TECHNICON ENGINEERING SERVICES, INC.

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GEOTECHNICAL & ENVIRONMENTAL ENGINEERING — CONSTRUCTION TESTING & INSPECTION

May 17, 2022 TES No. 220239.001

Tulare Joint Union High School District
426 N. Blackstone
Tulare, CA 93274

c/o Mr. Chris Hale
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Project: Proposed Aquatics Complex and CTE Building
Mission Oak High School
3442 E. Bardsley Avenue
Tulare, California

Subject: Geotechnical Investigation and Geologic-Seismic Hazards Evaluation Report

Dear Mr. Hale:

The enclosed report presents the results of a geotechnical investigation and geologic-seismic hazards evaluation for the proposed Pool Aquatics Complex and CTE Building at the Mission Oak High School in Tulare, 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 Tulare Joint Union High School 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,

TECHNICON Engineering Services, Inc.

Salvador Alvarez, PE
Geotechnical Engineering Manager

YA:SA:vm

TECHNICON
ENGINEERING SERVICES, INC.

GEOTECHNICAL INVESTIGATION AND GEOLOGIC-SEISMIC HAZARDS EVALUATION REPORT PROPOSED AQUATICS COMPLEX AND CTE BUILDING MISSION OAK HIGH SCHOOL 3442 E. BARDSLEY AVENUE TULARE, CALIFORNIA

Prepared for:

Tulare Joint Union High School District
426 N. Blackstone
Tulare, CA 93274

May 17, 2022

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PROPOSED AQUATICS COMPLEX AND CTE BUILDING
MISSION OAK HIGH SCHOOL
3442 E. BARDSLEY AVENUE
TULARE, CALIFORNIA

TECHNICON PROJECT
TES No. 220239.001

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May 17, 2022

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3442 E. BARDSLEY AVENUE
TULARE, CALIFORNIA

1 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation for the proposed aquatics complex and CTE buildings to be constructed within the existing Mission Oak High School campus at 3442 E. Bardsley Avenue in Tulare, 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 location of the project and the Site Map, presented on Figure 2, shows the location of 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 southern Tulare County. The project location is in the south part of the Mission Oak High School campus located in Tulare, California. Based on the Tulare, California 7 ½-minute quadrangle topographic map, the site lies within the southeast quarter of Section 7, R25E and T20S. The elevation of the site is approximately 290 feet above Mean Sea Level. Based on the USGS 7½-minute topographic map, the site coordinates are approximately:

Latitude:	36.1978° N
Longitude:	119.2988° W

1.3 PROPOSED CONSTRUCTION

An understanding of the project is based on a site plan by Darden Architects, the project architect. The project involves the design and construction of two (2) projects listed below:

Career Technical Education Buildings (CTE)

- Shade Structure – 10,000 square feet
- Construction Building – 5,500 square feet
- Automotive Building – 4,500 square feet

Aquatics Complex Project

- Competition Pool
- Community Pool
- Restroom/Locker Building – 2,300 square feet
- Service Building – 2,300 square feet
- Several Shade Structures
- Equipment Building – 1,550 square feet
- Covered Entrance – 1,000 square feet
- Pool Storage Canopy – 2,850 square feet
- Parking Lot

The structures are anticipated to be supported on shallow reinforced concrete foundations and concrete slab-on-grade floors. Maximum wall and column loads are estimated to be 5 kips per foot and 50 kips; respectively. Appurtenant improvements will include asphalt concrete paved parking lot, concrete flatwork, underground utilities, artificial turf areas, bus drop off area, and sport lights. Cuts and fills may be on the order of 1 to 2 feet for site access 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 approval, design, and construction. The scope of services consisted of a field exploration program, laboratory testing, design analysis, and preparation of this written report as described in **TECHNICON** proposal, dated February 8, 2022 (TES No. GP22-023). 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 2019 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;
- 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 April 4 and April 5, 2022 consisted of drilling ten (10) exploratory test borings, and a site reconnaissance by a staff engineer. The test borings were drilled with a CME 45 truck-mounted drill rig using 4-inch inside diameter solid stem auger drilling techniques. The borings extended to depths of 16.5, 21.5, 26.5 and 51.5 feet below existing ground surface (bgs). Additionally, four (4) 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 (California 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 7.6, “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 geologic map is presented on Figure 3.

