for all applications and formulation of the design response spectrum. The Site-Specific MCE_R was then calculated by a single factor such that the maximum response spectral acceleration equals $1.5F_a$, with F_a determined using Table 11.4.1 in the ASCE 7-16. In accordance with ASCE 7-16 Section 21.3, the design spectral response had to be checked that no period shall be taken as less than 80% of S_a determined in accordance with Section 11.4.6, where F_a is determined using Table 11.4.1 and F_v is taken as 2.4 for $S_1 < 0.2$ or 2.5 for $S_1 > \text{or equal to } 0.2$. After checking design spectrum is greater than 80% of code-based spectrum for all periods, using the design spectrum graph, design acceleration parameters such as S_{DS} is taken as 90% of max S_a between periods $T=0.2$ and 5 seconds and parameter S_{D1} taken as the maximum value of the product, TS_a , for periods from 1 to 5 seconds for sites with $V_s < 365.76$ m/s in accordance with ASCE 7-16 Section 21.4. The parameters S_{MS} and S_{M1} are then taken as 1.5 times S_{DS} and S_{D1} , respectively. Lastly, the maximum considered earthquake geometric mean peak ground acceleration is taken by comparing deterministic peak ground acceleration from 84th spectral acceleration at $T=0.01$ seconds to $0.5F_{PGA}$, following with the greater of those two values being compared to the probabilistic peak ground acceleration, with the lesser of the two values being the site-specific peak ground acceleration (0.332) in accordance with ASCE 7-16, Section 21.5. Based on this analysis, a peak ground acceleration of 0.332 is recommended for the evaluation of liquefaction. The site specific ground motion analysis is included in Appendix D.

TABLE 4.5-1
2022 CBC/ASCE 7-16 SITE SPECIFIC GROUND MOTION PARAMETERS

Seismic Item	Design Value	Seismic Item	Design Value
Site Class	D	Seismic Design Category	D
S_s	0.531	S_{MS}	0.840
S_1	0.213	S_{M1}	0.653
Site Coefficient, F_v	2.500	S_{DS}	0.560
Site Coefficient, F_a	1.375	S_{D1}	0.435
T_s	1.093		

5 GEOLOGIC AND SEISMIC HAZARDS

5.1 GENERAL

A discussion of specific geologic hazards that could impact the site is included below. The hazards considered include: surface fault rupture; seismically induced ground failures (liquefaction, lateral spreading, dynamic compaction, and landslides), general flooding and seismically induced flooding (tsunami, seiche, and dam failure); and hydrocompactive, expansive, and corrosive soils.

5.2 SURFACE FAULT RUPTURE

The site is not in an Alquist-Priolo Earthquake Fault Zone. Based upon the reviewed geologic and seismologic reports, maps, and aerial photographs, no mapped active faults cross or project toward the site. Additionally, no evidence of active faulting was visible on the site during our site reconnaissance. Therefore, it is our opinion that the potential for fault-related surface rupture at the project site is very low.

5.3 SEISMICALLY INDUCED GROUND FAILURE

5.3.1 Liquefaction

In order for soil liquefaction due to ground shaking, and possible associated effects to occur, it is generally accepted that four conditions are required:

- ☐ The subsurface soils are in a relatively loose state,
- ☐ The soils are saturated,
- ☐ The soils are fine, granular, and uniform, and
- ☐ Ground shaking of sufficient intensity to act as a triggering mechanism.

Geologic age also influences the potential for liquefaction. Sediments deposited within the past few thousand years are generally much more susceptible to liquefaction than older Holocene sediments; Pleistocene sediments are often more resistant; and pre-Pleistocene sediments are generally immune to liquefaction (Youd, et al., 2001).

Saturated granular sediments can experience liquefaction if subject to seismically induced ground motion of sufficient intensity and duration. Liquefaction analysis used procedures by

Youd et. al. (2001) and considered the relative density and fines content of the granular sediments. The analysis considered a historical high design groundwater depth of 46 feet bgs and measured groundwater depth of greater than 50 feet bgs, ground acceleration (PGA_M) of 0.332g, and earthquake moment magnitude, $M_w = 6.15$.

The coarse-grained layers of sand were evaluated for potential liquefaction using the cyclic liquefaction analysis model by Youd et. al. (2001). Liquefaction analysis performed on the granular sediments indicates that liquefaction and seismically induced settlement is not likely to occur.

Seismically induced settlement due to liquefaction was evaluated to be negligible. The general guidelines of the CGS indicate the differential seismically induced settlement across a building would be about one-half the total settlement. This would also result in differential settlement across buildings to be negligible. The estimated differential settlement is anticipated to be within the tolerance of the proposed structures and will not result in significant damage or collapse. Therefore, no mitigation against liquefaction and/or settlement is necessary. The liquefaction and settlement calculations are included in Appendix E.

5.3.2 Dynamic Compaction

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. Dry sand settlement will be minimal (0.1-inch), and mitigation measures are not warranted.

5.3.3 Landslides and Ground Failure

According to the Fresno County General Plan (FCGP, 2024), the county has identified areas of seismic hazards and landslide susceptibility. Based on the mapped area, the project site does not lie in a seismic hazards zone that has a susceptibility to landslides.

Since the project site is located on relatively flat terrain, the potential for landslides or other slope failures from earthquake-induced ground shaking is unlikely. Furthermore, strong shaking also has the potential for activating slope failures on creek banks (lurch cracking) and tension cracking in areas underlain by loose, low density soil such as uncompacted fill. Since the project

site is not located near any creek banks, the potential for landslides or other slope failures from earthquake-induced ground shaking is considered unlikely.

5.4 FLOODING

5.4.1 Tsunamis, Seiches, Earthquake Induced Flooding

Tsunamis are sea waves of unusual size that occur from significant earthquakes either under the ocean floor or adjacent to shorelines and can travel great distances to impact low-lying communities and developments. Considering that the Coast Range protects the site from the sea, the potential for the site to be affected by a tsunami is nil.

A seiche is a free or standing wave oscillation that occurs in a confined body of water, such as a reservoir or lake. Earthquake-generated ground waves, which have a period that matches the natural period of the lake or reservoir, may cause the water to oscillate, which can cause damage to shoreline improvements. The FCGP indicates that earthquake-induced seiches are not considered a risk in in Fresno County

5.4.2 Potential for Inundation Due to Dam Failure

According to the California Department of Water Resources Dam Breach Inundation Map Web Publisher, there are no dams that would cause substantial flooding at the project area. Therefore, no mitigation measures are required.

5.4.3 Flood Insurance Rate Maps

According to the Federal Emergency Management Agency (FEMA), the project site lies within a Zone X and Zone A flood designation (Map Number 06019C1040H, dated February 18, 2009) indicating the area is determined to be outside the 0.2 percent annual chance floodplain and an area that is subject to inundation by the 1 percent annual flood with no base flood elevations determined. The civil engineer should plan site grades accordingly.

5.5 EXPANSIVE SOILS

One (1) Expansion Index (EI) test was performed on a soil sample collected from the near surface soils of the site. The test indicated the near surface soils have a very low potential for expansion as indicated by an EI of 4. The soils are not susceptible to volume changes

associated with changes in soil moisture content. Therefore, no mitigation is needed for expansive soils.

5.6 HYDROCOMPACTION (SOIL COLLAPSE)

Our experience has found that some of the alluvial soils in the San Joaquin Valley are subject to hydrocompaction. Hydrocompactive soil has a relatively loose skeletal structure, which is weakly cemented by soluble salts or a slight clay mineral content. Moisture increase breaks down the inter-particle cementation causing a collapse of the skeletal structure. The significant loss in soil volume can result in settlement of overlying structures. The geotechnical exploration and laboratory testing identified that hydrocompactive characteristics were minimal. Based on the laboratory testing, post saturation of soil samples obtained from the site indicated moderate collapse potential upon inundation. Analysis indicates that settlement due to hydrocompaction is approximately 0.85 inches. The hydrocompaction is indicative of near surface disturbed soils. The earthwork recommendations in Section 6 require over-excavation to recompact the upper 3 feet of the site to mitigate the hydrocompactive soils.

5.7 CORROSIVE SOILS

The corrosion characteristics of the near surface foundation soils and any necessary mitigation measures are discussed in Section 7.7, "Corrosion Potential".

5.8 REGIONAL SUBSIDENCE

Land subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. The FCGP does not identify specific areas of subsidence within the City limits; however, FCGP acknowledges subsidence is possible. Furthermore, it is noted that on rare occasions subsidence may occur due to earthquake-induced ground movement. Due to the significant depth to groundwater withdraw in the San Joaquin Valley, the occurrence of subsidence is typically regional and unlikely to affect isolated locations, as such, the potential for damaging differential settlement of the proposed building due to subsidence is very low.

6 EARTHWORK

6.1 GENERAL

Based on the laboratory data, field exploration, and geotechnical analyses, it is feasible to construct the proposed sports complex as currently envisioned. The use of spread and continuous reinforced concrete footings bearing on engineered fill are considered appropriate for structure support provided that the recommendations presented in this report are incorporated into the project design and construction.

Site grading recommendations are presented in subsequent sections of this report. All references to relative compaction, maximum density, and optimum moisture are based on ASTM Test Method D1557. All earthwork should extend a minimum of 5 feet beyond the perimeter of proposed improvements.

6.2 SITE PREPARATION

6.2.1 Stripping

All surface vegetation and any miscellaneous surface obstructions should be removed from the project area, prior to any site grading. It is anticipated that stripping of vegetation and grass landscape will involve the upper 1 to 3 inches. Surface strippings should not be incorporated into fill unless they can be sufficiently blended to result in an organic content less than 3 percent by weight (ASTM D2974). Stripped topsoil, with an organic content between 3 and 12 percent by weight, may be stockpiled and used as non-structural fill (i.e. on landscape areas). If used in landscape areas, soil with an organic content between 3 and 12 percent should be placed within 2 feet of finished grade, and at least 5 feet outside of building perimeters. Soil with an organic content greater than 12 percent by weight should be excluded from fill.

6.2.2 Disturbed Soil, Undocumented Fill and Subsurface Obstructions

Initial site grading should include a reasonable search to locate disturbed soil, undocumented fill soils, debris, abandoned underground structures, and/or existing utilities that may exist within the area of construction. All underground utilities should be rerouted beyond the perimeter of the proposed improvements and all previous trench backfill and any loose soils generated by the utility removal should be removed to expose undisturbed native soil. If any areas or pockets of soft or loose soils or void spaces made by burrowing animals, undocumented fill, or other

disturbed soil are encountered, they should be excavated to expose approved undisturbed native soil. Excavations for removal of the above items should be dish-shaped and backfilled with engineered fill (see Section 6.3).

6.2.3 Over-Excavation

After performing the removals described in Sections 6.2.1 and 6.2.2, the proposed project site should be over-excavated a minimum depth of 3 feet below existing ground surface to mitigate hydrocompactive soils. The bottom of the excavation should be processed in accordance with Section 6.2.4 and the scarified soil should be recompact to at least 90 percent relative compaction. The lateral limits of the over-excavation should extend at least 5 feet beyond the perimeter of the proposed improvements. The over-excavation is intended to mitigate the observed hydrocompactive soils.

6.2.4 Scarification and Compaction

After stripping the site, over-excavation, and any elective removals, the exposed subgrade soil to receive fill or areas to support proposed foundations/improvements should be scarified to a minimum depth of 8 inches, uniformly moisture conditioned, and compacted to at least 90 percent relative compaction. Soft or pliant areas should be excavated to expose firm undisturbed soil approved by the project Geotechnical Engineer as described in section 6.2.2.

6.2.5 Construction Considerations

Should site grading be performed during or subsequent to wet weather, near-surface site soils may be significantly above optimum moisture content. These conditions could hamper equipment maneuverability and efforts to compact site soils to the recommended compaction criteria. Disking to aerate, chemical treatment, replacement with drier material, stabilization with a geotextile fabric or grid, or other methods may be required to mitigate the effects of excessive soil moisture and facilitate earthwork operations. Any consideration of chemical treatment (e.g. lime) to facilitate construction would require additional soil chemistry evaluation and could affect landscape areas and some construction materials.

6.3 ENGINEERED FILL

6.3.1 Materials

All engineered fill soils should be nearly free of organic or other deleterious debris and less than 3 inches in maximum dimension. The on-site soil exclusive of debris may be used as engineered fill, provided it contains less than 3 percent organics by weight (ASTM D2874).

Recommended requirements for any imported soil to be used as engineered fill, as well as applicable test procedures to verify material suitability, are provided on Table 6.3-1.

**TABLE 6.3-1
IMPORT FILL CRITERIA**

<u>Gradation</u> (ASTM C136)			
<u>Sieve Size</u>		<u>Percent Passing</u>	
76 mm (3-inch)		100	
19 mm (¾-inch)		80 – 100	
No. 4		60 – 100	
No. 200		20 – 50	
<u>Expansion Index</u> (ASTM D4829)		<u>Plasticity</u> (ASTM D4318)	
		<u>Liquid Limit</u>	<u>Plasticity Index</u>
< 20		< 25	< 9
<u>Organic Content</u> (ASTM D 2974)			
< 3% by dry weight			
<u>Corrosivity</u>			
pH	Minimum Resistivity (ohm-cm)	Soluble Sulfate (ppm)	Soluble Chloride (ppm)
6 to 8	> 2,000	< 2,000	< 500
<u>Resistance Value</u> (California Test Method No. 301)			
R-value = 13			

The import criteria for corrosion are typical threshold limits for non-corrosive soil. All imported fill materials to be used for engineered fill should be sampled and tested by a representative of the

project Geotechnical Engineer prior to being transported to the site. In addition, import fill should meet the requirements of the Department of Toxic Substances Control (DTSC), Information Advisory for Clean Imported Fill Material. The purpose of testing import soils is to ensure that “clean” fill soils are imported to otherwise “clean” sites. The testing does not require notification of the DTSC, rather the testing should be performed as part of the routine due diligence of constructing on state property and the results filed with the school district.

6.3.2 Compaction Criteria

Soils used as engineered fill should be uniformly moisture-conditioned to at least optimum moisture, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction. Disking and/or blending may be required to uniformly moisture condition soils used for engineered fill.

The upper 12 inches of pavement subgrade should be compacted to at least 95 percent relative compaction. Relative compaction is to be determined by Caltrans No. 216 (dry weight determination) or ASTM D1557 test procedures.

6.4 TEMPORARY EXCAVATIONS

6.4.1 General

All excavations must comply with applicable local, State, and Federal safety regulations including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. Construction site safety is generally the responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. The information provided is a service to the client. Under no circumstances should the information provided be interpreted to mean that **TECHNICON** is assuming responsibility for construction site safety or the Contractor’s activities; such responsibility is not being implied and should not be inferred.

6.4.2 Excavations and Slopes

The Contractor should be aware that slope height, slope inclination, or excavation depths (including utility trench excavations) should in no case exceed those specified in local, State, and/or Federal Safety regulations (e.g., OSHA health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations). All excavations should be constructed and

maintained in conformance with current OSHA requirements (29 CFR Part 1926) for a Type C (Clayey SAND) soil.

6.4.3 Construction Considerations

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should be kept sufficiently away from the top of any excavation to prevent any unanticipated surcharging. If it is necessary to encroach upon the top of an excavation, **TECHNICON** can provide comments on slope gradients or loads on shoring to address surcharging, if provided with the geometry. Shoring, bracing, or underpinning required for the project (if any), should be designed by a professional engineer registered in the State of California.

During wet weather, earthen berms or other methods should be used to prevent run-off water from entering all excavations. All run-off should be collected and disposed of outside construction limits.

TRENCH BACKFILL

6.4.4 Materials

Pipe zone backfill (i.e., material beneath and in the immediate vicinity of the pipe) should consist of soil compatible with design requirements for the specific types of pipes. It is recommended that the project designer or pipe supplier develop the material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this investigation. Randomly excavated near surface soil will likely be Class III material per ASTM D2321. Trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of native soil which meets the requirements for engineered fill.

6.4.5 Compaction Criteria

All trench backfill should be placed and compacted in accordance with recommendations provided for engineered fill. Mechanical compaction is recommended; ponding or jetting should not be used.

7 DESIGN RECOMMENDATION

7.1 GENERAL

The proposed structures may be supported by conventional shallow spread footings and pier foundations supported on properly engineered fill. The following recommendations are based on the assumption that the recommendations in Section 6, “Earthwork”, have been implemented. Recommendations regarding the geotechnical aspects of building design are presented in subsequent sections.

7.2 SPREAD FOOTINGS

7.2.1 Vertical Bearing Pressures and Settlements – Strip and Spread Foundations

Generally, two geotechnical issues determine the design bearing pressure for conventional spread footing foundations: strength of the foundation soil, and tolerable settlement. For lightly loaded structures, design bearing may be determined by constructability considerations or code-required minimum dimensions.

Table 7.2-1 presents the allowable available bearing capacity for static loading which includes dead load plus live load (D.L. + L.L.) and total combined loading (D.L. + L.L. + transient loading, such as wind or seismic), and unfactored nominal bearing.

**TABLE 7.2-1
BEARING CAPACITY**

	Bearing Capacity (psf)
Static Loading	$665 B + 1,345 D$
Total Combined Loading	$1,000 B + 2,015 D$
Unfactored Ultimate Bearing	$1,995 B + 4,030 D$

Note: B is footing width in feet and D is the footing embedment depth in feet

The above values are appropriate for design using the Basic and Alternate Load Combinations in Section 1605.3 of the 2022 CBC. To simplify design, an allowable bearing pressure of 2,000 psf (static loading, D.L. + L.L.) could be considered. The bearing pressure could be increased 50 percent for evaluating transient loads, such as, wind or seismic.

If evaluating the foundation as a beam on an elastic foundation, a modulus of subgrade reaction, K_p ($B_p = 1$ foot), of 300 pci can be used for undisturbed on-site soil. The subgrade modulus is most appropriately applicable to consideration of static loads with deformations within an elastic range.

Analysis, based on Schmertmann, determined the following estimated static settlement based on assumed structural loads. The settlement assumes the sustained load on the footings is equal to 80 percent of the total load. Settlement is expected to occur rapidly with load application.

**TABLE 7.2-2
ESTIMATED SETTLEMENT**

Footing Type	Loading (DL + LL)	Design Bearing (psf)	Estimated Settlement (inch)
Strip	3 kips/ft	2,750	Less than 0.50
Square	30 kips	3,800	Less than 0.50

If deemed necessary by the design engineer, **TECHNICON** can provide the estimated settlement for other loading conditions.

7.2.2 Lateral Resistance

Lateral loads applied to foundations can be resisted by a combination of passive lateral bearing and base friction. Table 7.2-3 presents the allowable and ultimate passive pressures and frictional coefficients.

**TABLE 7.2-3
PASSIVE PRESSURES AND FRICTIONAL RESISTANCE**

	Allowable		Ultimate
	Static	Total Combined	
Frictional Coefficient	0.43	0.52	0.65
Passive Pressure (psf/ft)	375	500	750
Lateral Translation Needed to Develop Passive Pressure	0.004 D	0.007 D	0.022 D

Note: 1) D is the footing depth (ft)

If the deflection resulting from the strain necessary to develop the passive pressure is beyond structural tolerance, additional passive pressure values could be provided based on tolerable deflection. The passive pressure and frictional resistance can be used in combination. The

allowable values already incorporate a factor of safety and, as such, would be compared directly to the driving loads. If analytical approaches require the input of a safety factor, the ultimate values would be used.

7.2.3 Design and Construction Considerations

Prior to placing steel or concrete, footing excavations should be cleaned of all debris, loose soft soil, and water. All footing excavations should be observed by a representative of the project Geotechnical Engineer immediately prior to placing steel or concrete. The purpose of these observations is to verify that the bearing soils encountered in the foundation excavations are similar to those assumed in the analysis and to verify these recommendations are implemented.