3.2 AREA AND SITE GEOLOGY

The geology at the site is mapped as Quaternary Pleistocene aged basin deposits (Qb), described as older alluvium and dissected fan deposits composed of granitic sand, silt, and clay. 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, both locations of the proposed aquatics complex and the proposed CTE buildings was observed to be undeveloped land with flat bare soil. Both project sites are located on Mission Oak High School. The high school is surrounded by parking lots, baseball fields, school buildings, the stadium/track, and tennis courts. The overall site topography is relatively flat and approximately at the same elevation of the surrounding grade.

3.4 EARTH MATERIALS

The subsurface soils consist of Pleistocene aged basin sediments. The earth material encountered by the subsurface exploration consisted of sandy silt in the upper 18 feet and underlain by sandy clay, silty sand, and sandy silt extending to the maximum depth explored, 51.5 feet bgs. The granular soils generally had a relative density of medium dense and the fine-grained soils had a consistency of 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 Figure 5 through 7.

3.5 GROUNDWATER CONDITIONS

Groundwater was not encountered within the depth explored, 51.5 feet bgs. The California Department of Water Resources “Sustainable Groundwater Management Agency Data Viewer” Spring 2020, indicates the current groundwater depth in the area is approximately 125 feet bgs.

Research utilizing the California Department of Water Resources (DWR) website shows the nearest well with recorded data to be approximately 1.0 miles to the east (Well No. 20S25E17A001M). Based on the groundwater elevation data collected at this well measurements from 1925 to 1969 ranged from 14.8 feet to 60.7 feet bgs. The shallowest groundwater depth was recorded at 14.8 feet bgs in 1943. Additionally, a nearby well (Well No. 20S25E06R002M) was also reviewed and located with recorded data to be approximately 1.0 mile to the northeast. Based on the groundwater elevation data collected at this well measurements from 2011 to 2019 ranged from 119 feet to 125 feet bgs. The deepest groundwater depth was recorded at 125 feet bgs in 2019.

Considering the groundwater trends noted above, a design groundwater depth of 14.8 feet is recommended for project planning, design, and the evaluation of liquefaction and any seismically induced effects. This depth coincides with water elevations recorded in 1969.

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 based on the current estimated depth, 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 low to 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)
Kettleman Hills	1985	40	5.6
Coalinga	1983	70	6.4
Owens Valley	1872	150	6.5
Parkfield	1922	85	6.5
Great Fort Tejon	1857	150	7.9

Epicenters of significant earthquakes ($M \geq 5.5$) within the vicinity of the site are shown on Figure 8. Data for earthquakes that occurred from 1800 to 2018 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 Toppozada 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 the active and late Quaternary faults in the area with respect to the subject site are shown on Figure 9, 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 (Mw)
Great Valley	Thrust	220	7.1
Independence	Normal	65	7.2
Owens Valley	Normal	75	7.3
San Andreas	Right Lateral/ Strike Slip	65	7.9

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 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
2019 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	0.587	S_{MS}	0.781
S_1	0.229	S_{M1}	0.491
Site Coefficient, F_v	2.142*	S_{DS}	0.521
Site Coefficient, F_a	1.33	S_{D1}	0.327
T_S	0.628		

*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 0.362g and a weighted magnitude of Mw = 6.21.

4.5 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 (0.229g) 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 2019 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 84^{th}-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. 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 > 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 84^{th} 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 (0.362) in accordance with ASCE 7-16, Section 21.5. Based on this analysis, a peak ground acceleration of 0.362g is recommended for the evaluation of liquefaction. The site specific ground motion analysis is included in Appendix D.