7.3 EARTH RETAINING STRUCTURES

If project improvements will include retained earth systems, the lateral earth pressure against retaining structures will be dependent upon the ability of the wall to deflect. Presented in Table 7.3-1 are the active, at-rest, and braced lateral earth pressures for on-site soil. The active pressure is applicable to walls able to rotate 0.0005 radians at the top or bottom. The at-rest soil pressure is applicable to retaining structures that are fully fixed against both rotation and translation. Walls restrained from translation at the top and bottom, but able to deflect 0.0005 radian between restrained points should be designed for the braced lateral pressure.

**TABLE 7.3-1
LATERAL EARTH PRESSURES**

	Lateral Earth Pressures
Active Pressure (psf/ft of depth)	38
At-Rest Pressure (psf/ft of depth)	59
Braced Pressure (psf)	25 H

Note: H in the expression represents the retained height in feet (measured from finished grade to bottom of footing).

The recommended values incorporate saturated soil conditions but not the lateral pressure due to hydrostatic forces. Wall backfill should be adequately drained.

Retaining wall foundation design can utilize the passive pressures and frictional resistance given in Table 7.2-3 and the bearing capacities given in Table 7.2-1. When utilizing the bearing capacities of Table 7.2-1, the static loading value represents the average bearing for the footing

and the total combined loading value presents the allowable maximum toe pressure.

7.4 SLABS-ON-GRADE

7.4.1 Subgrade Preparation

Slabs-on-grade should be supported on recompacted soils or engineered fill placed as described in Section 6.3 of this report. Subgrade soils within 12 inches of pad grade should have a moisture content of at least optimum, immediately prior to placing the slab concrete, or placing the vapor retarding membrane.

7.4.2 Capillary and Moisture/Vapor Break

Considering the soil type and regional groundwater depth, a capillary break (i.e. clean sand or gravel layer) is not considered necessary.

In areas to receive moisture-sensitive floor coverings, it is recommended that the subgrade be covered by a 10 mil vapor retarding membrane meeting the specifications of ASTM E1745, (Class C with minimum puncture resistance of 475 grams). The subgrade surface should be smooth and care should be exercised to avoid tearing, ripping, or otherwise puncturing the vapor retarding membrane. If the vapor retarding membrane becomes torn or disturbed, it should be removed and replaced or properly patched. Considering the soil type and regional groundwater depth, a capillary break (i.e., clean sand or gravel layer) is considered unnecessary.

The vapor retarding membrane could be covered with approximately 1 to 2 inches of saturated surface dry (SSD) sand to protect it during construction. Concrete should not be placed if sand overlying the vapor barrier has been allowed to attain a moisture content greater than about 5 percent (due to precipitation or excessive moistening). In addition, penetrations through the concrete slab shall be sealed or protected to prevent inadvertently introducing excess water into the sand cushion layer due to curing water, wash-off water, rainfall, etc. Excessive water beneath interior floor slabs could result in future significant vapor transmission through the slab, adversely affecting moisture-sensitive floor coverings and could inhibit proper concrete curing.

According to American Concrete Institute (ACI) 302.2R-06, concrete could be placed directly on the vapor retarding membrane to minimize the potential for developing a reservoir of moisture in the sand layer, which could lead to future moisture entrapment and potential moisture and flooring problems. If concrete is placed directly on the membrane, care should be taken to not

damage the membrane and special concrete curing methods implemented to minimize potential slab curing problems. If the protective sand layer is not used, the building designer should be in agreement. Many slab designers feel the sand cushion is important to proper concrete curing as well as minimizing slab curling issues.

Although slab support currently the industry standard, this system might not be completely effective in preventing floor slab moisture vapor transmission problems. This system will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels will not inhibit mold growth. A qualified specialist(s) with knowledge of slab moisture protection systems, flooring design and other potential components that may be influenced by moisture, should address these post-construction conditions separately. The purpose of a geotechnical investigation is to address subgrade conditions only, and consequently, it does not evaluate future potential conditions.

7.4.3 Conventional Slab Design

There are no geotechnical considerations (e.g., expansive soil), which would require special design of slabs. Therefore, the thickness and reinforcement of slabs-on-grade should be determined by structural considerations and should be designed by the project structural engineer or building designer. A modulus of subgrade reaction, K_p ($B_p = 1$ foot), of 300 pci may be used for elastic analysis of slabs on properly compacted subgrade.

Slab concrete should have good density, a low water/cement ratio, and proper curing to promote a low porosity and reduce moisture vapor transmission.

7.5 PIER FOUNDATIONS

Pier foundations may be desirable for support of shade structures, lighting, etc. Presented in Table 7.5-1 are expressions for the allowable and ultimate friction resistance vales for vertical compression loads on pier foundations.

**TABLE 7.5-1
ALLOWABLE AXIAL CAPACITY**

	Frictional Resistance for Vertical Loads in Compression (lbs)
Static Loading	65 DL ²
Total Combined Loading	85 DL ²
Unfactored Ultimate Capacity	125 DL ²

Note: 1) D is pier diameter in feet and L is embedment length in feet.
2) The allowable uplift resistance would be 70 percent of the compressional resistance.

The allowable passive pressure to resist lateral loads on isolated piers may be taken as 215 psf per foot of depth of embedment. The value may be increased by one-third for the total combined loads, including wind and seismic. The passive pressure values already consider arching and, as such, should not be increased further. The passive pressure only considers soil strength. Tolerable pier deflection may govern the design lateral resistance. If provided with pier geometry, lateral load, and loading eccentricity, **TECHNICON** can provide the estimated pier head deflection.

7.6 PAVEMENT DESIGN

7.6.1 Design R-value and Traffic Assumptions

The R-value for the on-site soil was evaluated in the laboratory on bulk samples of subgrade soil taken at three (3) locations from the upper 3 feet within proposed pavement areas. The tested soils had measured R-values of 13, 14, and 19. The laboratory testing conformed to Caltrans Test Method 301. Based on the tested values, an R-value of 13 is recommended for pavement design. If requested, additional samples could be collected after grading has been performed in order to reevaluate the design R-value.

Detailed vehicular load and frequency information was not provided for this project at the time this report was prepared. Traffic on the site is anticipated to consist of parking and drives for automobiles and regular school bus traffic. Consequently, a range of pavement sections have been provided based on Traffic Indexes (T.I.'s) of 4.5, 5.0, 6.0, 7.0, 8.0, 9.0 and 10.0. These traffic design assumptions should be reviewed for compatibility with the actual development, and revised pavement sections developed, as necessary.

7.6.2 Asphalt Concrete Pavement Design

Flexible pavement design recommendations have been developed for the given T.I.'s based upon the California Department of Transportation (Caltrans) design procedures and a design R-value of 13. The flexible asphalt concrete pavement sections associated with the assumed T.I.'s for on-site asphalt pavements are summarized in Table 7.6-1.

**TABLE 7.6-1
 RECOMMENDED MINIMUM PAVEMENT SECTIONS**

Traffic Index	Asphalt Concrete (inches)	Aggregate Base – Class 2 (inches)
4.5	2.5	8.0
5.0	2.5	9.5
6.0	3.5	11.0
7.0	4.0	13.5
8.0	4.5	16.5
9.0	5.5	18.0
10.0	6.0	21.0

The design criteria assumes a 20-year design period and that normal maintenance (crack sealing, etc.) is performed. The traffic index is a measure of the volume of truck traffic that will be applied to a pavement section in the design life. The allowable average daily truck traffic (ADTT) for the assumed traffic indexes is presented in Table 7.6-2.

**TABLE 7.6-2
 AVERAGE DAILY TRUCK TRAFFIC**

Traffic Index	2-Axle Vehicle	or	3-Axle Vehicle	or	5-Axle Vehicle
4.5	2.2		0.8		0.2
5.0	5.2		2.0		0.5
6.0	24.1		9.0		2.4
7.0	88.1		33.0		8.8
8.0	270.6		101.5		27.1
9.0	728.0		273.0		72.9
10.0	1764.7		661.8		176.7

The flexible pavement should conform to and be placed in accordance with the Caltrans Standard Specifications, 2022. The aggregate base (Class 2) should comply with the specifications in Sections 26. The aggregate base and upper 12 inches of subgrade should be compacted to a minimum of 95 percent relative compaction as determined by Caltrans Test Method 216 (Dry determination) or ASTM D1557 test procedures.

7.6.3 Moisture Considerations

The pavement design should consider both the vehicular loading, as well as the environmental factors. The vehicular loading will depend on the amount and type of traffic anticipated for the pavement design life. Environmental factors include the potential for moisture variations beneath the pavement structural section. It is recommended that all pavement areas conform to the following criteria:

- ☐ All trench backfill, including utility and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade.
- ☐ Adequate drainage should be provided to prevent surface water from ponding and saturating the subgrade soil.
- ☐ A periodic maintenance program should be incorporated.
- ☐ All concrete curbs separating pavement and landscaped areas should extend to the subgrade.

7.6.4 Construction Considerations

In the event unstable (pumping) subgrades are encountered within planned pavement areas, we recommend a heavy, rubber-tired vehicle (typically a loaded water truck) be used to test the load/deflection characteristics of the finished subgrade materials. It is recommended this vehicle have a minimum rear axle load (at the time of testing) of 16,000 pounds with tires inflated to at least 65 psi pressure. If the tested surface shows a visible deflection extending more than 6 inches from the wheel track at the time of loading, or a visible crack remains after loading, corrective measures should be implemented. Such measures could include diskings to aerate, chemical treatment, replacement with drier material, or other methods. It is recommended **TECHNICON** be retained to assist in developing which method (or methods) would be applicable for this project.

7.7 CORROSION POTENTIAL

One (1) soil sample from the near surface of the site was tested for pH, minimum electrical resistivity, and soluble sulfate and chloride.

The pH of the soil tested was 6.12 and the minimum electrical resistivity was 4,761 ohm-cm. These values are generally representative of an environment that could be moderately corrosive to buried unprotected metals. Utilizing methods provided in Caltrans California Test 643, "Method for Estimating the Service Life of Steel Culverts", an 18-gauge steel zinc-coated culvert is estimated to have a maintenance-free service life (years to perforation) exceeding 18 years. Therefore, if project improvements will involve metal that comes into contact with the on-site soil, the design should consider this potential soil corrosiveness described.

Test results suggest that low levels of soluble sulfates (25 ppm) and low levels of soluble chlorides (60 ppm) are present in on-site soils. Normal cement (Type II) and normal reinforcement cover should be adequate for foundation concrete that comes in contact with the foundation soils.

Corrosion is dependent upon a complex variety of conditions, which are beyond the geotechnical practice. Consequently, a qualified corrosion engineer should be consulted if the owner desires more specific recommendations.

7.8 SITE DRAINAGE

Providing and maintaining adequate site drainage to prevent entrapment and ponding of surface water and excessive moisture migration into the subgrade soil is very important. Poor perimeter or surface drainage could cause reduced subgrade support. The site should incorporate the basis for good drainage. This includes:

- ☐ Sufficient pad height to allow for proper drainage; and
- ☐ Defined drainage gradients away from the structure to points of conveyance, such as drainage swales and/or area drains and discharge pipe.

The maintenance personnel must maintain the established drainage by not blocking or obstructing gradients away from structures without providing some alternative drainage means (e.g., area drains and subsurface pipes). If planter or landscape areas are established near the

structures, it is important to prevent surface run-off from entering the planter and care must be taken not to over irrigate and to maintain a leak-free sprinkler piping system. Consideration should be given to use of low volume emitter irrigation systems for planters. Well-maintained low-volume emitter irrigation (drip system) is best suited for planters adjacent to structures. Watering practices must strive to use only sufficient water to sustain and promote plant growth.

8 ADDITIONAL SERVICES

8.1 DESIGN REVIEW AND CONSULTATION

It is recommended that **TECHNICON** be retained to review those portions of the contract drawings and specifications that pertain to earthwork, foundations, and pavements prior to finalization to determine whether they are consistent with our recommendations.

8.2 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that a representative of **TECHNICON** observe the excavation, earthwork, pavements, and foundation, phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design. **TECHNICON** can conduct the necessary field testing and provide results on a timely basis so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of our observations, field testing, and conclusions regarding the conformance of the completed work to the intent of the plans and specifications will be provided. This additional service is not part of this current contractual agreement. **TECHNICON** firm will not be responsible for establishing or confirming building or foundations depths or locations unless retained to do so.

9 LIMITATIONS

The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of our field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations. The nature and extent of the variations between borings may not become evident until construction. If variations or undesirable conditions are encountered during construction, our firm should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. The unexpected conditions frequently require additional expenditures for proper construction of the project. **TECHNICON Engineering Services, Inc.** will not assume any responsibility for errors or omissions if the final extent and depth of earthwork is not determined by our firm at the time of construction due to said variations or undesirable conditions encountered.

If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work at the site, or if conditions have changed due to natural causes, or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing. Such conditions may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. This report does not relieve the contractors of responsibility for temporary excavation construction, bracing and shoring in accordance with CAL OSHA requirements.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. This report should not be construed as an environmental audit or study.

This report has been prepared for the sole use by State Center Community College District and their designated consultants for the proposed Sports Complex to be located at 10309 N. Willow Avenue in Clovis, California. Recommendations presented in this report should not be extrapolated to other areas or used for other projects without prior review. This report has been prepared with the intent that the firm of **TECHNICON** will be performing the construction testing and observation for the complete project. If, however, another firm or individual(s) should be retained or employed to use this geotechnical investigation report for the purpose of construction testing and observation, notice is hereby given that **TECHNICON** will not assume any responsibility for errors or omissions, if any, which may occur and which could have been avoided, corrected, or mitigated if **TECHNICON**, had performed the work. This notice also applies to the misuse or misinterpretation of the conclusions and recommendations outlined in this report. Furthermore, the other firm or individual(s) performing construction testing and observation should accept transfer of responsibility of the work, as required by the California Building Code, in writing to the project owner and **TECHNICON**. The firm accepting transfer of responsibility should perform additional investigation(s) as may be necessary to develop their own conclusions, evaluations, and recommendations for design and construction.

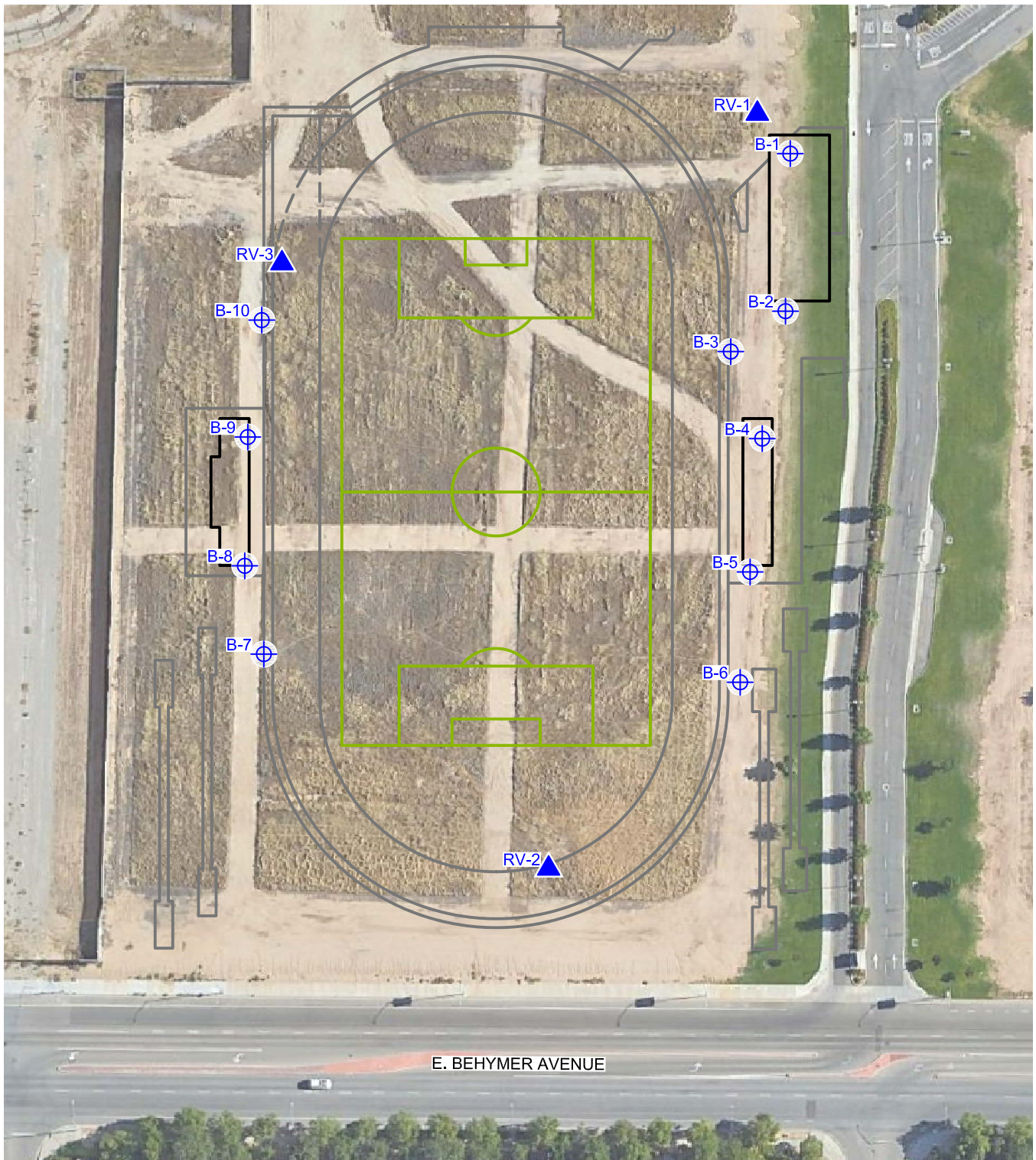
10 REFERENCES

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FIGURES

1 through 8



▲ =R-VALUE LOCATIONS
 ⊕ =SOIL BORING LOCATIONS

NORTH
 SCALE: 1"=100'
 0' 50' 100'