TABLE 4.5-1
2019 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	0.587	S_{MS}	0.907
S_1	0.229	S_{M1}	0.693
Site Coefficient, F_v	2.500	S_{DS}	0.605
Site Coefficient, F_a	1.330	S_{D1}	0.462
T_S	0.733		

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 design groundwater depth of 14.8 feet bgs, ground acceleration (PGA_M) of 0.362g, and earthquake moment magnitude, Mw = 6.21

Liquefaction analysis indicates that the soils at the project site are not susceptible to liquefaction. Seismically induced settlement due to earthquake ground shaking was evaluated to be minimal. 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 result in negligible differential settlement. The anticipated differential settlement is low and is anticipated to be within the tolerance of the proposed structure and will not result in significant damage or collapse and no surface manifestation or bearing loss is anticipated. 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 (less than 0.15-inch), and mitigation measures are not warranted.

5.3.3 Landslides and Ground Failure

The Tulare County General Plan (TCGP, 2030), indicates that Tulare County is characterized as Severity Zone “Nil” and “Low” groundshaking with zero (no) declared landslides. Furthermore, foothill and mountain areas where fractured and steep slopes are present are more prone to landslide hazards. Since the project site is located on relatively flat terrain, the potential for landslides or other slope failures from earthquake-inducted ground shaking is unlikely. 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 TCGP indicates that earthquake-induced seiches are not considered a risk in Tulare County.

5.4.2 Potential for Inundation Due to Dam Failure

According to the TCGP, two major dams could cause substantial flooding in Tulare County in the event of a failure: Terminus Dam on Lake Kaweah, and Lake Success Dams. Based on the Inundation Map, Figure 10-1, included in the TCGP, the project site is located within a flood inundation zone due to dam failure. Therefore, necessary mitigation should be performed.

5.4.3 Flood Insurance Rate Maps

According to the Federal Emergency Management Agency (FEMA), the project site lies within a Zone X flood designation (Map Number 06107C1275E, dated June 16, 2009) indicating areas determined to be outside the 0.2 percent annual chance flood. The civil engineer should plan site grades accordingly.

5.5 EXPANSIVE SOILS

Two (2) Expansion Index (EI) tests were performed on soil samples collected from the near surface soils of the site. The tests indicated the near surface soils are slightly expansive as indicated by an EI of 22 and 27. These expansive soils are susceptible to volume changes associated with changes in soil moisture content. The potential for future differential movement resulting from these soils can be reduced to normally tolerable levels by following the moisture conditions and compaction recommendations presented in this report. Moisture conditions and compaction mitigation implemented during the grading should be consistent with the expansiveness determined. Careful attention must be paid to future maintenance, including site drainage and irrigation practices.

Note that the moisture content attained during grading and building pad preparation should be maintained between the completion of grading and the placement of the vapor retarder, concrete slabs, and footings. If the moisture content is not maintained between the conclusion of grading and the start of construction, the moisture content will need to be re-established.

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 exist within the upper 5 feet of near surface soils. Laboratory testing of soil samples obtained from the site indicated collapse potential upon inundation with a normal load equal to 2,000 psf (2 to 5.3 percent compression) to be approximately 2.6 inches. Based on past experience and the variability of future moisture increase, the potential settlement could be totally differential over a distance of about 15 feet. The post construction settlement below hardscape areas (i.e. sidewalks, pavements, etc.) is negligible.

It is assumed the proposed Aquatics Complex and CTE Buildings Project cannot tolerate the post construction settlement described above. Consequently, mitigating the potential effect of these soils will be necessary to support foundations. Over-excavation is the most effective means of mitigating the potential settlement due to hydrocompaction. The over-excavation should extend to a depth of at least 5 feet below existing grade (see Section 6.2.3). Where practical, the over-excavation should extend laterally to a distance of at least 5 feet beyond the perimeter of the outer lines of foundations. The exposed excavation bottom should be processed and the excavated soil be recompacted as described in Sections 6.2.4 and 6.2.5. with these recommendations, the post construction settlement would be 0.6 inches.

5.7 CORROSIVE SOILS

The corrosion characteristics of the near surface foundation soils and any necessary mitigation measures are discussed in Section 7.6, “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 TCGP does not identify subsidence within Tulare County; however, TCGP acknowledges soils particularly subject to subsidence include those with high silt or clay content. 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 Aquatics and CTE Buildings as currently envisioned. The use of spread and continuous reinforced concrete footings bearing on undisturbed native soil or approved engineered fill are considered appropriate for structure support provided that the recommendations presented in this report are incorporated into the project design and construction.

The investigation has revealed that a surface horizon of mildly expansive sandy silt soils. These expansive soils are susceptible to volume changes associated with changes in soil moisture content. The potential for future differential movement resulting from these soils can be reduced to normally tolerable levels by following the foundation and moisture conditioning and compaction recommendations presented in this report.

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

The impacts of hydrocompactive soil (see Section 5.6) could lead to non-uniform bearing conditions and differential settlement of the proposed Aquatics Complex and CTE Buildings Project and therefore, mitigation by over-excavation and recompaction is recommended.

After performing the removals described in Sections 6.2.1 and 6.2.2, the project area and other site improvements that may be sensitive to settlement should be over-excavated a minimum depth of 5 feet below the existing site grade. The bottom of the excavation should be processed in accordance with Section 6.2.4 and the scarified soil should be recompacted to at least 90 percent relative compaction.

Over-excavation is not required below non-critical improvements, such as pavement and landscaped areas.

6.2.4 Scarification and Compaction

After stripping the site and performing the over-excavation and any required removals, all areas to receive fill or to support structures, or concrete flatwork should be scarified at least 8 inches below exposed subgrade elevation. The subgrade soil should be uniformly moisture conditioned, proof rolled to detect soft or pliant areas, and compacted to the requirements for engineered fill, as indicated in Table 6.3-2. Soft or pliant areas should be mitigated in accordance with Section 6.2.2.

The expansive soil conditions will necessitate moisture conditioning to a depth of 6 inches below footings and 18 inches below slabs (refer to Sections 7.2 and 7.4). Therefore, additional over excavation and scarification may be necessary to achieve the required moisture content below footings and slabs-on-grade.

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

Gradation (ASTM C136)			
Sieve Size		**Percent Passing**	
76 mm (3-inch)		100	
19 mm (¾-inch)		80 – 100	
No. 4		60 – 100	
No. 200		20 – 50	
Expansion Index (ASTM D4829)		**Plasticity (ASTM D4318)**	
		Liquid Limit	**Plasticity Index**
< 20		< 25	< 9
Organic Content (ASTM D 2974)			
< 3% by dry weight			
Corrosivity			
pH	**Minimum Resistivity (ohm-cm)**	**Soluble Sulfate (ppm)**	**Soluble Chloride (ppm)**
6 to 8	> 2,000	< 2,000	< 500
Resistance Value			
R-value > 11			

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 the percentages above optimum moisture indicated in Table 6.3-2, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to within the required range of relative compaction indicated in Table 6.3-2. Discing and/or blending may be required to uniformly moisture-condition soils used for engineered fill. The actual level of moisture conditioning and compaction will be based on the expansion potential and moisture density relationships determined during grading. The general intent is to bring the expansive material to about 80 to 85 percent saturation at the time of construction. Preliminary design with use of on-site soil should consider criteria (bold values) for the EI range of 21 - 50 (PI 16 - 25).