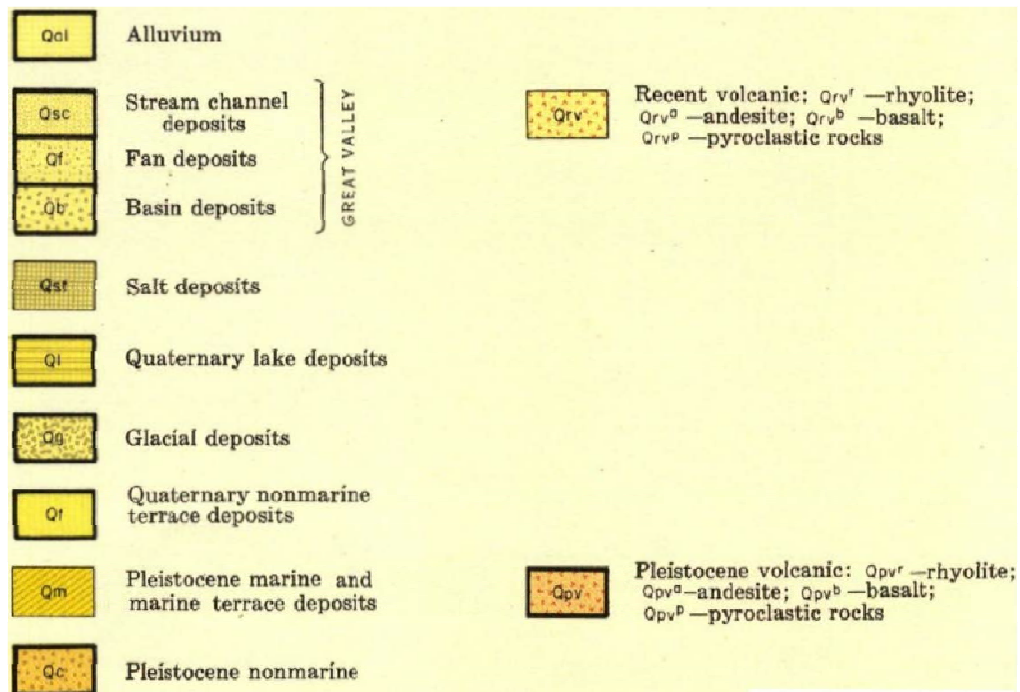
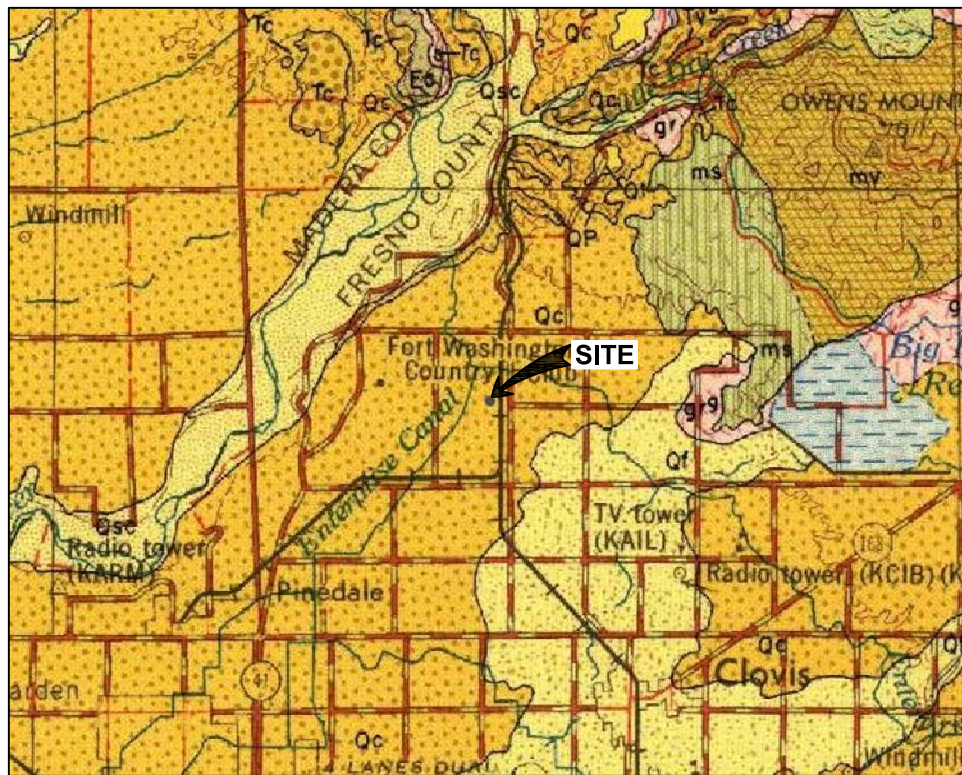
PROJECT:
240005

SOURCE:
GOOGLE EARTH

SITE MAP
 PROPOSED SPORTS COMPLEX
 CLOVIS COMMUNITY COLLEGE
 10309 N. WILLOW AVENUE
 CLOVIS, CALIFORNIA

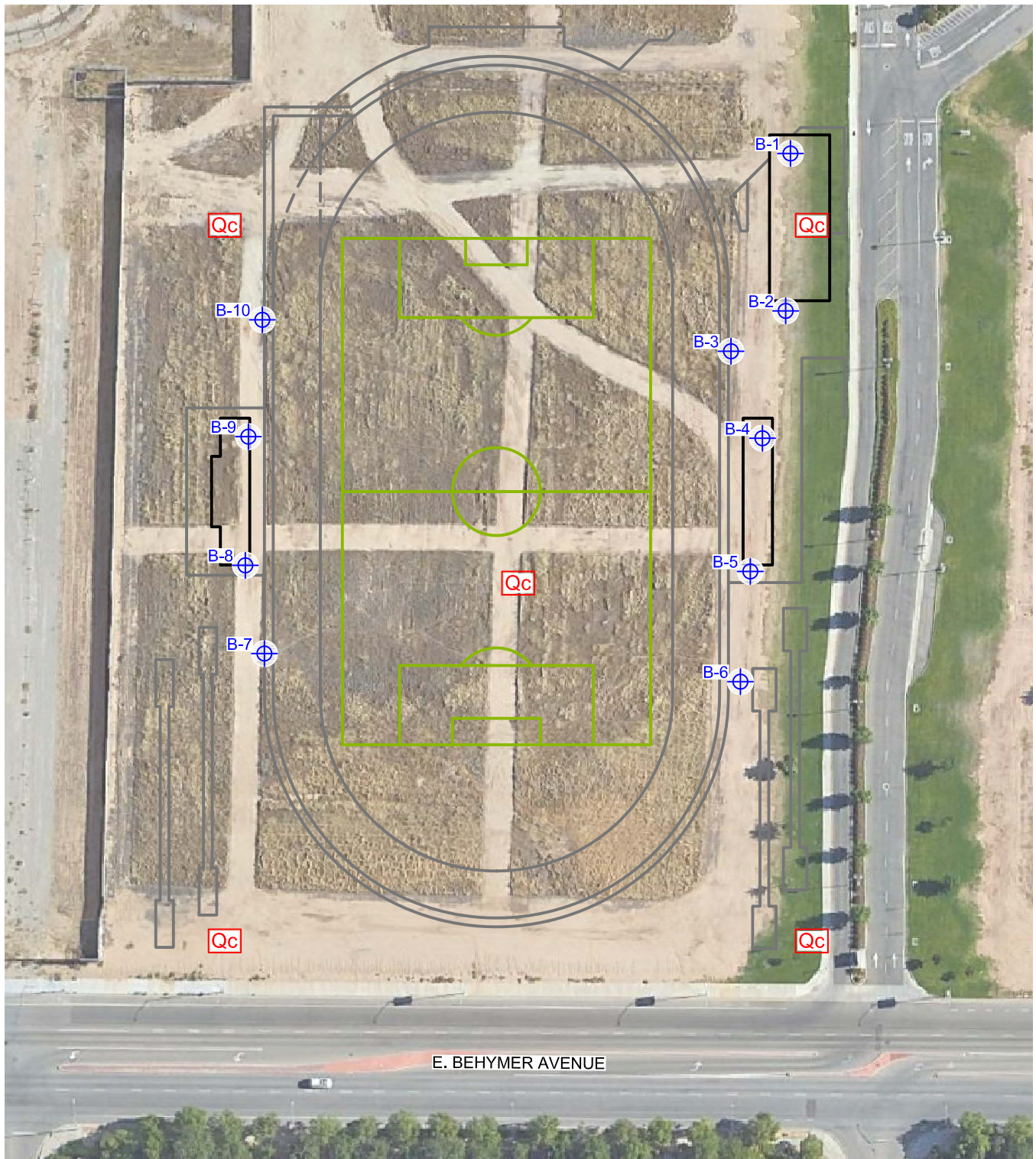
FIGURE

2



GEOLOGIC MAP OF CALIFORNIA : FRESNO SHEET, SCALE 1:250,000 - 1965





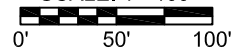
Qc =PLEISTOCENE NONMARINE

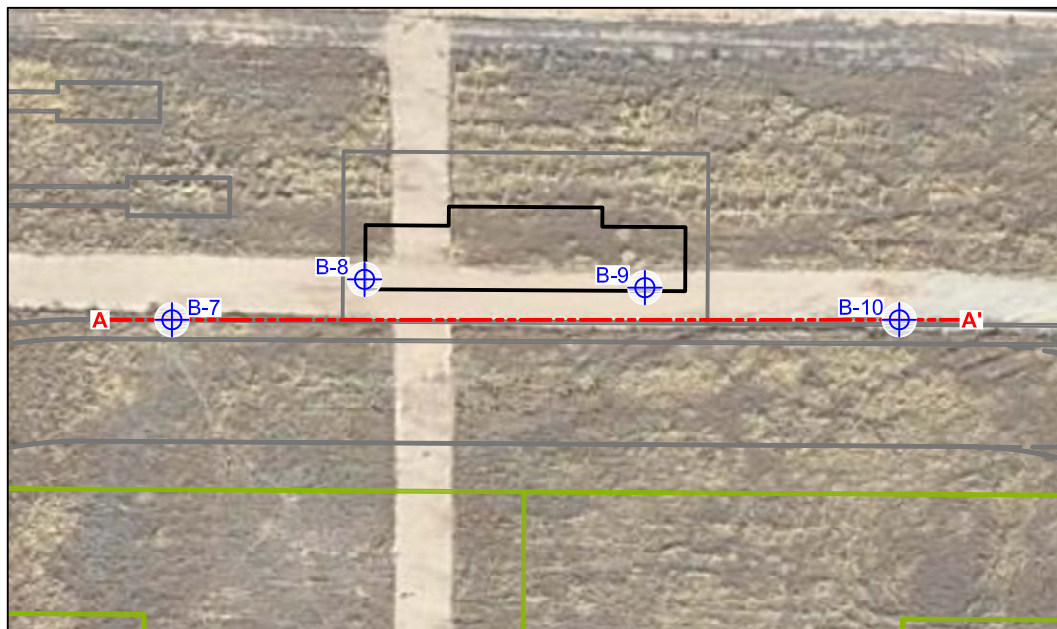
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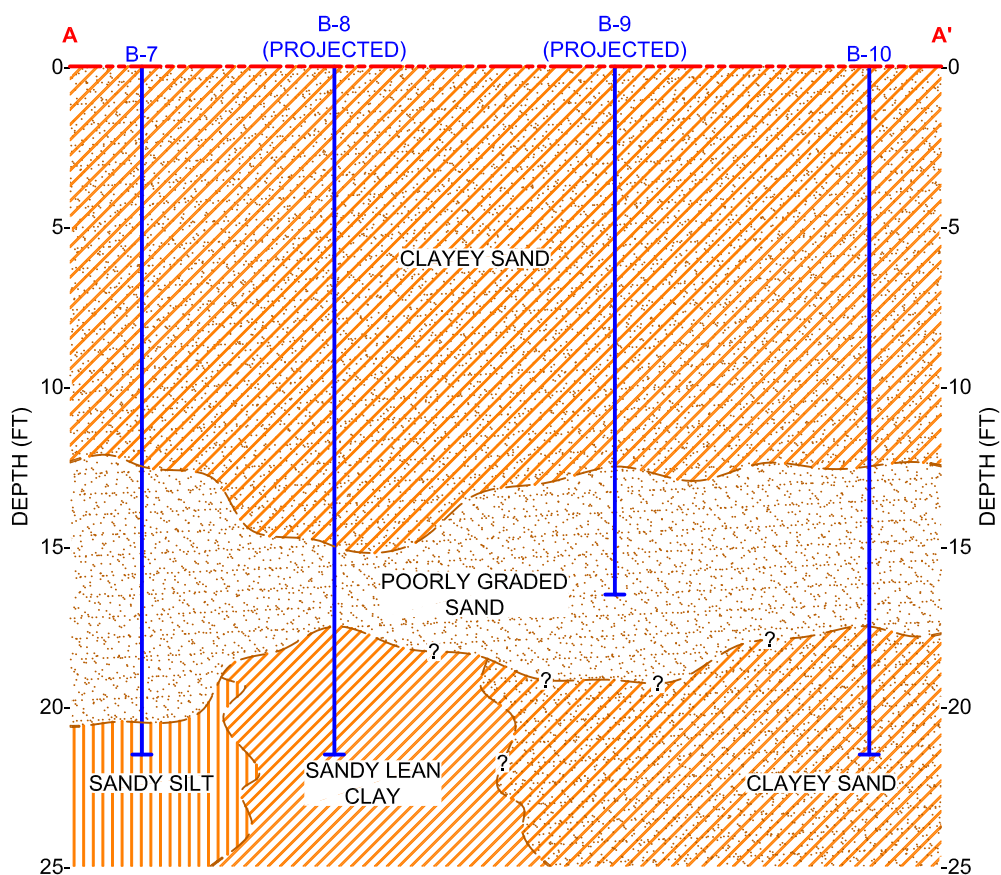
NORTH

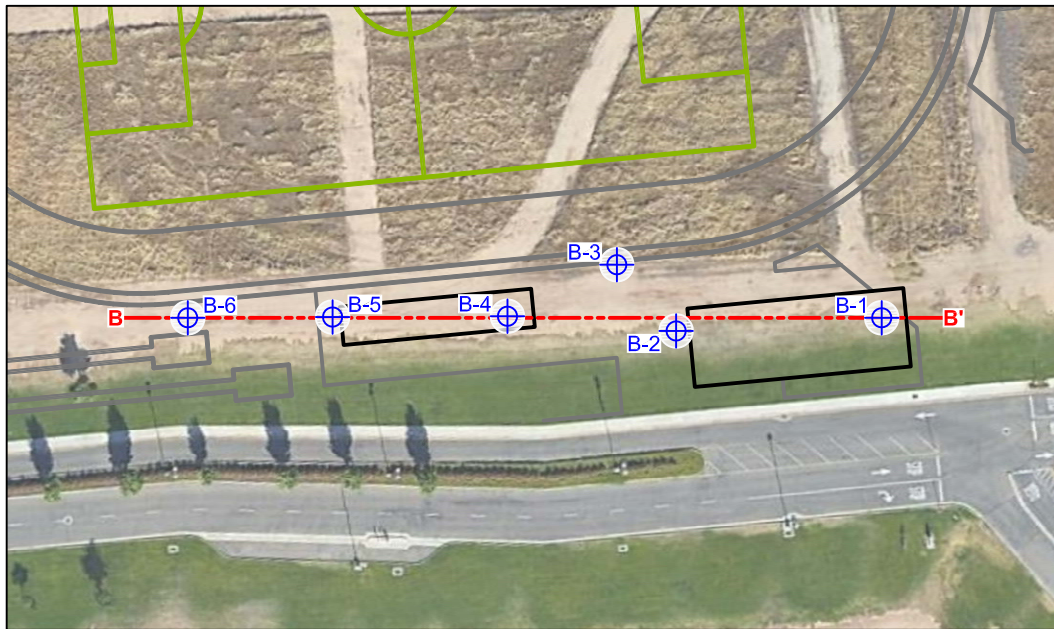
SCALE: 1"=100'



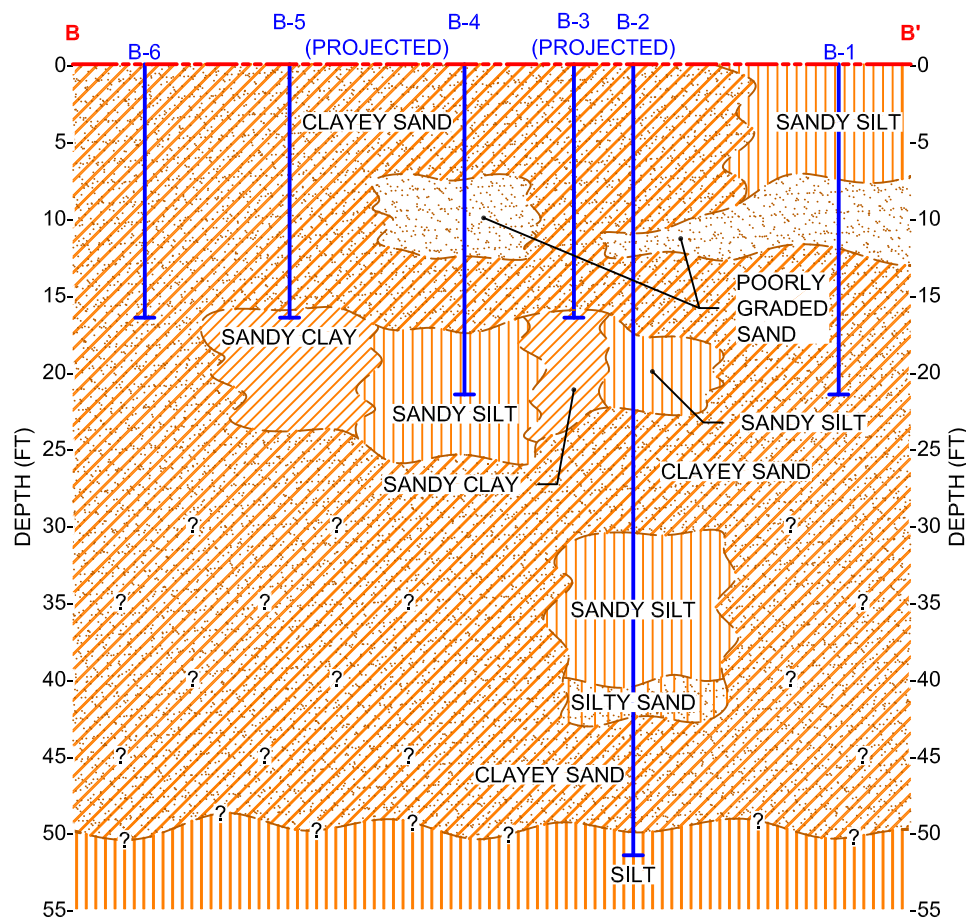


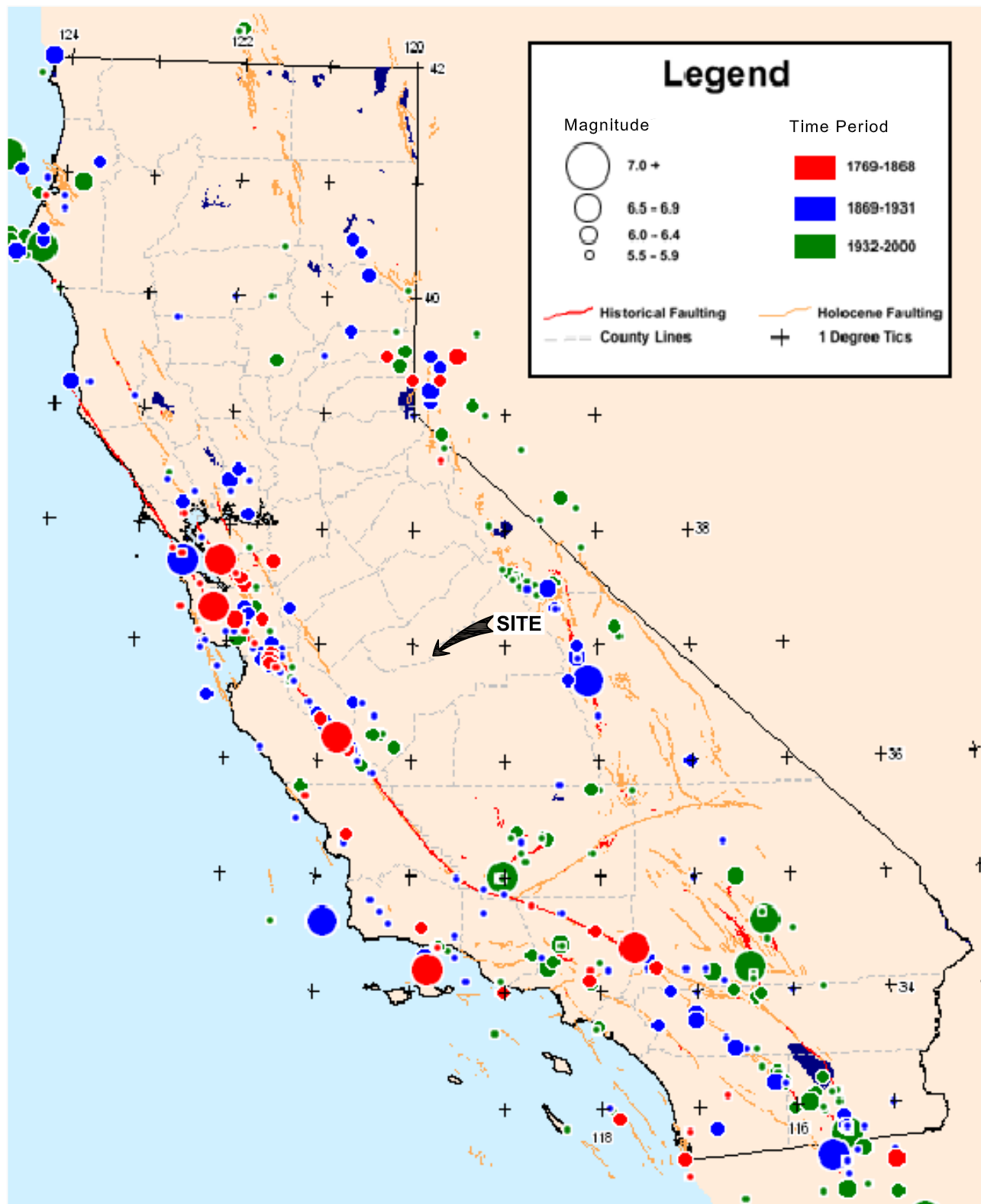
NORTH
SCALE: 1"=60'

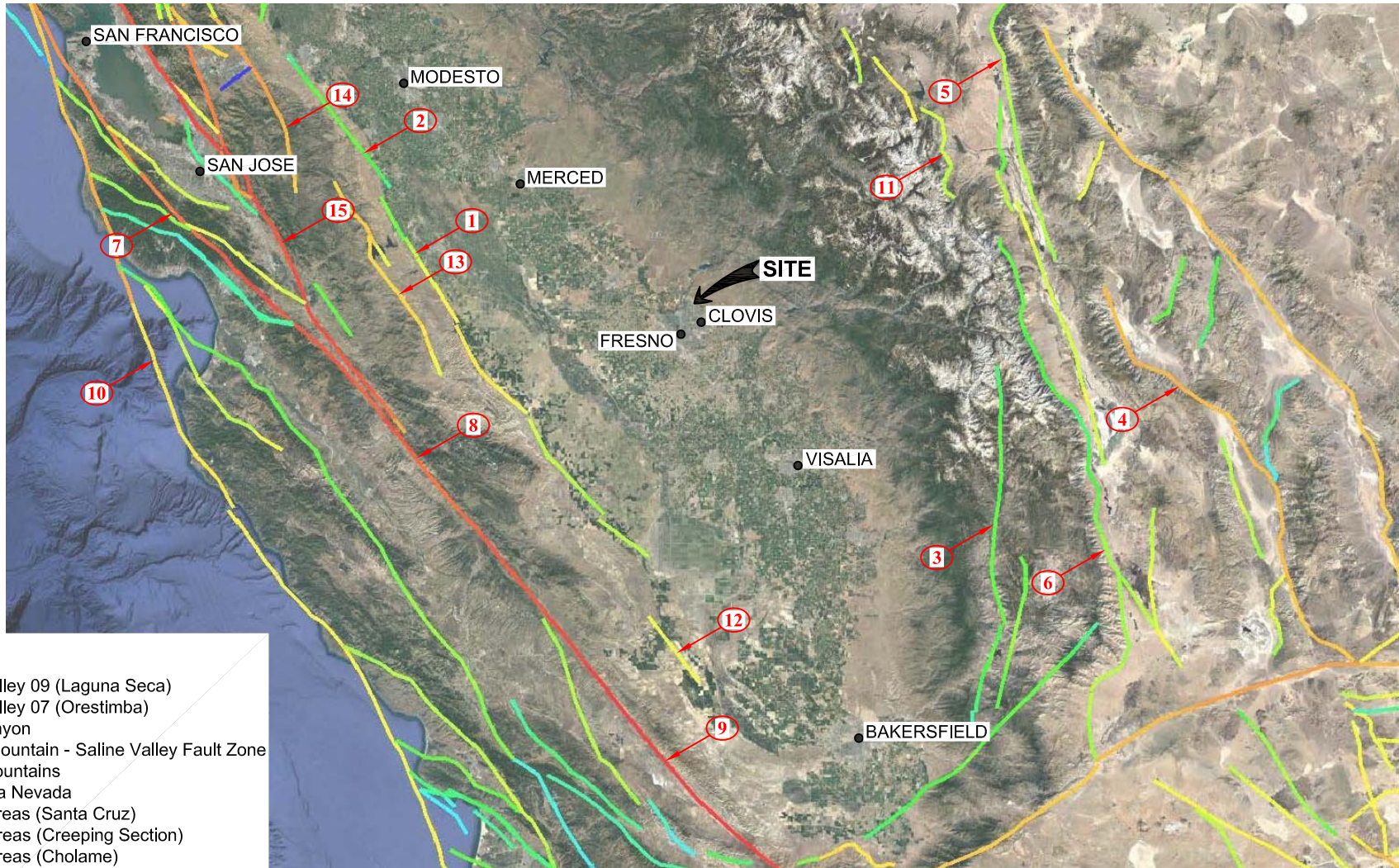





 NORTH
 SCALE: 1"=100'







FAULTS

1. Great Valley 09 (Laguna Seca)
2. Great Valley 07 (Orestimba)
3. Kern Canyon
4. Hunter Mountain - Saline Valley Fault Zone
5. White Mountains
6. SO Sierra Nevada
7. San Andreas (Santa Cruz)
8. San Andreas (Creeping Section)
9. San Andreas (Cholame)
10. San Gregorio
11. Round Valley
12. Lost Hills
13. Ortigalita
14. Greenville
15. Calaveras



PROJECT:
240005

DATE:
3/18/24

SOURCE:
WGCEP

APPROVED BY:
SA

REGIONAL FAULT ACTIVITY MAP
PROPOSED SPORTS COMPLEX
CLOVIS COMMUNITY COLLEGE
10309 N. WILLOW AVENUE
CLOVIS, CALIFORNIA

FIGURE

8

NTS

BORING LOGS AND LOG KEY

APPENDIX A



TECHNICON Engineering Services, Inc.
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Fresno, CA 93722

KEY TO SYMBOLS

PROJECT NAME Clovis Community College Sports Complex

DATE OF EXPLORATION 1/12/2024

PROJECT LOCATION Clovis, California

PROJECT NUMBER 240005

LITHOLOGIC SYMBOLS (Unified Soil Classification System)



FILL



SW WELL GRADED SAND



SP POORLY GRADED SAND



SM SILTY SAND



SC CLAYEY SAND



PT PEAT



OL LOW PLASTICITY ORGANIC SILT



OH HIGH PLASTICITY ORGANIC SILT



ML LOW PLASTICITY SILT



MH HIGH PLASTICITY SILT



GW WELL GRADED GRAVEL



GP POORLY GRADED GRAVEL



GM SILTY GRAVEL



GC CLAYEY GRAVEL



CL LOW PLASTICITY CLAY



CH HIGH PLASTICITY CLAY

SAMPLER SYMBOLS



STANDARD PENETRATION TEST



CALIFORNIA SAMPLER



MODIFIED CALIFORNIA SAMPLER



SHELBY TUBE SAMPLER



ROCK CORE BARREL



BULK SAMPLE



Water Level at Time of Drilling



Water Level at End of Drilling



Water Level After 24 Hours



Assumed stratum line



Observed stratum line

Note 1: The degree of saturation shown on the boring logs is based on an assumed specific gravity of 2.65. The actual degree of saturation may vary.

Note 2: The stratum lines shown on the logs represent the approximate boundary between soil types; the actual in-situ transition may be gradual.

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
S - DEGREE OF SATURATION (%)
NP - NON PLASTIC
200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)
ND - NOT DETECTED

TV -TORVANE
PID -PHOTOIONIZATION DETECTOR
UC -UNCONFINED COMPRESSION
ppm -PARTS PER MILLION
TPH-d -TOTAL PETROLEUM HYDROCARBON AS DIESEL
TPH-mo -TOTAL PETROLEUM HYDROCARBON AS MOTOR OIL



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Fresno, CA 93722

BORING B-01

PAGE 1 OF 1

PROJECT NAME Clovis Community College Sports Complex **PROJECT NUMBER** 240005
PROJECT LOCATION Clovis, California **SURFACE DESCRIPTION** Bare Soil
DATE STARTED 1/12/24 **COMPLETED** 1/12/24 **GROUND ELEVATION** _____
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc. **GROUND WATER LEVEL** No groundwater encountered.
DRILL RIG TYPE SIMCO 2800 **BORING DEPTH** 21.5 ft
DRILLING METHOD 4-inch Solid Flight Auger **LOGGED BY** C. Odneal **CHECKED BY** A. AhTye

BOREHOLE - TECHNICON GDT - 3/14/24 09:50 - Z:\TESDATA\PROJECTS\PROJECTS\240000-240099\240005 CLOVIS COMM. COLLEGE SPORTS COMPLEX\REPORTS\240005 - GINT.GPJ

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0								
2	GB	20-29-36 (65)		Sandy SILT (ML) - hard, brown, moist	111.1	12.0	S = 65 %	
5	CAL	22-18-15 (33)		Very stiff	122.2	6.6	S = 50 %	
10	SPT	4-6-5 (11)		Poorly Graded SAND (SP) - medium dense, light brown, moist, fine to coarse grained, trace clay				
15	CAL	16-21-22 (43)		Clayey SAND (SC) - dense, light brown, moist, fine to medium grained	106.3	20.8	S = 100 %	
20	SPT	9-12-16 (28)		Medium dense				

NOTES:

- Bottom of boring at 21.5 feet.
- No groundwater encountered.
- Boring backfilled with auger cuttings.



TECHNICON Engineering Services, Inc.
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Fresno, CA 93722

BORING B-02

PAGE 1 OF 2

PROJECT NAME Clovis Community College Sports Complex

PROJECT NUMBER 240005

PROJECT LOCATION Clovis, California

SURFACE DESCRIPTION Grass

DATE STARTED 1/12/24

COMPLETED 1/12/24

GROUND ELEVATION

DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.

GROUND WATER LEVEL No groundwater encountered.

DRILL RIG TYPE SIMCO 2800

BORING DEPTH 51.5 ft

DRILLING METHOD 4-inch Solid Flight Auger

LOGGED BY C. Odneal

CHECKED BY A. AhTye

BOREHOLE - TECHNICON GDT - 3/14/24 09:50 - Z:\TESDATA\PROJECTS\PROJECTS\240000-240099\240005 CLOVIS COMM. COLLEGE SPORTS COMPLEX\REPORTS\240005 - GINT.GPJ

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0								
	CAL	19-20-18 (38)		Clayey SAND (SC) - medium dense, brown, moist, fine to medium grained	118.9	3.1	S = 21 %	
	GB							
5								
	CAL	14-20-20 (40)			125.4	8.3	S = 69 %	
10								
	SPT	7-7-7 (14)		Poorly Graded SAND (SP) - medium dense, light brown, moist, fine to coarse grained				
				Clayey SAND (SC) - dense, brown, moist, fine to medium grained				
15								
	CAL	20-31-25 (56)			124.1	9.4	S = 75 %	
				Sandy SILT (ML) - stiff, brown, moist				
20								
	SPT	7-7-8 (15)						
				Clayey SAND (SC) - very dense, brown, moist, fine to coarse grained, trace fine gravel				
25								
	CAL	18-32-50 (82)			124.5	10.8	S = 87 %	
30								
	SPT	6-8-9 (17)		Sandy SILT (ML) - very stiff, light brown, moist				
35								

(Continued Next Page)



TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, CA 93722

BORING B-02

PAGE 2 OF 2

PROJECT NAME Clovis Community College Sports Complex **PROJECT NUMBER** 240005
PROJECT LOCATION Clovis, California **SURFACE DESCRIPTION** Grass
DATE STARTED 1/12/24 **COMPLETED** 1/12/24 **GROUND ELEVATION** _____
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc. **GROUND WATER LEVEL** No groundwater encountered.
DRILL RIG TYPE SIMCO 2800 **BORING DEPTH** 51.5 ft
DRILLING METHOD 4-inch Solid Flight Auger **LOGGED BY** C. Odneal **CHECKED BY** A. AhTye

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
35								
	CAL	6-28-30 (58)		Sandy SILT (ML) - very stiff, light brown, moist (continued) Hard, grayish brown	96.8	4.3	S = 16 %	
40								
	SPT	15-25-27 (52)		Silty SAND (SM) - dense, light brown, moist, fine to medium grained				
45				Clayey SAND (SC) - dense, light brown, moist, fine to coarse grained, trace fine gravel				
	CAL	15-27-26 (53)			81.9	4.4	S = 11 %	
50								
	SPT	19-28-26 (54)		SILT (ML) - hard, grayish white, moist				

NOTES:

1. Bottom of boring at 51.5 feet.
2. No groundwater encountered.
3. Boring backfilled with auger cuttings.



TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, CA 93722

BORING B-03

PAGE 1 OF 1

PROJECT NAME Clovis Community College Sports Complex **PROJECT NUMBER** 240005
PROJECT LOCATION Clovis, California **SURFACE DESCRIPTION** Bare Soil
DATE STARTED 1/12/24 **COMPLETED** 1/12/24 **GROUND ELEVATION**
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc. **GROUND WATER LEVEL** No groundwater encountered.
DRILL RIG TYPE SIMCO 2800 **BORING DEPTH** 16.5 ft
DRILLING METHOD 4-inch Solid Flight Auger **LOGGED BY** C. Odneal **CHECKED BY** A. AhTye

BOREHOLE - TECHNICON GDT - 3/14/24 09:50 - Z:\TESDATA\PROJECTS\PROJECTS\240000-240099\240005 CLOVIS COMM. COLLEGE SPORTS COMPLEX\REPORTS\240005 - GINT.GPJ

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0								
	CAL	5-18-19 (37)		Clayey SAND (SC) - medium dense, brown, moist, fine to medium grained	125.5	6.4	S = 53 %	
5	SPT	9-10-10 (20)		Increased sand				
10	CAL	5-7-8 (15)			109.6	6.5	S = 34 %	
15	SPT	6-12-10 (22)		Sandy LEAN CLAY (CL) - stiff, brown, moist				

NOTES:

1. Bottom of boring at 16.5 feet.
2. No groundwater encountered.
3. Boring backfilled with auger cuttings.



TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, CA 93722

BORING B-04

PAGE 1 OF 1

PROJECT NAME Clovis Community College Sports Complex **PROJECT NUMBER** 240005
PROJECT LOCATION Clovis, California **SURFACE DESCRIPTION** Bare Soil
DATE STARTED 1/12/24 **COMPLETED** 1/12/24 **GROUND ELEVATION**
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc. **GROUND WATER LEVEL** No groundwater encountered.
DRILL RIG TYPE SIMCO 2800 **BORING DEPTH** 21.5 ft
DRILLING METHOD 4-inch Solid Flight Auger **LOGGED BY** C. Odneal **CHECKED BY** A. AhTye

BOREHOLE - TECHNICON GDT - 3/14/24 09:50 - Z:\TESDATA\PROJECTS\PROJECTS\240000-240099\240005 CLOVIS COMM. COLLEGE SPORTS COMPLEX\REPORTS\240005 - GINT.GPJ

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0								
	CAL	9-19-20 (39)		Clayey SAND (SC) - medium dense, brown, moist, fine to coarse grained	124.5	5.9	S = 48 %	
	GB							
5	CAL	10-10-9 (19)		Trace fine gravel	123.6	8.9	S = 70 %	
10	SPT	4-5-5 (10)		Poorly Graded SAND (SP) - medium dense, light brown, moist, fine to coarse grained, trace clay				
15	CAL	12-23-19 (42)		Clayey SAND (SC) - medium dense, brown, moist, fine to medium grained	120.6	11.4	S = 81 %	
20	SPT	5-10-11 (21)		Sandy SILT (ML) - very stiff, grayish brown, moist				

NOTES:

1. Bottom of boring at 21.5 feet.
2. No groundwater encountered.
3. Boring backfilled with auger cuttings.



TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, CA 93722

BORING B-05

PAGE 1 OF 1

PROJECT NAME Clovis Community College Sports Complex **PROJECT NUMBER** 240005
PROJECT LOCATION Clovis, California **SURFACE DESCRIPTION** Bare Soil
DATE STARTED 1/15/24 **COMPLETED** 1/15/24 **GROUND ELEVATION**
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc. **GROUND WATER LEVEL** No groundwater encountered.
DRILL RIG TYPE SIMCO 2800 **BORING DEPTH** 16.5 ft
DRILLING METHOD 4-inch Solid Flight Auger **LOGGED BY** C. Odneal **CHECKED BY** A. AhTye

BOREHOLE - TECHNICON GDT - 3/14/24 09:50 - Z:\TESDATA\PROJECTS\PROJECTS\240000-240099\240005 CLOVIS COMM. COLLEGE SPORTS COMPLEX\REPORTS\240005 - GINT.GPJ

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0								
	CAL	13-21-36 (57)		Clayey SAND (SC) - dense, brown, moist, fine to coarse grained	128.5	7.2	S = 66 %	
5	SPT	12-13-15 (28)		Medium dense				
10	CAL	18-25-23 (48)		Dense, increased sand	98.8	8.2	S = 32 %	
15	SPT	10-10-9 (19)		Sandy LEAN CLAY (CL) - very stiff, brown, moist				

NOTES:

1. Bottom of boring at 16.5 feet.
2. No groundwater encountered.
3. Boring backfilled with auger cuttings.



TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, CA 93722

BORING B-06

PAGE 1 OF 1

PROJECT NAME Clovis Community College Sports Complex **PROJECT NUMBER** 240005
PROJECT LOCATION Clovis, California **SURFACE DESCRIPTION** Bare Soil
DATE STARTED 1/15/24 **COMPLETED** 1/15/24 **GROUND ELEVATION**
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc. **GROUND WATER LEVEL** No groundwater encountered.
DRILL RIG TYPE SIMCO 2800 **BORING DEPTH** 16.5 ft
DRILLING METHOD 4-inch Solid Flight Auger **LOGGED BY** C. Odneal **CHECKED BY** A. AhTye

BOREHOLE - TECHNICON GDT - 3/14/24 09:50 - Z:\TESDATA\PROJECTS\PROJECTS\240000-240099\240005 CLOVIS COMM. COLLEGE SPORTS COMPLEX\REPORTS\240005 - GINT.GPJ

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0								
				Clayey SAND (SC) - very dense, brown, moist, fine to coarse grained				
	GB CAL	15-33-50 (83)			131.0	7.3	S = 74 %	
5								
	CAL	6-22-25 (47)		Dense	131.3	5.9	S = 60 %	
10								
	SPT	10-12-12 (24)		Medium dense, increased sand				
15								
	CAL	17-22-33 (55)		Dense, fine grained, decreased sand	116.0	8.4	S = 52 %	

NOTES:

- Bottom of boring at 16.5 feet.
- No groundwater encountered.
- Boring backfilled with auger cuttings.



TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, CA 93722

BORING B-07

PAGE 1 OF 1

PROJECT NAME Clovis Community College Sports Complex **PROJECT NUMBER** 240005
PROJECT LOCATION Clovis, California **SURFACE DESCRIPTION** Bare Soil
DATE STARTED 1/15/24 **COMPLETED** 1/15/24 **GROUND ELEVATION**
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc. **GROUND WATER LEVEL** No groundwater encountered.
DRILL RIG TYPE SIMCO 2800 **BORING DEPTH** 21.5 ft
DRILLING METHOD 4-inch Solid Flight Auger **LOGGED BY** C. Odneal **CHECKED BY** A. AhTye

BOREHOLE - TECHNICON GDT - 3/14/24 09:50 - Z:\TESDATA\PROJECTS\PROJECTS\240000-240099\240005 CLOVIS COMM. COLLEGE SPORTS COMPLEX\REPORTS\240005 - GINT.GPJ

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0								
	CAL	7-6-17 (23)		Clayey SAND (SC) - medium dense, brown, moist, fine to medium grained	119.4	6.4	S = 44 %	
	GB							
5	CAL	6-19-22 (41)		Dense	130.4	8.4	S = 83 %	
10	SPT	5-7-6 (13)		Medium dense, increased sand				
15	CAL	8-17-16 (33)		Poorly Graded SAND (SP) - medium dense, light brown, moist, fine to coarse grained, trace clay	104.3	3.2	S = 14 %	
20	SPT	7-11-13 (24)		Sandy SILT (ML) - very stiff, light brown, moist				

NOTES:

- Bottom of boring at 21.5 feet.
- No groundwater encountered.
- Boring backfilled with auger cuttings.



TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, CA 93722

BORING B-08

PAGE 1 OF 1

PROJECT NAME Clovis Community College Sports Complex **PROJECT NUMBER** 240005
PROJECT LOCATION Clovis, California **SURFACE DESCRIPTION** Bare Soil
DATE STARTED 1/15/24 **COMPLETED** 1/15/24 **GROUND ELEVATION**
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc. **GROUND WATER LEVEL** No groundwater encountered.
DRILL RIG TYPE SIMCO 2800 **BORING DEPTH** 21.5 ft
DRILLING METHOD 4-inch Solid Flight Auger **LOGGED BY** C. Odneal **CHECKED BY** A. AhTye

BOREHOLE - TECHNICON GDT - 3/14/24 09:50 - Z:\TESDATA\PROJECTS\PROJECTS\240000-240099\240005 CLOVIS COMM. COLLEGE SPORTS COMPLEX\REPORTS\240005 - GINT.GPJ

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0								
	CAL	7-13-11 (24)		Clayey SAND (SC) - medium dense, brown, moist, fine to medium grained	117.0	4.0	S = 26 %	
5								
	SPT	14-28-33 (61)		Very dense, fine to coarse grained				
10								
	CAL	8-20-19 (39)		Medium dense, increased sand	114.2	5.0	S = 30 %	
15								
	SPT	7-8-10 (18)		Poorly Graded SAND (SP) - medium dense, light brown, moist, fine to coarse grained, trace clay				
				Sandy LEAN CLAY (CL) - hard, brown, moist				
20								
	CAL	15-20-29 (49)			99.8	21.8	S = 88 %	

NOTES:

1. Bottom of boring at 21.5 feet.
2. No groundwater encountered.
3. Boring backfilled with auger cuttings.



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Fresno, CA 93722

BORING B-09

PAGE 1 OF 1

PROJECT NAME Clovis Community College Sports Complex **PROJECT NUMBER** 240005
PROJECT LOCATION Clovis, California **SURFACE DESCRIPTION** Bare Soil
DATE STARTED 1/15/24 **COMPLETED** 1/15/24 **GROUND ELEVATION**
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc. **GROUND WATER LEVEL** No groundwater encountered.
DRILL RIG TYPE SIMCO 2800 **BORING DEPTH** 16.5 ft
DRILLING METHOD 4-inch Solid Flight Auger **LOGGED BY** C. Odneal **CHECKED BY** A. AhTye

BOREHOLE - TECHNICON GDT - 3/14/24 09:50 - Z:\TESTDATA\PROJECTS\PROJECTS\240000-240099\240005 CLOVIS COMM. COLLEGE SPORTS COMPLEX\REPORTS\240005 - GINT.GPJ

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0								
	CAL	15-27-27 (54)		Clayey SAND (SC) - dense, brown, moist, fine to coarse grained	124.2	6.8	S = 55 %	
	GB							
5	CAL	17-25-21 (46)		Increased clay	132.0	8.2	S = 86 %	
10	SPT	4-5-7 (12)		Medium dense, increased sand				
				Poorly Graded SAND (SP) - medium dense, light brown, fine to coarse grained, trace clay				
15	CAL	8-15-13 (28)			106.0	4.0	S = 19 %	

NOTES:

- Bottom of boring at 16.5 feet.
- No groundwater encountered.
- Boring backfilled with auger cuttings.



TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, CA 93722

BORING B-10

PAGE 1 OF 1

PROJECT NAME Clovis Community College Sports Complex **PROJECT NUMBER** 240005
PROJECT LOCATION Clovis, California **SURFACE DESCRIPTION** Bare Soil
DATE STARTED 1/15/24 **COMPLETED** 1/15/24 **GROUND ELEVATION**
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc. **GROUND WATER LEVEL** No groundwater encountered.
DRILL RIG TYPE SIMCO 2800 **BORING DEPTH** 21.5 ft
DRILLING METHOD 4-inch Solid Flight Auger **LOGGED BY** C. Odneal **CHECKED BY** A. AhTye

BOREHOLE - TECHNICON GDT - 3/14/24 09:50 - Z:\TESDATA\PROJECTS\PROJECTS\240000-240099\240005 CLOVIS COMM. COLLEGE SPORTS COMPLEX\REPORTS\240005 - GINT.GPJ

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0								
	CAL	8-33-47 (80)		Clayey SAND (SC) - very dense, brown, moist, fine to medium grained, trace clay	127.0	8.5	S = 74 %	
5	SPT	14-20-22 (42)		Dense				
10	CAL	10-14-14 (28)			111.3	3.2	S = 17 %	
15	SPT	5-6-7 (13)		Poorly Graded SAND (SP) - medium dense, light brown, moist, fine to coarse grained, trace clay				
20	CAL	8-13-15 (28)		Clayey SAND (SC) - medium dense, brown, moist, fine to medium grained	116.1	12.1	S = 76 %	

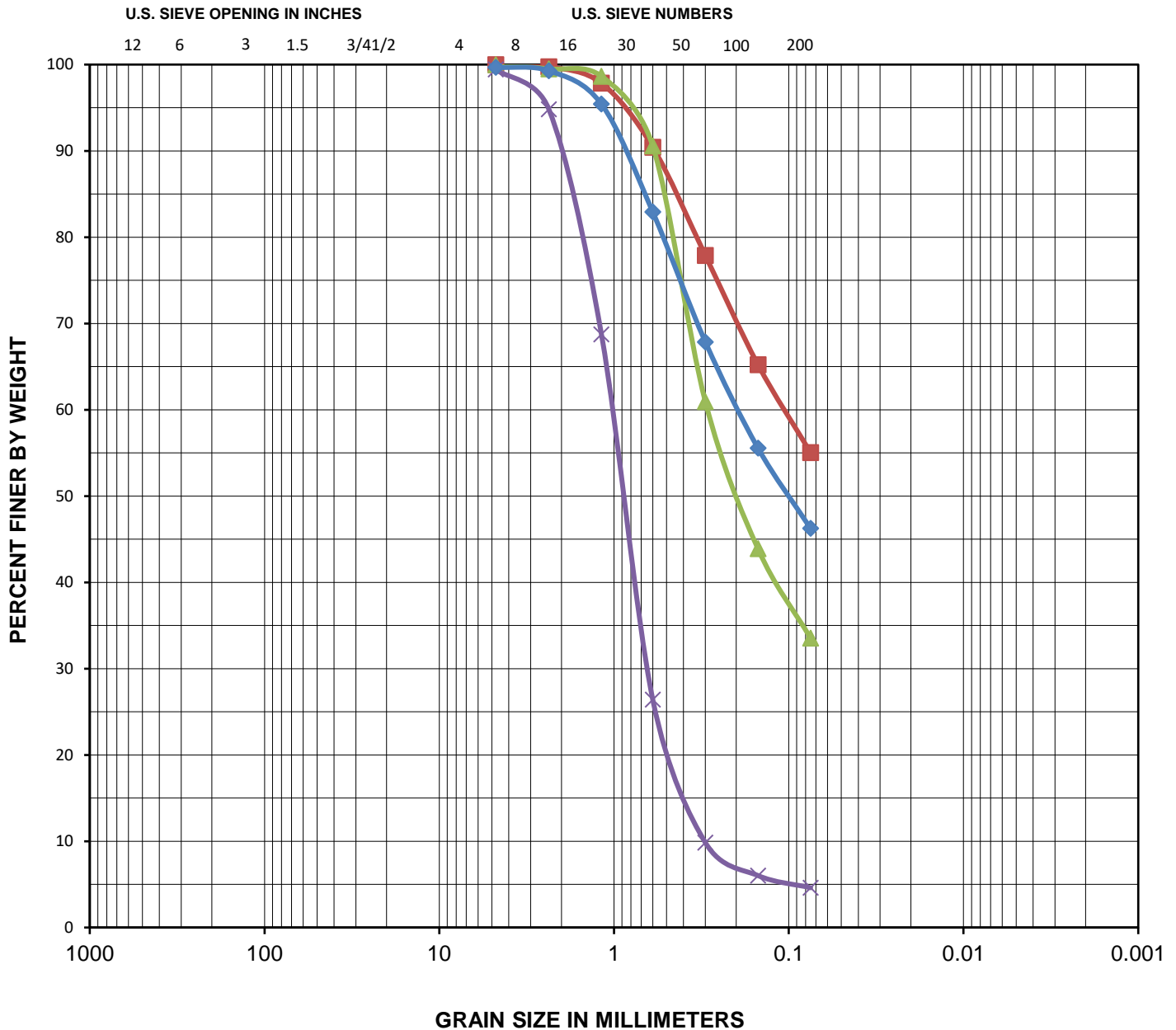
NOTES:

1. Bottom of boring at 21.5 feet.
2. No groundwater encountered.
3. Boring backfilled with auger cuttings.

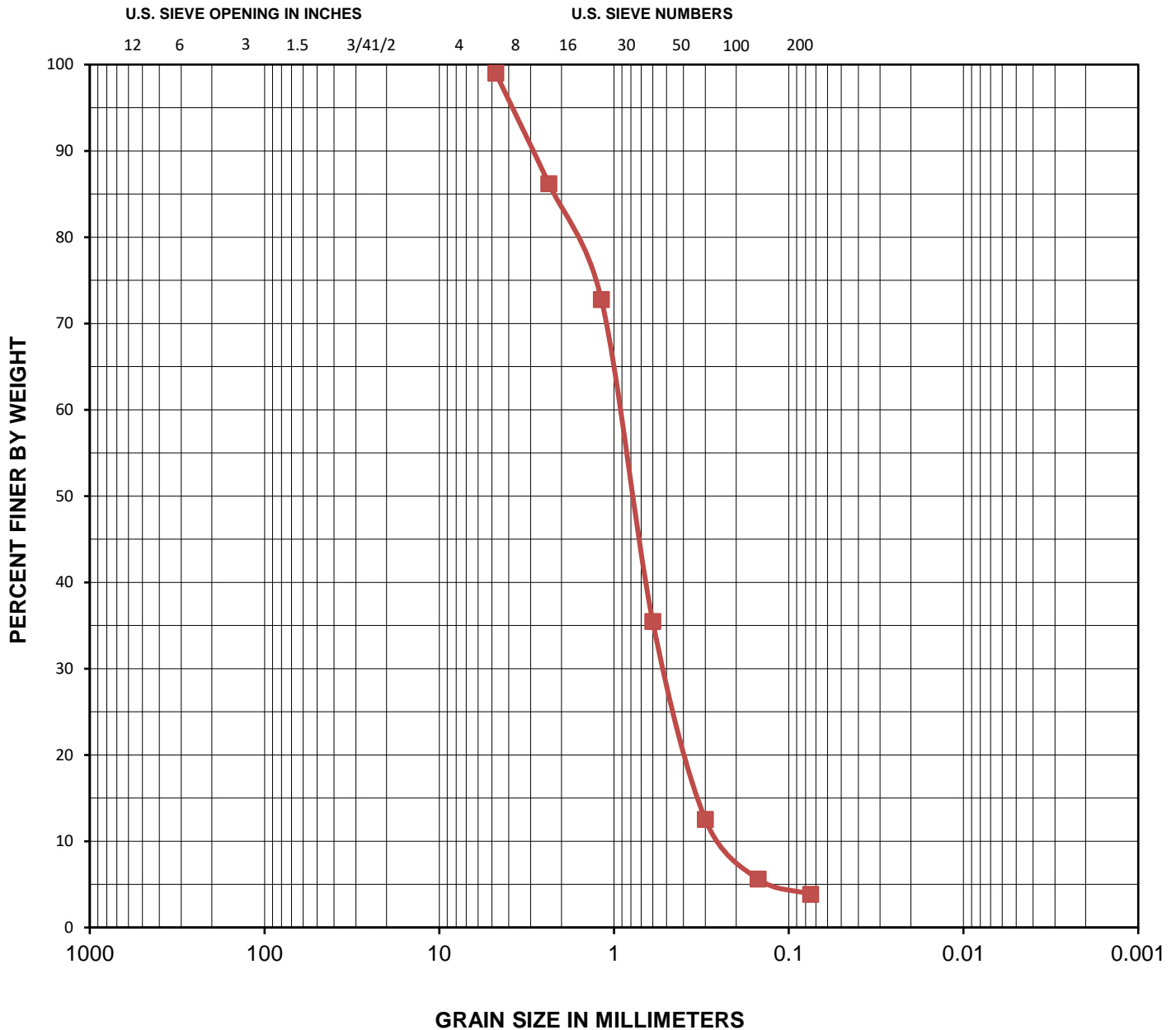
LABORATORY TESTS

APPENDIX B

BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY
		coarse	fine	coarse	medium	fine		



BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY
		coarse	fine	coarse	medium	fine		



Boring	Depth (ft.)	Sample Description	Passing 3/4"	Passing #4	Passing #200
— B-10	15	Poorly Graded SAND (SP)	100.0	99.0	3.9

PROJECT NO.: 240005
 LAB TECH: SA
 INPUT BY: SA
 CHECKED BY: SA
 DATE: 2/5/2024
 REVISED: -

SIEVE ANALYSIS

CLOVIS COMM. COLLEGE SPORTS COMPLEX
 10309 N WILLOW AVE
 CLOVIS, CALIFORNIA



Boring	Depth (ft.)	Sample Description
B-9	0-5	Clayey SAND (SC)

Moisture		
Wet Weight (g)	Dry Weight (g)	Water Content (%)
200.0	187	7.0

Soil Specimen		
Mold Weight (g)	Soil + Mold Weight (g)	Soil Weight (g)
367.4	799.9	432.5
Mold Diameter (in)	Mold Height (in)	Mold Volume (ft ³)
4.0	1.0	12.57
Moist Density (pcf)	Dry Density (pcf)	Saturation (%)
130.4	122.0	49.2

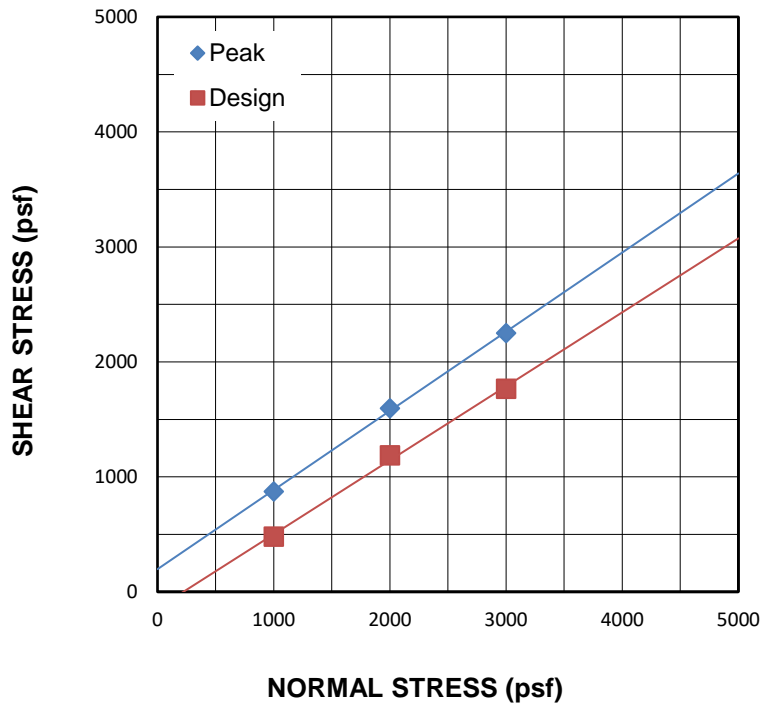
Expansion		
Initial Reading (in)	Final Reading (in)	Expansion (in)
0.0000	0.0043	0.0043

Expansion Index, EI	
EI _{measured}	EI ₅₀
4.3	4.0

Expansion Index, EI	Potential Expansion
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
> 130	Very High

Testing performed in general accordance with ASTM D4829

PROJECT NO	240005	EXPANSION INDEX	
LAB TECH:			
INPUT BY:	CO	CLOVIS COMM. COLLEGE SPORTS COMPLEX	
CHECKED BY:	AA	10309 N WILLOW AVE	
DATE:	2/5/2024	CLOVIS, CALIFORNIA	
REVISED:	-		



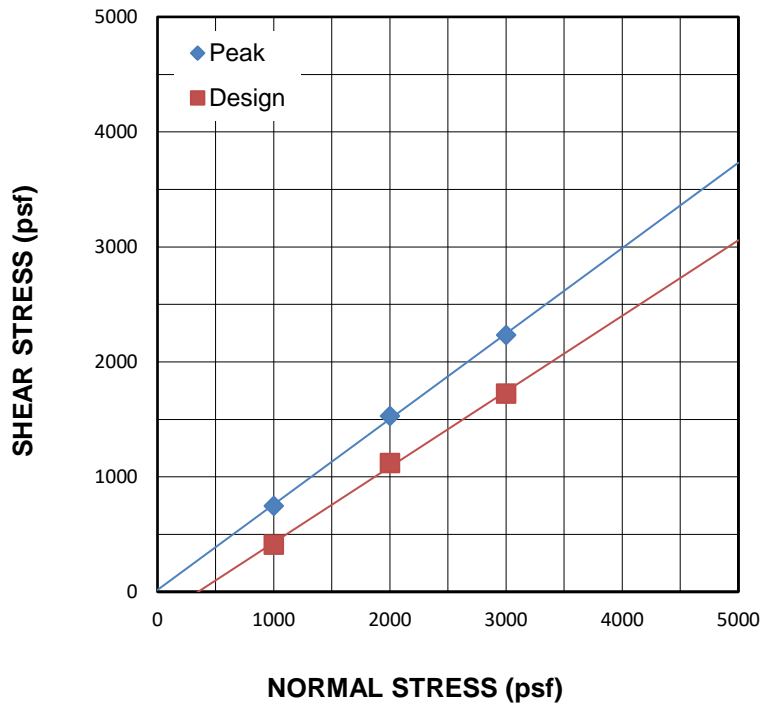
Depth (ft.)	Sample Description
B-2 1	Clayey SAND (SC)

Initial	Specimen No.	Dry Unit Weight (pcf)	Water Content (%)	Saturation (%)	Area (in ²)	Height (in)
	1	115.5	4.3	26.4	4.60	1.00
	2	115.5	4.3	26.4	4.60	1.00
	3	115.5	4.3	26.4	4.60	1.00
At Test	Specimen No.	Dry Unit Weight (pcf)	Water Content (%)	Saturation (%)	Area (in ²)	Height (in)
	1	116.9	12.6	80.6	4.60	0.988
	2	118.6	12.3	82.7	4.60	0.973
	3	118.6	12.2	82.0	4.60	0.973

Specimen No.	Peak Shear Stress (psf)	Design Shear Stress (psf)	Normal Stress (psf)	Strain Rate (in/min)
1	873.8	479.0	1000	0.005
2	1597.2	1186.5	2000	0.005
3	2250.4	1765.7	3000	0.005

Results	Cohesion (psf)	Friction ϕ (deg)
Peak	197	34.5
Design	0	32.8

PROJECT NO	240005	DIRECT SHEAR	
LAB TECH:			
INPUT BY:	CO	CLOVIS COMM. COLLEGE SPORTS COMPLEX	
CHECKED BY:	AA	10309 N WILLOW AVE	
DATE:	2/5/2024	CLOVIS, CALIFORNIA	
REVISED:	-		