TABLE 6.3-2
MOISTURE CONDITIONING AND COMPACTION

Soils		Relative Compaction (min – max)	Minimum Moisture Conditioning (% Over Optimum)
PI	EI		
< 9	< 20	90%	+ 0%
9 - 15	**21 - 50**	**90-95%**	**+ 3%**
16 - 25	51-90	88-92%	+ 4%
> 25	> 90	88-92%	+ 5%

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 (Sandy Silt) 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 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 on-site 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 Aquatics Complex and CTE Buildings may be supported by conventional shallow spread footings supported on approved undisturbed native soil or 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

Based on the expansive nature of the foundation soils, it is recommended that footings consist of continuous reinforced foundation, embedded at least 18 inches below the lowest adjacent grade. Continuous footings should be reinforced with #4 bars on center in both principal directions. Foundation depths and reinforcement should also satisfy structural and constructability considerations. Subgrade within 6 inches of the bottom of footings and within footing sidewalls should have moisture content of at least 3 percent above optimum, immediately prior to placing the footing concrete.

These recommendations are based on engineering judgment and experience associated with expansive soil and are not based on any structural analysis. Any additional reinforcement for structural considerations should be provided by the structural engineer. The recommendations should be reviewed by the project structural engineer or building designer and they should concur with the recommendations provided.

7.2.1 Allowable Vertical Bearing Pressures and Settlements

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.

The bearing capacity, based only on the shear strength of the soil, will be dependent upon the footing geometry. Table 7.2-1 presents the expressions for the 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).

TABLE 7.2-1
BEARING CAPACITY

	Bearing Capacity (psf)
Static Loading	415 B + 925 D
Total Combined Loading	325 B + 1,390 D
Unfactored Ultimate Bearing	1,245 B + 2,780 D

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

The above expressions are appropriate for design using the Basic and Alternative Load Combinations in Section 1605.3 of the 2019 CBC. To simplify design, an allowable bearing pressure of 1,500 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, Kp (Bp = 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 methods by Schmertmann, determined the following estimated static settlement based on a range of assumed design bearing and estimated structural loads. The estimated settlements presented in Table 7.2-2 are based on the assumption that the sustained load of footings is equal to 80 percent of the total load.

TABLE 7.2-2
ESTIMATED SETTLEMENT

Footing Type	**Loading (DL + LL)**	**Design Bearing (psf)**	**Estimated Settlement (inch)**
Strip	5 kips/ft	1,500	0.25
Square	50 kips	1,500	0.27

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 COEFFICIENTS

	Allowable		Ultimate
	Static	Total Combined	
Frictional Coefficient	0.30	0.40	0.60
Passive Pressure (psf/ft)	280	375	565
Lateral Translation Needed to Develop Passive Pressure	0.003 D	0.006 D	0.014 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)	40
At-Rest Pressure (psf/ft of depth)	60
Braced Pressure (psf)	26 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 sliding 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.

The project will incorporate pool walls of over 6 feet in height. Therefore, evaluation of increments to earth pressure due to seismic forces was performed according to Lew, Sitar, and SEAOC Standards. Since the maximum ground acceleration at this location is less than 0.4g, there is no seismic increment of earth 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 18 inches of pad grade should have a moisture content of at least 3 percent 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. 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

To accommodate the potential for expansive soils, the minimum reinforcement of concrete floor slabs should consist of #3 bars at 30 inches on center in both principal directions or equivalent. The reinforcement is based on engineering judgement and experience with expansive soils, not on any structural analysis. Slab thickness and reinforcement should also satisfy 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 to 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	$55\ DL^2$
Total Combined Loading	$70\ DL^2$
Unfactored Ultimate Capacity	$105\ DL^2$

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 175 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 CORROSION POTENTIAL

Soil samples obtained from the near surface of the site was tested for pH, minimum electrical resistivity, and soluble sulfate and chloride.

Provided in Table 7.6-1 are the pH, minimum electrical resistivity, and soluble sulfate and chloride content for both locations throughout the project.

TABLE 7.6-1
CORROSION POTENTIAL

Boring	Depth (ft)	pH	Minimum Resistivity (ohm-cm)	Soluble Sulfate (ppm)	Soluble Chloride (ppm)
B-2	0 to 5	7.97	735	0.4	6.5
B-10	0 to 5	7.98	1,172	0.4	1.8

The following sections provide brief descriptions of the corrosion characteristics of the soil to buried metal and concrete based on the soil testing and general knowledge of corrosion. 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 designer desires more specific recommendations.