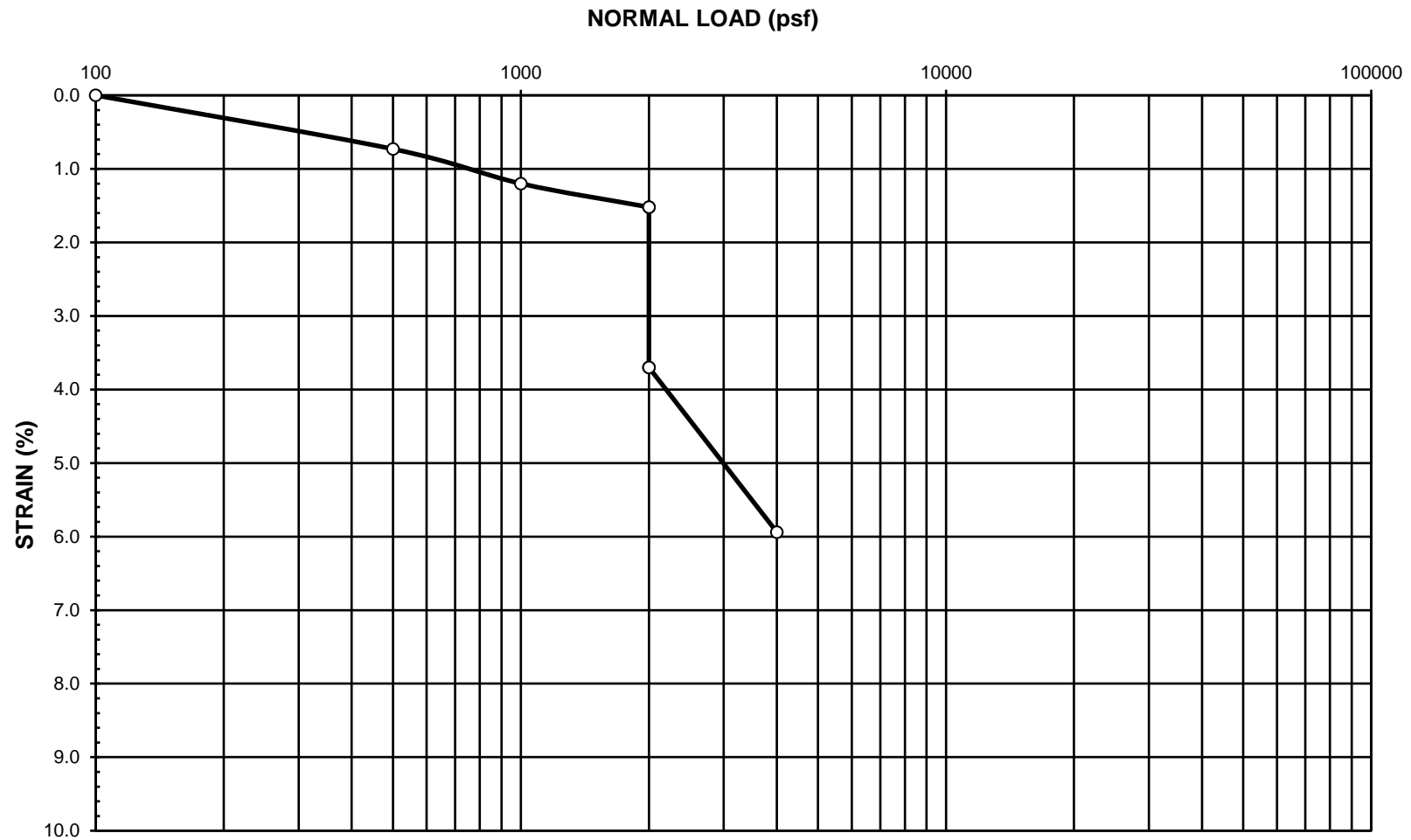
Depth (ft.)	Sample Description
B-7 1	Clayey SAND (SC)

Initial	Specimen No.	Dry Unit Weight (pcf)	Water Content (%)	Saturation (%)	Area (in ²)	Height (in)
	1	115.5	4.3	26.4	4.60	1.00
	2	115.5	4.3	26.4	4.60	1.00
	3	115.5	4.3	26.4	4.60	1.00
At Test	Specimen No.	Dry Unit Weight (pcf)	Water Content (%)	Saturation (%)	Area (in ²)	Height (in)
	1	117.7	15.3	100.2	4.60	0.981
	2	117.8	16	105.0	4.60	0.980
	3	118.5	13.5	90.6	4.60	0.974

Specimen No.	Peak Shear Stress (psf)	Design Shear Stress (psf)	Normal Stress (psf)	Strain Rate (in/min)
1	746.1	410.1	1000	0.002
2	1530.0	1120.8	2000	0.002
3	2232.1	1725.0	3000	0.002

Results	Cohesion (psf)	Friction ϕ (deg)
Peak	17	36.6
Design	0	33.3

PROJECT NO	240005	DIRECT SHEAR CLOVIS COMM. COLLEGE SPORTS COMPLEX 10309 N WILLOW AVE CLOVIS, CALIFORNIA	
LAB TECH:			
INPUT BY:	CO		
CHECKED BY:	AA		
DATE:	2/5/2024		
REVISED:	-		



Boring	Depth (ft)	Sample Description
B-4	1.0	Clayey SAND (SC)

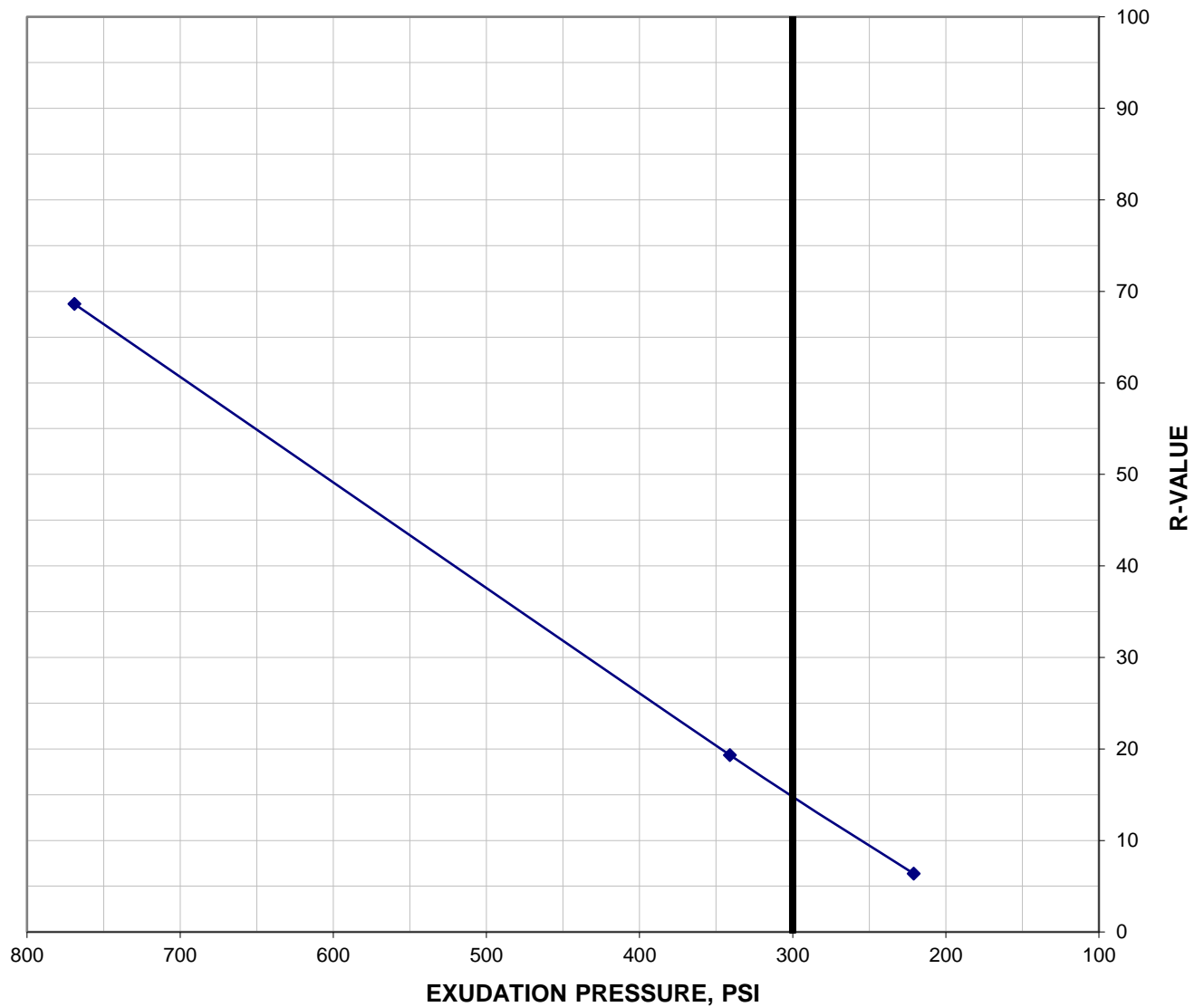
	Sample Diameter (in)	Sample Height (in)	Moisture Content (%)	Dry Density (pcf)
Initial	2.42	1.0000	5.9	120.1
Final	2.42	0.9406	11.8	127.7

PROJECT NO.: 240005
 LAB TECH: WJD
 INPUT BY: CO
 CHECKED BY: AA
 DATE: 2/5/2024
 REVISED: -

COLLAPSE POTENTIAL

CLOVIS COMM. COLLEGE SPORTS COMPLEX
 10309 N WILLOW AVE
 CLOVIS, CALIFORNIA




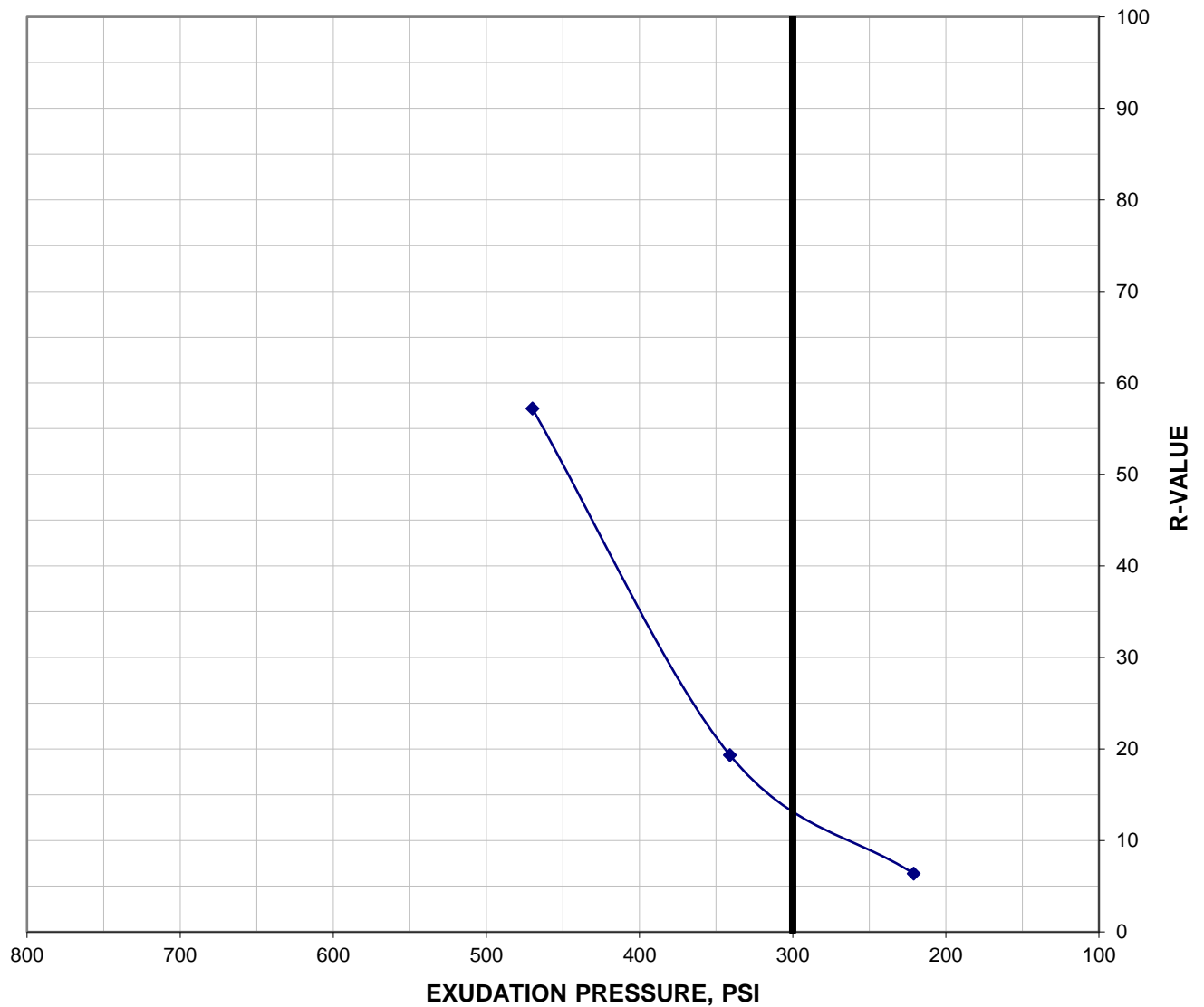


Boring	Depth (ft)	Sample Description
RV-1	0-5	Clayey SAND (SC)

Specimen	1	2	3
Exudation Pressure (psi)	221	341	769
Moisture Content at Test (%)	12.0	11.4	8.6
Dry Density (pcf)	124.9	127.5	129.5
Expansion Pressure (psf)	91	117	156
R-Value by Stabilometer	6	19	69
R-Value by Expansion Pressure (TI = 4.5)	NA		
R-Value at 300 psi Exudation Pressure	19		

Controlling R-Value	19
---------------------	----


PROJECT NO:	240005	RESISTANCE VALUE	
LAB TECH:	JC		
INPUT BY:	CO	LOVIS COMM. COLLEGE SPORTS COMPLE	
CHECKED BY:	AA	10309 N WILLOW AVE	
DATE:	2/5/2024	CLOVIS, CALIFORNIA	
REVISED:	-		

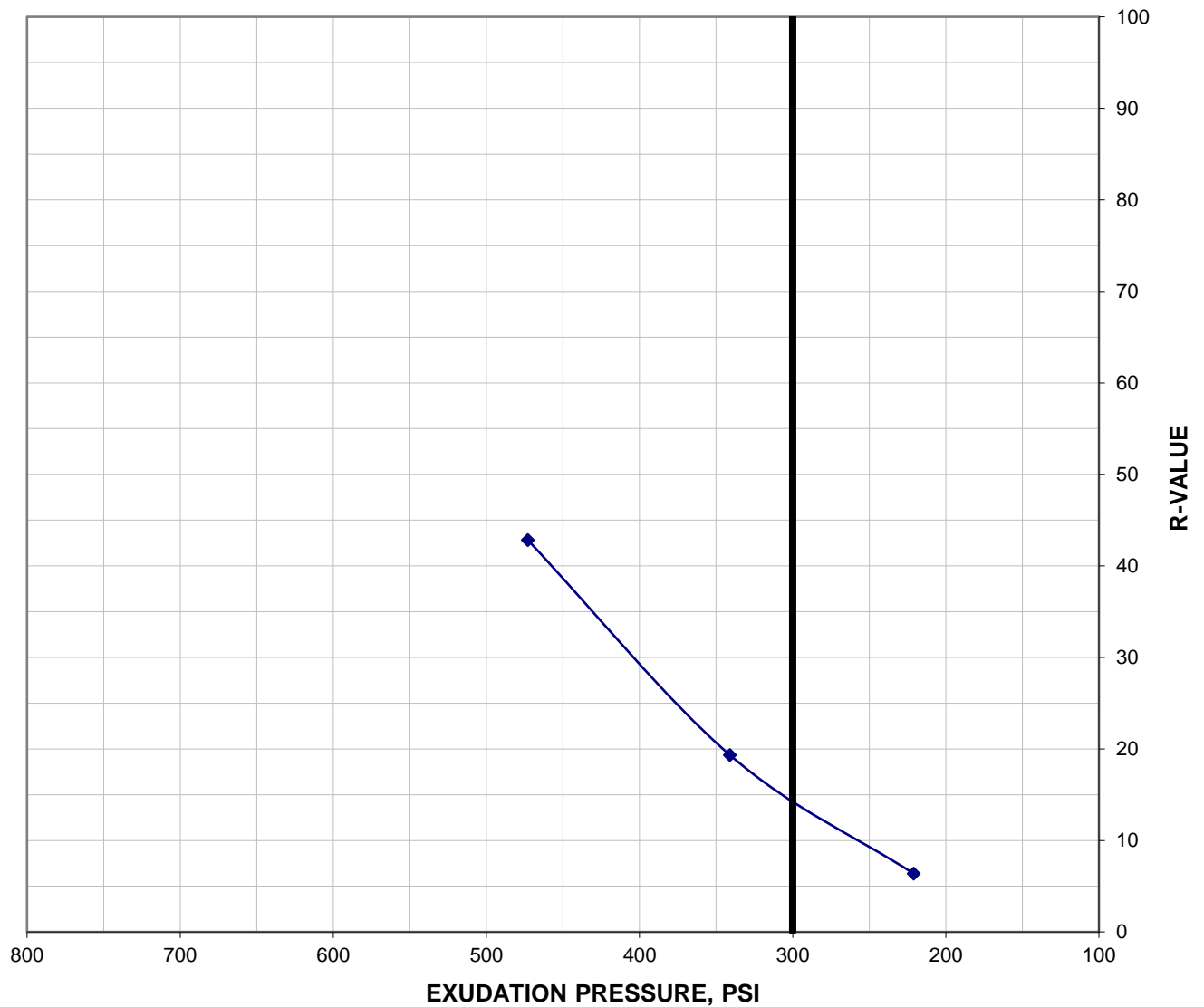


Boring	Depth (ft)	Sample Description
RV-2	0-5	Clayey SAND (SC)

Specimen	1	2	3
Exudation Pressure (psi)	221	341	470
Moisture Content at Test (%)	10.5	9.9	8.2
Dry Density (pcf)	126.6	129.2	130.7
Expansion Pressure (psf)	91	117	48
R-Value by Stabilometer	6	19	57
R-Value by Expansion Pressure (TI = 4.5)	NA		
R-Value at 300 psi Exudation Pressure	13		

Controlling R-Value	13
---------------------	----


PROJECT NO:	240005	RESISTANCE VALUE	
LAB TECH:	JC		
INPUT BY:	CO	LOVIS COMM. COLLEGE SPORTS COMPLE	
CHECKED BY:	AA	10309 N WILLOW AVE	
DATE:	2/5/2024	CLOVIS, CALIFORNIA	
REVISED:	-		



Boring	Depth (ft)	Sample Description
RV-3	0-5	Clayey SAND (SC)

Specimen	1	2	3
Exudation Pressure (psi)	221	341	473
Moisture Content at Test (%)	9.9	9.2	8.5
Dry Density (pcf)	127.3	129.9	131.1
Expansion Pressure (psf)	91	117	74
R-Value by Stabilometer	6	19	43
R-Value by Expansion Pressure (TI = 4.5)	NA		
R-Value at 300 psi Exudation Pressure	14		

Controlling R-Value	14
---------------------	----

PROJECT NO:	240005	RESISTANCE VALUE	
LAB TECH:	JC		
INPUT BY:	CO	LOVIS COMM. COLLEGE SPORTS COMPLE	
CHECKED BY:	AA	10309 N WILLOW AVE	
DATE:	2/5/2024	CLOVIS, CALIFORNIA	
REVISED:	-		

Boring	Depth (ft)	Sample Description
B-1	0-5	Sandy SILT (ML)

--

MINIMUM RESISTIVITY									
---------------------	--	--	--	--	--	--	--	--	--

Water Added (ml)	0	150	250	350					
Resistance (ohm)	1,000,000	15,000	4,470	7,860					
Resistivity (ohm-cm)*	1,065,000	15,975	4,761	8,371					

Box Constant=1.065

Minimum Resistivity (ohm-cm)	4,761
pH	6.12

Years to perforation*	18
-----------------------	----

* Caltrans California Test 643 - Method for Estimating the Service Life of Steel Culverts


--

CHEMICAL ANALYSIS

	<table><tr><th>Soluble Sulfate SO₄-S</th></tr><tr><td>24.3 mg/kg</td></tr><tr><td>22.6 mg/kg</td></tr><tr><td>28.8 mg/kg</td></tr></table>	Soluble Sulfate SO ₄ -S	24.3 mg/kg	22.6 mg/kg	28.8 mg/kg	<table><tr><th>Soluble Chloride Cl</th></tr><tr><td>58.5 mg/kg</td></tr><tr><td>62 mg/kg</td></tr><tr><td>58.5 mg/kg</td></tr></table>	Soluble Chloride Cl	58.5 mg/kg	62 mg/kg	58.5 mg/kg
Soluble Sulfate SO ₄ -S										
24.3 mg/kg										
22.6 mg/kg										
28.8 mg/kg										
Soluble Chloride Cl										
58.5 mg/kg										
62 mg/kg										
58.5 mg/kg										
Average	25.2 mg/kg	59.7 mg/kg								

--

Testing performed in general accordance with California Test Method Nos. 643, 417, and 422

PROJECT NO.: 240005	<div>CORROSIVITY TESTS</div> <div>CLOVIS COMM. COLLEGE SPORTS COMPLEX</div> <div>10309 N WILLOW AVE</div> <div>CLOVIS, CALIFORNIA</div>	
LAB TECH:		
INPUT BY: CO		
CHECKED BY: AA		
DATE: 2/5/2024		
REVISED: -		

USGS DEAGGREGATION SUMMARIES

APPENDIX C

Unified Hazard Tool



Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new [USGS Earthquake Hazard Toolbox](#) for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

^ Input

Edition

Dynamic: Conterminous U.S. 2014 (u...