7.6.1 General Corrosion – Ferrous Metals

The test results and corrosion calculations indicate a mild corrosion potential at the school site. An example of the range of corrosion characteristics of the on-site soils to buried unprotected ferrous metal was estimated utilizing methods provided in Caltrans California Test 643, “Method for Estimating the Service Life of Steel Culverts”. The calculation is based on an 18-gauge steel zinc-coated culvert, which is estimated to have a maintenance-free service life (years to perforation) ranging from 22 to 26 years. The calculation is dependent on pH and minimum resistivity of the soil and thus, a range of service lives was determined due to the ranging test results.

7.6.2 Sulfate Attack

Test results suggest that low levels of soluble sulfates are present in on-site soils. Consequently, with respect to sulfate content, normal cement (Type II) should be adequate in foundation concrete.

7.6.3 Chloride Attack

Test results suggest that low levels of soluble chlorides are present in on-site soils. Reinforcement cover need not be increased for concrete that comes in contact with the on-site soil at the project sites.

8 PAVEMENT DESIGN

8.1.1 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 four (4) locations within proposed pavement areas. The tested soil had measured R-values of 31, 11, 28, and 11. The laboratory testing conformed to Caltrans Test Method 301. Based on the variability of the R-value test results an R-value of 11 is recommended for preliminary pavement design. Additional R-values could be collected and tested after rough grading and pavement design recommendations may be revised if appropriate.

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 occasional delivery truck traffic and trash collection traffic. Consequently, a range of pavement sections have been provided based on Traffic Indexes (T.I.'s) of 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5 and 8.0. These traffic design assumptions should be reviewed for compatibility with the actual development, and revised pavement sections developed, as necessary.

8.1.2 Asphalt Concrete Pavement Design

Flexible pavement design recommendations have been developed fort the given T.I.’s based upon the California Department of Transportation (Caltrans) design procedures and a design R-value of 11. The flexible asphalt concrete pavement sections associated with the assumed T.I.’s for on-site asphalt pavements are summarized in Table 8.1-1.

TABLE 8.1-1
RECOMMENDED MINIMUM PAVEMENT SECTIONS

Traffic Index	Asphalt Concrete (inches)	Aggregate Base – Class 2 (inches)
4.5	2.5	8.0
5.0	2.5	10.0
5.5	3.0	10.5
6.0	3.0	12.5
6.5	3.5	13.0
7.0	4.0	14.0
7.5	4.0	16.0
8.0	4.5	16.5

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 8.1-2.

TABLE 8.1-2
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
5.5	11.6		4.3		1.1
6.0	24.1		9.0		2.4
6.5	47.3		17.7		4.7
7.0	88.1		33.0		8.8
7.5	157.3		59.0		15.8
8.0	270.6		101.5		27.1

The flexible pavement should conform to and be placed in accordance with the Caltrans Standard Specifications. 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.

8.2 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.

9 ADDITIONAL SERVICES

9.1 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.

9.2 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that a representative of **TECHNICON** observe the excavation, earthwork, foundation, and pavement 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.

10 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 Tulare Joint Union High School District and their designated consultants for the proposed Aquatics Complex and CTE Buildings to be located at Mission Oak High School, 3442 E. Bardsley Avenue in Tulare, 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.

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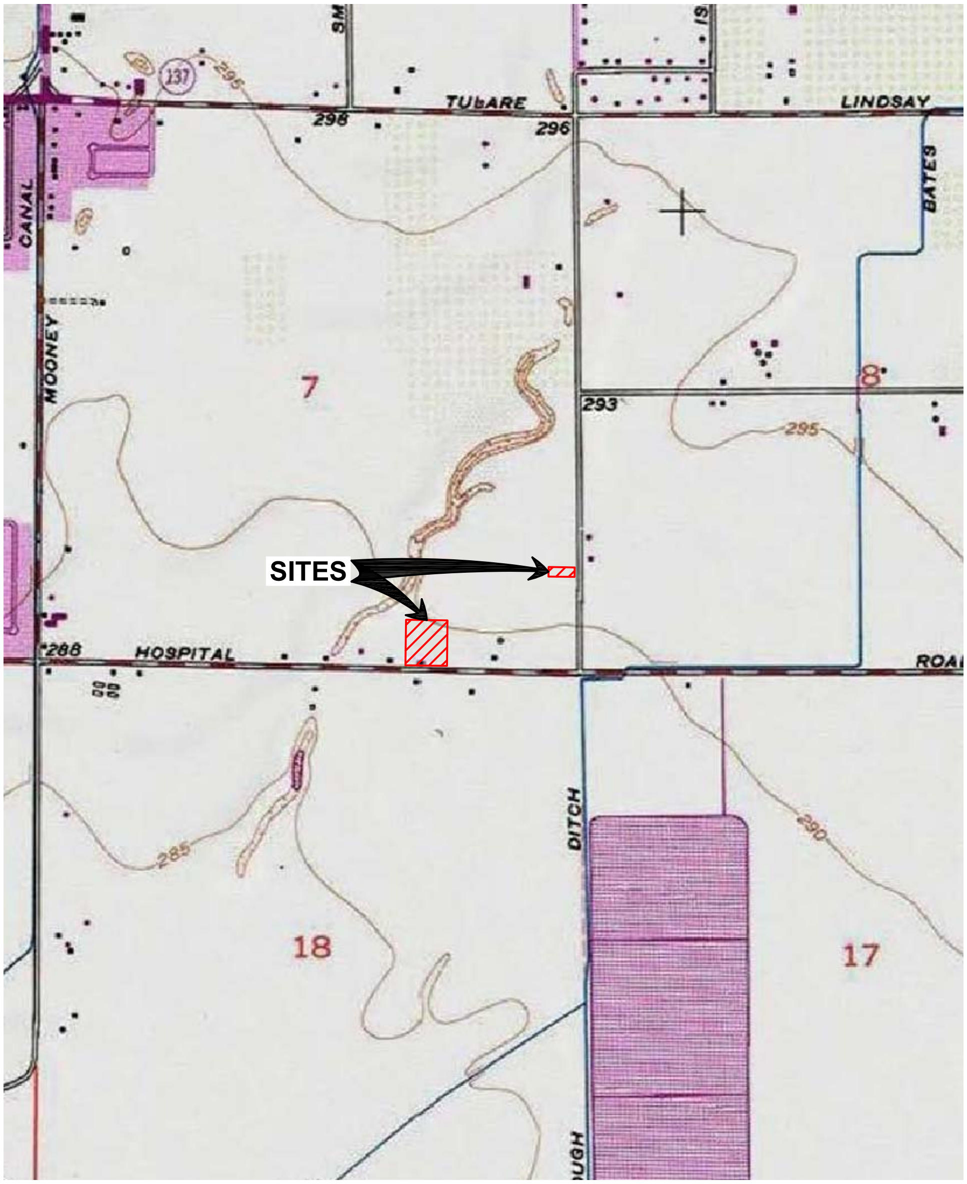
- USGS, Earthquake Hazards Program, California Earthquake History (1800 – present), http://earthquake.usgs.gov
- Youd, T.L., et al. (2001) Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils: Journal of Geotechnical and Environmental Engineering , Vol. 127, No. 10, October 2000