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

36.8825

Time Horizon

Return period in years

2475

Longitude

Decimal degrees, negative values for western longitudes

-119.7338

Site Class

259 m/s (Site class D)

^ Hazard Curve



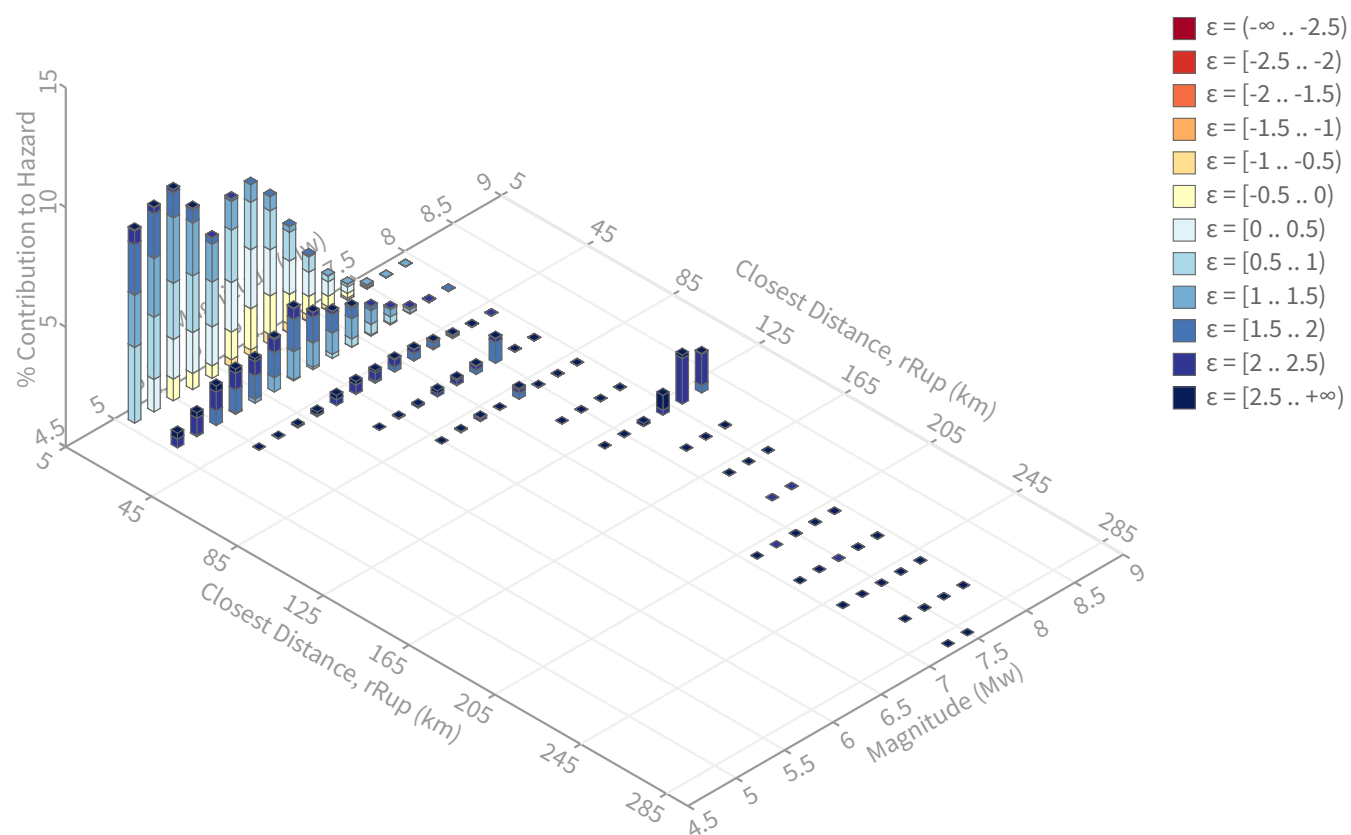
Please select “Edition”, “Location” & “Site Class” above to compute a hazard curve.

Compute Hazard Curve

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr⁻¹
PGA ground motion: 0.33178019 g

Recovered targets

Return period: 2703.4612 yrs
Exceedance rate: 0.00036989619 yr⁻¹

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.16 %

Mean (over all sources)

m: 6.15
r: 22.41 km
ε₀: 1.04 σ

Mode (largest m-r bin)

m: 5.5
r: 10.86 km
ε₀: 0.86 σ
Contribution: 8.81 %

Mode (largest m-r-ε₀ bin)

m: 5.1
r: 6.67 km
ε₀: 0.73 σ
Contribution: 3.16 %

Discretization

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

Epsilon keys

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

Deaggregation Contributors

Source Set	Source	Type	r	m	ϵ_0	lon	lat	az	%
UC33brAvg_FM31 (opt)		Grid							46.66
	PointSourceFinite: -119.734, 36.914		6.13	5.66	0.19	119.734°W	36.914°N	0.00	3.47
	PointSourceFinite: -119.734, 36.914		6.13	5.66	0.19	119.734°W	36.914°N	0.00	3.01
	PointSourceFinite: -119.734, 36.941		7.96	5.73	0.43	119.734°W	36.941°N	0.00	2.70
	PointSourceFinite: -119.734, 36.941		7.96	5.73	0.43	119.734°W	36.941°N	0.00	2.53
	PointSourceFinite: -119.734, 37.013		13.74	5.97	0.97	119.734°W	37.013°N	0.00	1.82
	PointSourceFinite: -119.734, 36.977		10.75	5.85	0.72	119.734°W	36.977°N	0.00	1.80
	PointSourceFinite: -119.734, 36.968		10.02	5.82	0.66	119.734°W	36.968°N	0.00	1.78
	PointSourceFinite: -119.734, 36.977		10.75	5.85	0.72	119.734°W	36.977°N	0.00	1.74
	PointSourceFinite: -119.734, 37.013		13.74	5.97	0.97	119.734°W	37.013°N	0.00	1.69
	PointSourceFinite: -119.734, 36.968		10.02	5.82	0.66	119.734°W	36.968°N	0.00	1.67
	PointSourceFinite: -119.734, 37.040		16.05	6.05	1.12	119.734°W	37.040°N	0.00	1.31
	PointSourceFinite: -119.734, 37.040		16.05	6.05	1.12	119.734°W	37.040°N	0.00	1.09
	PointSourceFinite: -119.734, 36.995		12.23	5.91	0.85	119.734°W	36.995°N	0.00	1.02
UC33brAvg_FM32 (opt)		Grid							46.59
	PointSourceFinite: -119.734, 36.914		6.13	5.66	0.19	119.734°W	36.914°N	0.00	3.47
	PointSourceFinite: -119.734, 36.914		6.13	5.66	0.19	119.734°W	36.914°N	0.00	3.01
	PointSourceFinite: -119.734, 36.941		7.96	5.73	0.44	119.734°W	36.941°N	0.00	2.70
	PointSourceFinite: -119.734, 36.941		7.96	5.73	0.44	119.734°W	36.941°N	0.00	2.53
	PointSourceFinite: -119.734, 37.013		13.74	5.97	0.97	119.734°W	37.013°N	0.00	1.82
	PointSourceFinite: -119.734, 36.977		10.75	5.85	0.72	119.734°W	36.977°N	0.00	1.79
	PointSourceFinite: -119.734, 36.968		10.03	5.82	0.66	119.734°W	36.968°N	0.00	1.78
	PointSourceFinite: -119.734, 36.977		10.75	5.85	0.72	119.734°W	36.977°N	0.00	1.74
	PointSourceFinite: -119.734, 37.013		13.74	5.97	0.97	119.734°W	37.013°N	0.00	1.69
	PointSourceFinite: -119.734, 36.968		10.03	5.82	0.66	119.734°W	36.968°N	0.00	1.67
	PointSourceFinite: -119.734, 37.040		16.05	6.05	1.12	119.734°W	37.040°N	0.00	1.30
	PointSourceFinite: -119.734, 37.040		16.05	6.05	1.12	119.734°W	37.040°N	0.00	1.09
	PointSourceFinite: -119.734, 36.995		12.23	5.91	0.85	119.734°W	36.995°N	0.00	1.02
UC33brAvg_FM32		System							3.38
	San Andreas (Creeping Section) [3]		120.70	8.16	2.24	120.703°W	36.133°N	226.38	1.83
UC33brAvg_FM31		System							3.37
	San Andreas (Creeping Section) [3]		120.70	8.16	2.24	120.703°W	36.133°N	226.38	1.83

SITE SPECIFIC GROUND MOTION ANALYSIS

APPENDIX D

Site-Specific Ground Motion Analysis (per ASCE 7-16)

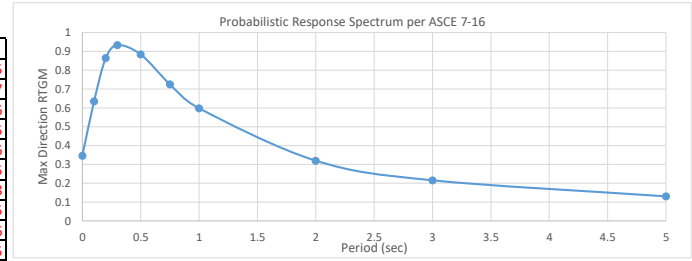
Technicon Engineering Services, Inc.	
Project:	Proposed Sports Complex
Job #:	240005
Date:	2/8/2024
Checked by:	S. Alvarez
S_s	0.531
S_1	0.213
S_{DS}	0.487
PGA_M	0.315
F_a	1.375



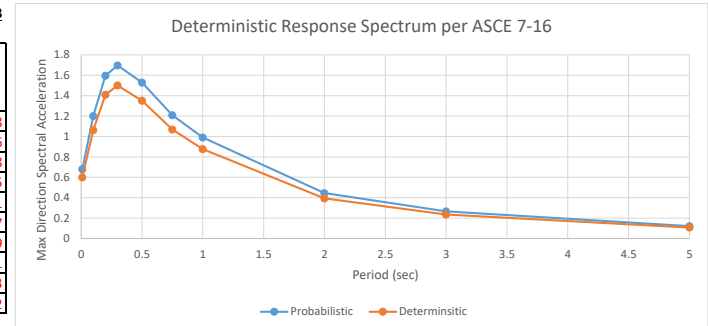
INPUT
OUTPUT
ANALYSIS

1. Use Unified Hazard Tool "raw data" from Hazard Curve & Risk-Targeted Ground Motion Calculator to get "UHGM & RTGM" values
 - a. Plot time vs. adjusted RTGM
2. Input M_w and R_{rup} into NGAW2 Excel worksheet. M_w & R_{rup} can be found with deagg sheet (unified hazard tool) "Mean (over all sources)".
 - a. PS_s Median + 5% damping is 84th – percentile spectral acceleration

		* from RTGM Calculator			
Period (s)		UHGM (g)	RTGM (g)	Max Dir Scale Factor	Max Dir RTGM (g)
0		0.332	0.315	1.1	0.3465
0.1		0.611	0.577	1.1	0.6347
0.2		0.826	0.786	1.1	0.8646
0.3		0.87	0.83	1.125	0.93375
0.5		0.786	0.752	1.175	0.8836
0.75		0.614	0.586	1.2375	0.725175
1		0.482	0.46	1.3	0.598
2		0.25	0.237	1.35	0.31995
3		0.164	0.154	1.4	0.2156
5		0.095	0.087	1.5	0.1305



			Scaling Factor:		0.883887803
*From NGA-West2 GMPE Worksheet					
Period (s)	84th- percentile spectral acceleration (+1. σ for 5 % damping)	Max Dir Scale Factor	Max Dir Deterministic SA (prob.)	ASCE 7-16 SECTION 21.2.2 (Det.)	
0.01	0.616824038	1.1	0.678506442	0.599723568	
0.1	1.091668116	1.1	1.200834928	1.061403346	
0.2	1.449903188	1.1	1.594893507	1.409706918	
0.3	1.508487082	1.125	1.697047967	1.5	
0.5	1.300665177	1.175	1.528281583	1.350829451	
0.75	0.977181226	1.2375	1.209261767	1.068851727	
1	0.761839703	1.3	0.990391615	0.875395069	
2	0.329941726	1.35	0.445421331	0.393702481	
3	0.190702689	1.4	0.266983764	0.235983693	
5	0.081030829	1.5	0.121546243	0.107433242	



<div>- ASCE 7-16 Section 21.2.2</div> <div>If Largest Deterministic Spectral acceleration < 1.5, then scaling by a factor of $F_a 1.5$.</div> <div>Table 11.4.1 : Site Class D @ $S_s \geq 1.5$ → $F_a =$ 1.375</div> <div>$F_a 1.5$ → $F_{a,s}$ 2.0625</div>	<div>- Section 21.3</div> <div>F_v is taken as 2.4 for $S_1 < 0.2$ or 2.5 for $S_1 > 0.2$</div> <div>F_v ⇒ 2.5</div>												
<div>- Section 11.4.6 - Design Response Spectrum</div> <div>$T_0 = 0.2 \left(\frac{S_{D1}}{S_{DS}} \right)$ $T_s = \left(\frac{S_{D1}}{S_{DS}} \right)$</div> <div>equ. 11.4-2: $S_{M1} = S_1 * F_v$ → 0.79875</div> <div>equ. 11.4-4: $S_{D1} = \left(\frac{2}{3} \right) S_{M1}$ → 0.533</div> <div>T_0 ⇒ 0.219</div> <div>T_s ⇒ 1.093</div>	<table><tr><td>S_s</td><td>0.531</td></tr><tr><td>S_1</td><td>0.213</td></tr><tr><td>S_{DS} * from seismic design map</td><td>0.487</td></tr><tr><td>S_{D1} * from section 11.4.6</td><td>0.533</td></tr><tr><td>T_0</td><td>0.219</td></tr><tr><td>T_s</td><td>1.093</td></tr></table>	S_s	0.531	S_1	0.213	S_{DS} * from seismic design map	0.487	S_{D1} * from section 11.4.6	0.533	T_0	0.219	T_s	1.093
S_s	0.531												
S_1	0.213												
S_{DS} * from seismic design map	0.487												
S_{D1} * from section 11.4.6	0.533												
T_0	0.219												
T_s	1.093												

Site-Specific Ground Motion Analysis (per ASCE 7-16) - cont.

Technicon Engineering Services, Inc.	
Project:	Proposed Sports Complex
Job #:	240005
Date:	2/8/2024
Checked by:	S. Alvarez
S_s	0.531
S_1	0.213
S_{D5}	0.487
PGA_M	0.315
F_a	1.375



INPUT
OUTPUT
ANALYSIS

Site-Specific Response Spectra (Section 11.4.6)

Period (T) (sec)	Code-Base -Spectrum Design spectral response acceleration (S_a)	*make sure below applies to period (T) sec	80% Code-Based	$S_a = (2/3)(S_{am})$ (prob. Design)	(Sec. 21.4) $T \cdot S_a$
0.01	0.208161634	T less than T_0	0.166529307	0.231	0.00231
0.1	0.328416338		0.26273307	0.423133333	0.042313333
0.2	0.462032676		0.369626141	0.5764	0.11528
0.3	0.487		0.3896	0.6225	0.18675
0.5	0.487		0.3896	0.589066667	0.294533333
0.75	0.487	$T_0 < T < T_g$; $T = S_{D5}$	0.3896	0.48345	0.3625875
1	0.487		0.3896	0.398666667	0.398666667
2	0.26625		0.213	0.2133	0.4266
3	0.1775		0.142	0.143733333	0.4312
5	0.1065		0.0852	0.087	0.435

- Section 21.4 Design Acceleration Parameters

Max S_a between $T = 0.2 - 5$ sec (From Design Spectrum (prob.) graph)

$S_{a_{max}} \rightarrow 0.6225$

$$S_{D5} = 90\% \cdot S_{a_{max}} \rightarrow 0.560$$

$$S_{M5} = 1.5 \cdot S_{D5} \rightarrow 0.840$$

$V_{s30} < 365$ m/s

Max $T \cdot S_a$ between $T = 1$ sec - 5 sec (From Design Spectrum (prob.) graph)

Max S_a between $T = 1 - 5$ sec $\rightarrow 0.435$

$S_{D1} \rightarrow 0.435$

$$S_{M1} = 1.5 \cdot S_{D1} \rightarrow 0.653$$

- Section 21.5.1 - Probabilistic MCE_g Peak Ground Acceleration

Probabilistic PGA from UHGM @ $T = 0$ sec

$PGA_{prob.} \rightarrow 0.332$

- Section 21.5.2 - Deterministic MCE_g Peak Ground Acceleration

Deterministic PGA from 84th Spectral Acceleration @ $T = 0.01$ sec

PGA $\rightarrow 0.617$

Table 11.8-1: Site Class D @ $PGA = 0.5 \rightarrow F_{PGA} = 1.37$

$0.5F_{PGA} = 0.685$

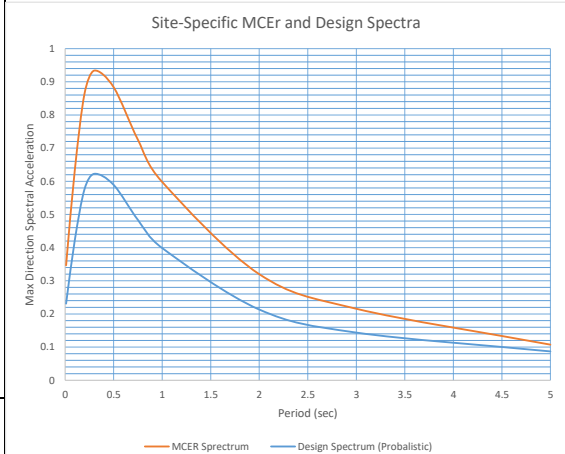
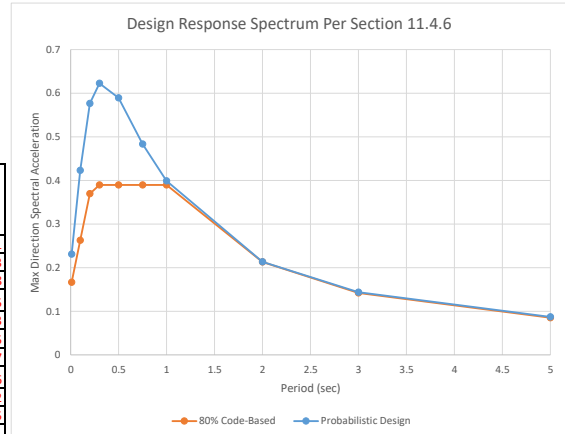
Use greater of PGA or $0.5F_{PGA}$

Therefore; $PGA_{det.} \rightarrow 0.685$

- Section 21.5.3 - Site Specific MCE_g Peak Ground Acceleration

$PGA_{prob.} \rightarrow 0.332$ *Take the lesser

$PGA_{det.} \rightarrow 0.685$ $PGA_{site} \rightarrow 0.332$



PGA CHECK

From Seismic Design Map: $PGA_M \rightarrow 0.315$

80 % of $PGA_M \rightarrow 0.252$

Site-Specific PGA \rightarrow *Take the greater 0.332

Final Seismic Design Values	
S_s	0.531
S_1	0.213
S_{M5}	0.840
S_{D5}	0.560
S_{D1}	0.435
S_{M1}	0.653
F_a	1.375
F_v	2.500
PGA_M	0.332

**LIQUEFACTION ANALYSIS AND SEISMICALLY
INDUCED SETTLEMENT CALCULATIONS
APPENDIX E**

Proposed Sports Complex
DSA File
DSA App No.

Calc by AA Date 2/13/24
Checked by SA Date 3/18/24

Project No: TES 240005
Boring: B-02

Liquefaction analysis is performed following Seed's Procedure, outlined by Seed and Harder (1990), as modified in 1998 NCEER Workshops. Reference Youd et al., 2001

**Includes revisions proposed by Youd (2001)

The induced cyclic stress ratio (CSR) by a given peak ground acceleration (a_{max}) is:

**CSR = $(s_{av})/s'_{vo} = 0.65 (s_{av}/s'_{vo})(a_{max}/g) r_d$ MSF

where: **Magnitude Scaling Factor, MSF = $31.623^{(exp(-0.4605^Mw))}$

 **Stress Reduction Factor, $r_d =$

$$\frac{1.000-0.4113z^{0.15}+0.04052z+0.001753z^{1.5}}{1.00-0.4177z^{0.15}+0.05729z-0.006205z^{1.5}+0.001210z^2}$$

a_{max} = maximum peak acceleration at the ground surface (g's)

g = acceleration of gravity Mw = Moment Magnitude

Rod Length = 1.22 meters above grounds surface

Hammer Efficiency = 88% Emean/E60 = Energy Ratio to correct to standard 60% Energy

Ring Sampler Corr. = 0.65

The cyclic resistance ratio (CRR) is now read directly from the curve for

clean sands under level ground conditions based on the corrected SPT value.

This SPT N value is now corrected for earthquake magnitude, fines, energy,

overburden pressure, & sampler factors.

The CSR factors in a magnitude scaling factor and a stress reduction coefficient.

Factor of Safety, F_L is:

$$F_L = \frac{CRR}{CSR} = \text{Uniform CSR necessary to trigger liquefaction/Equivalent, Uniform, earthquake induced CSR}$$

Hammer
Efficiencies -
Technicon Drilling
Rigs

CME 45	80.0%
CME 55	82.4%
CME 75	87.8%
SIMCO	88.0%

Emean/E60=		1.467		Sur.=		0		psf		Measured Ground Water Depth =				100				feet				Design Ground Water Depth =				46				feet				acc. max =				0.322				g				Earthq. Mw = 6.15			
Depth to Bottom of Layer (ft.)	Boring Diameter (in)	Soil Type	Layer Thickness (ft.)	Total Overburden Press. σ_{vs} (tsf)	Effect. Overburden Press. σ'_{ve} (tsf) at Measured Ground Water Depth	Effect. Overburden Press. σ'_{ve} (tsf) at Design Ground Water Depth	Midpoint Below Ground Surface (m)	Cn	Total Unit Wt. (pcf) at Measured Ground Water Depth	Total Unit Wt. (pcf) at Design Ground Water Depth	Sampler Type 1 = SPT 2=Ca.Mod	Field Blow Count N	α	β	Stress Reduct. Coeff. rd	MSF	Est. % Fines	C_u	C_r	C_e	$C_u C_r C_e$	Corrected Blow Count $(N_1)_{60}$	$(N_1)_{60cs}$	CSR _{r,s} Induced	CRR _{r,s} (Resist. - c.sand)	Factor of Safety F_L	Will It Liquefy?																						
3	4	SC	3	0.09	0.09	0.09	0.5	1.70	123	125	2	38	5.000	1.200	0.997	1.86	46.0	1.0	0.75	1.00	0.75	46.2	60.4	0.110	LARGE	LARGE	ABOVE																						
8	4	SC	5	0.35	0.35	0.36	1.7	1.44	136	136	2	40	5.000	1.200	0.987	1.86	46.0	1.0	0.75	1.00	0.75	41.2	54.5	0.110	LARGE	LARGE	ABOVE																						
11	4	SC	3	0.63	0.63	0.63	2.9	1.24	136	136	1	14	5.000	1.200	0.978	1.86	46.0	1.0	0.85	1.20	1.02	25.9	36.1	0.110	LARGE	LARGE	ABOVE																						
12.5	4	SP	1.5	0.77	0.77	0.77	3.6	1.15	108	108	1	14	0.000	1.000	0.973	1.86	4.0	1.0	0.85	1.20	1.02	24.1	24.1	0.109	0.276	2.52	ABOVE																						
17.5	4	SC	5	0.98	0.98	0.98	4.6	1.05	136	136	2	56	5.000	1.200	0.965	1.86	46.0	1.0	0.85	1.00	0.85	47.5	62.0	0.108	LARGE	LARGE	ABOVE																						
22.5	4	ML	5	1.28	1.28	1.28	6.1	0.93	101	101	1	15	5.000	1.200	0.953	1.86	55.0	1.0	0.95	1.20	1.14	23.2	32.9	0.107	LARGE	LARGE	ABOVE																						
27.5	4	SC	5	1.57	1.57	1.57	7.6	0.83	138	138	2	53	4.931	1.188	0.942	1.86	34.0	1.0	0.95	1.00	0.95	39.8	52.3	0.106	LARGE	LARGE	ABOVE																						
30.5	4	SC	3	1.85	1.85	1.85	8.8	0.76	138	138	1	53	4.931	1.188	0.932	1.86	34.0	1.0	1.00	1.20	1.20	70.6	88.9	0.105	LARGE	LARGE	ABOVE																						
32.5	4	ML	2	2.00	2.00	2.00	9.6	0.72	101	101	1	17	5.000	1.200	0.918	1.86	55.0	1.0	1.00	1.20	1.20	21.6	30.9	0.103	LARGE	LARGE	ABOVE																						
37.5	4	ML	5	2.18	2.18	2.18	10.7	0.69	101	101	2	58	5.000	1.200	0.889	1.86	55.0	1.0	1.00	1.00	1.00	37.9	50.5	0.100	LARGE	LARGE	ABOVE																						
40.5	4	ML	3	2.38	2.38	2.38	11.9	0.65	101	101	2	58	5.000	1.200	0.857	1.86	55.0	1.0	1.00	1.00	1.00	35.8	48.0	0.096	LARGE	LARGE	ABOVE																						
42.5	4	SM	2	2.51	2.51	2.51	12.6	0.63	101	105	1	52	4.931	1.188	0.836	1.86	34.0	1.0	1.00	1.20	1.20	57.3	73.0	0.094	LARGE	LARGE	ABOVE																						
46	4	SC	3.5	2.64	2.64	2.64	13.5	0.61	86	86	2	53	4.931	1.188	0.814	1.86	34.0	1.0	1.00	1.00	1.00	30.6	41.3	0.091	LARGE	LARGE	ABOVE																						
47.5	4	SC	1.5	2.74	2.74	2.73	14.2	0.59	86	113	2	53	4.931	1.188	0.794	1.86	34.0	1.0	1.00	1.00	1.00	29.8	40.4	0.090	LARGE	LARGE	NO																						
50	4	SC	2.5	2.83	2.83	2.78	14.9	0.40	86	113	2	53	4.931	1.188	0.777	1.86	34.0	1.0	1.00	1.00	1.00	20.2	28.9	0.089	LARGE	LARGE	NO																						

Proposed Sports Complex
DSA File
DSA App No.

Calc by AA Date 2/13/24
Checked by SA Date 3/18/24

Project No: **TES 240005**
Boring: **B-02**

Liquefaction analysis is performed following Seed's Procedure, outlined by Seed and Harder (1990), as modified in 1998 NCEER Workshops. Reference Youd et al., 2001

**Includes revisions proposed by Youd (2001)

The induced cyclic stress ratio (CSR) by a given peak ground acceleration (a_{max}) is:

$$**CSR = (t_{av})/s_{vo} = 0.65 (s_{vo}/s'_{vo})(a_{max}/g) r_d MSF$$

where: **Magnitude Scaling Factor, MSF = $31.623 \cdot (\exp(-0.4605 \cdot Mw))$

$$**Stress Reduction Factor, r_d =$$

$$1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}$$

$$1.00 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2$$

a_{max} = maximum peak acceleration at the ground surface (g's)

g = acceleration of gravity

Mw = Moment Magnitude

The cyclic resistance ratio (CRR) is now read directly from the curve for

clean sands under level ground conditions based on the corrected SPT value.

This SPT N value is now corrected for earthquake magnitude, fines, energy,

overburden pressure, & sampler factors.

The CSR factors in a magnitude scaling factor and a stress reduction coefficient.

Settlement = e * Layer thickness in inches (Figure 9 1997 NCEER)

Rod Length = **1.22** meters above grounds surface

Hammer Efficiency = **88%**

E_{mean}/E₆₀ = Energy Ratio to correct to standard 60% Energy

Surcharge = Any surcharge on top of the ground (psf)

$^1C_N = (P_d/s'_{vo})^{0.5}$ Youd and Idriss 2001 Formula (9)

Ring Sampler Corr. = **0.65**

E _{mean} /E ₆₀ =		1.467		Sur.=		0		psf		Measured Ground Water Depth =		100		feet		Design Ground Water Depth =		46.0		feet		acc. max =		0.322		g		Earthq. Mw =		6.15	
Depth to Bottom of Layer (ft.)	Boring Diameter (in)	Soil Type	Layer Thickness (ft.)	Total Overburden Press. σ_{vo} (tsf)	Effect. Overburden Press. σ'_{vo} (tsf) at Measured Ground Water Depth	Effect. Overburden Press. σ'_{vo} (tsf) at Design Ground Water Depth	Midpoint Below Ground Surface (ft)	C _n	Total Unit Wt. (pcf) at Measured Ground Water Depth	Total Unit Wt. (pcf) at Design Ground Water Depth	Sampler Type 1 = SPT 2=Ca.Mod	Field Blow Count N	Stress Reduct. Coeff. r_d	MSF	Est. % Fines	C_u/C_L	Corrected Blow Count (N_1) _{CS}	ΔN	(N_1) _{CS}	CSR _{1.5} Induced	Factor of Safety F_L	σ (Only if FS<1.3) (%)	Settlement, inches								
3	4	SC	3	0.09	0.09	0.09	0.5	1.70	123	125	2	38	0.997	1.86	46.0	0.75	46.2	3.7	49.9	0.110	LARGE	-	ABOVE								
8	4	SC	5	0.35	0.35	0.36	1.7	1.44	136	136	2	40	0.987	1.86	46.0	0.75	41.2	3.7	44.9	0.110	LARGE	-	ABOVE								
11	4	SC	3	0.63	0.63	0.63	2.9	1.24	136	136	1	14	0.978	1.86	46.0	1.02	25.9	3.7	29.6	0.110	LARGE	-	ABOVE								
12.5	4	SP	1.5	0.77	0.77	0.77	3.6	1.15	108	108	1	14	0.973	1.86	4.0	1.02	24.1	0.4	24.5	0.109	2.52	-	ABOVE								
17.5	4	SC	5	0.98	0.98	0.98	4.6	1.05	136	136	2	56	0.965	1.86	46.0	0.85	47.5	3.7	51.2	0.108	LARGE	-	ABOVE								
22.5	4	ML	5	1.28	1.28	1.28	6.1	0.93	101	101	1	15	0.953	1.86	55.0	1.14	23.2	4.4	27.6	0.107	LARGE	-	ABOVE								
27.5	4	SC	5	1.57	1.57	1.57	7.6	0.83	138	138	2	53	0.942	1.86	34.0	0.95	39.8	2.7	42.6	0.106	LARGE	-	ABOVE								
30.5	4	SC	3	1.85	1.85	1.85	8.8	0.76	138	138	1	53	0.932	1.86	34.0	1.20	70.6	2.7	73.4	0.105	LARGE	-	ABOVE								
33	4	ML	2	2.00	2.00	2.00	9.6	0.72	101	101	1	17	0.918	1.86	55.0	1.20	21.6	4.4	26.0	0.103	LARGE	-	ABOVE								
37.5	4	ML	5	2.18	2.18	2.18	10.7	0.69	101	101	2	58	0.889	1.86	55.0	1.00	37.9	4.4	42.3	0.100	LARGE	-	ABOVE								
40.5	4	ML	3	2.38	2.38	2.38	11.9	0.65	101	101	2	58	0.857	1.86	55.0	1.00	35.8	4.4	40.2	0.096	LARGE	-	ABOVE								
42.5	4	SM	2	2.51	2.51	2.51	12.6	0.63	101	105	1	52	0.836	1.86	34.0	1.20	57.3	2.7	60.0	0.094	LARGE	-	ABOVE								
46	4	SC	3.5	2.64	2.64	2.64	13.5	0.61	86	86	2	53	0.814	1.86	34.0	1.00	30.6	2.7	33.4	0.091	LARGE	-	ABOVE								
47.5	4	SC	1.5	2.74	2.74	2.73	14.2	0.59	86	113	2	53	0.794	1.86	34.0	1.00	29.8	2.7	32.6	0.090	LARGE	-	NONE								
50	4	SC	2.5	2.83	2.83	2.78	14.9	0.40	86	113	2	53	0.777	1.86	34.0	1.00	20.2	2.7	22.9	0.089	LARGE	-	NONE								
51.5	4	ML	1.5	2.92	2.92	2.84	15.5	0.40	101	122.7	1	54	0.761	1.86	85.0	1.20	38.0	6.8	44.8	0.088	LARGE	-	NONE								

Total Settlement 0.0

May be off by 0.1 inches due to rounding

Proposed Sports Complex
DSA File
DSA App No.

Calc by AA Date 2/13/24
Checked by SA Date 3/18/24

Project No: TES 240005
Boring: B-02

Dynamic Dry Sand Settlement

$$g_{cyc} = [(t_{av})/s'_{vo}]/G_{max} = 0.65 (a_{max}/g) s_o t_d / G_{max}$$

$$\text{Where: } G_{max} = 20,000 [(N_1)_{60,cs}]^{0.33} [s'_m]^{0.5}$$

$$\text{Stress Reduction Factor, } r_d =$$

$$\frac{1.000-0.4113z^{0.5}+0.04052z+0.001753z^{1.5}}{1.00-0.4177z^{0.5}+0.05729z-0.006205z^{1.5}+0.001210z^2}$$

a_{max} = maximum peak acceleration at the ground surface (g's)

g = acceleration of gravity

- Notes: 1) Figure 9.51, Geotechnical Earthquake Engineering, Kramer
2) Figure 9.52b, Geotechnical Earthquake Engineering, Kramer
3) Table 9-4, Geotechnical Earthquake Engineering, Kramer

Sur.= 0 psf		Measured Ground Water Depth = 100 feet										acc. max = 0.322 g		Earthq. Mw = 6.15					
Elev. Base of Layer (ft)	Boring Diameter (in)	Soil Type	Layer Thickness (ft)	Depth to Midpoint (m)	Total Unit Wt. (pcf)	Total Overburden Pressure s_{vo} (psf)	Sampler Type 1 = SPT 2=Ca.Mod	Field Blow Count N (SPT)	Stress Reduct. Coeff. r_d	$(N_1)_{60cs}$	g_{eff} (G_{eff}/G_{max})	Cyclic Overburden Pressure s_{vo} (tsf)	⁽¹⁾ Cyclic Shear Strain, g_{eff}	Cyclic Shear Strain, g_{eff} (%)	⁽²⁾ Volumetric Strain, $e_{c,M=7.5}$ (%)	⁽³⁾ Volumetric Strain Ratio ($e_{c,M}/e_{c,M=7.5}$)	Volumetric Strain, $e_{c,M}$ (%)	Multi Direction Vol. Strain (%)	Settlement (in)
3	4	SC	3	0.5	123	184.5	2	38	0.997	60.4	4.48E-05	0.06	6.50E-05	6.50E-03	1.60E-03	0.6670	0.0011	0.0021	0.0008
8	4	SC	5	1.7	136	709.0	2	40	0.987	54.5	9.00E-05	0.23	1.30E-04	1.30E-02	1.10E-03	0.6670	0.0007	0.0015	0.0009
11	4	SC	3	2.9	136	1253.0	1	14	0.978	36.1	1.36E-04	0.41	3.00E-04	3.00E-02	3.30E-02	0.6670	0.0220	0.0440	0.0158
12.5	4	SP	1.5	3.6	108	1538.0	1	14	0.973	24.1	1.71E-04	0.50	2.80E-04	2.80E-02	1.80E-02	0.6670	0.0120	0.0240	0.0043
17.5	4	SC	5	4.6	136	1959.0	2	56	0.965	62.0	1.40E-04	0.64	3.00E-04	3.00E-02	1.80E-02	0.6670	0.0120	0.0240	0.0144
22.5	4	ML	5	6.1	101	2551.5	1	15	0.953	32.9	1.95E-04	0.83	2.40E-04	2.40E-02	1.00E-03	0.6670	0.0007	0.0013	0.0008
27.5	4	SC	5	7.6	138	3149.0	2	53	0.942	52.3	1.83E-04	1.02	3.80E-04	3.80E-02	2.10E-02	0.6670	0.0140	0.0280	0.0168
30.5	4	SC	3	8.8	138	3701.0	1	53	0.932	88.9	1.65E-04	1.20	3.00E-04	3.00E-02	2.00E-03	0.6670	0.0013	0.0027	0.0010
32.5	4	ML	2	9.6	101	4009.0	1	17	0.918	30.9	2.40E-04	1.30	2.60E-04	2.60E-02	1.00E-03	0.6670	0.0007	0.0013	0.0003
37.5	4	ML	5	10.7	101	4362.5	2	58	0.889	50.5	2.06E-04	1.42	3.20E-04	3.20E-02	9.00E-03	0.6670	0.0060	0.0120	0.0072
40.5	4	ML	3	11.9	101	4766.5	2	58	0.857	48.0	2.11E-04	1.55	3.20E-04	3.20E-02	9.00E-03	0.6670	0.0060	0.0120	0.0043
42.5	4	SM	2	12.6	101	5019.0	1	52	0.836	73.0	1.84E-04	1.63	2.30E-04	2.30E-02	1.00E-03	0.6670	0.0007	0.0013	0.0003
46	4	SC	3.5	13.5	86	5270.5	2	53	0.814	41.3	2.22E-04	1.71	1.00E+00	1.00E+02	1.00E+00	0.6670	0.6676	1.3353	0.5608
47.5	4	SC	1.5	14.2	86	5485.5	2	53	0.794	40.4	2.22E-04	1.78	2.00E+00	2.00E+02	2.00E+00	0.6670	1.3346	2.6692	0.4805
50	4	SC	2.5	14.9	86	5657.5	2	53	0.777	28.9	2.47E-04	1.84	3.00E+00	3.00E+02	3.00E+00	0.6670	2.0016	4.0031	1.2009
51.5	4	ML	1.5	15.5	101	5840.8	1	54	0.761	50.6	2.04E-04	1.90	4.00E+00	4.00E+02	4.00E+00	0.6670	2.6685	5.3371	0.9607
Total Settlement																			0.07