FIGURES

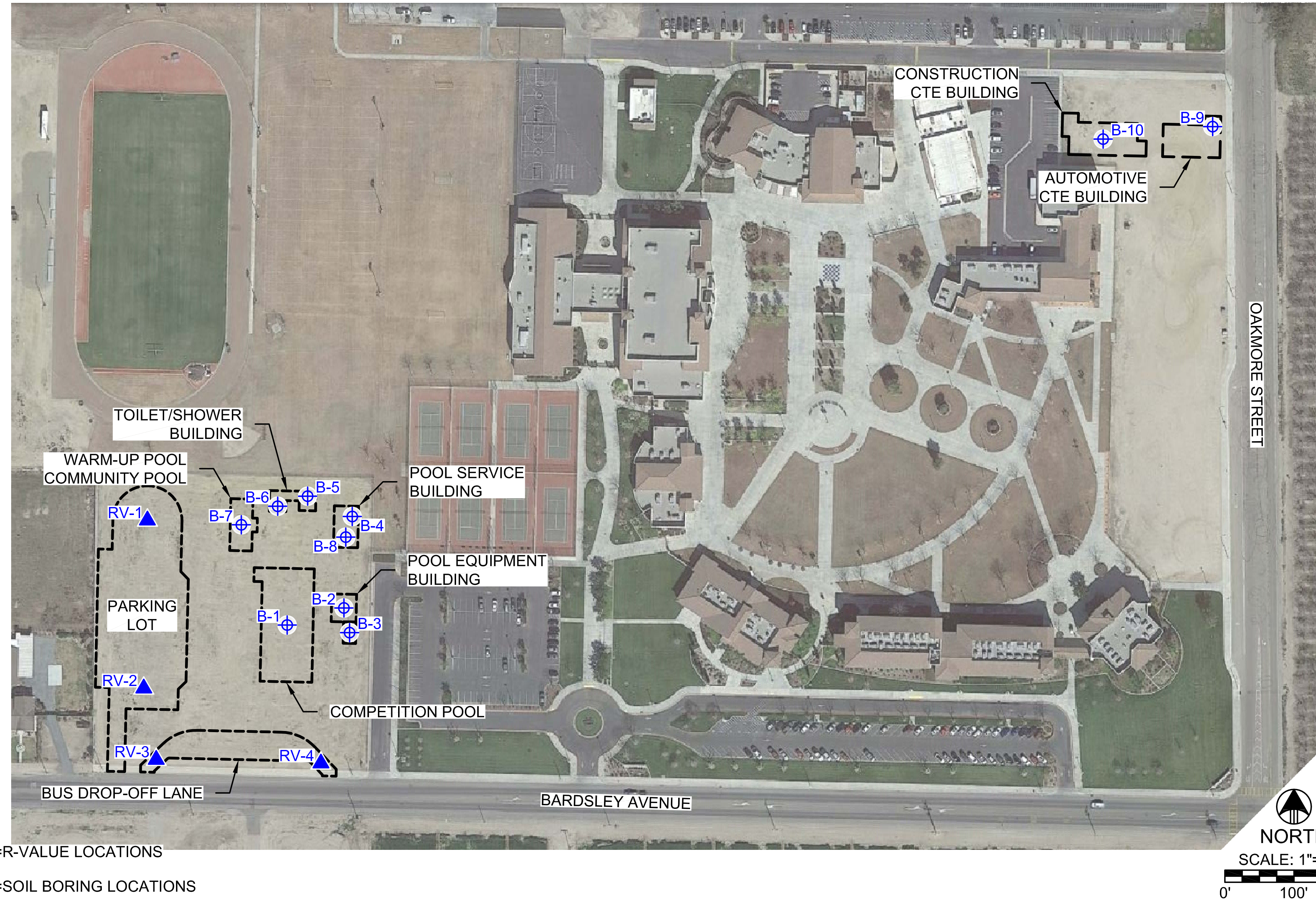
1 through 9



LAT.: 36.19789°N, LONG.: 119.29886°W, 7-T20S-R25E, MDB&M

NORTH

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 220239	VICINITY MAP PROPOSED AQUATICS COMPLEX AND CTE BUILDING - MISSION OAK HIGH SCHOOL 3442 E. BARDSLEY AVENUE TULARE, CALIFORNIA	FIGURE 1
	SOURCE: USGS TOPOGRAPHIC MAPS		NTS

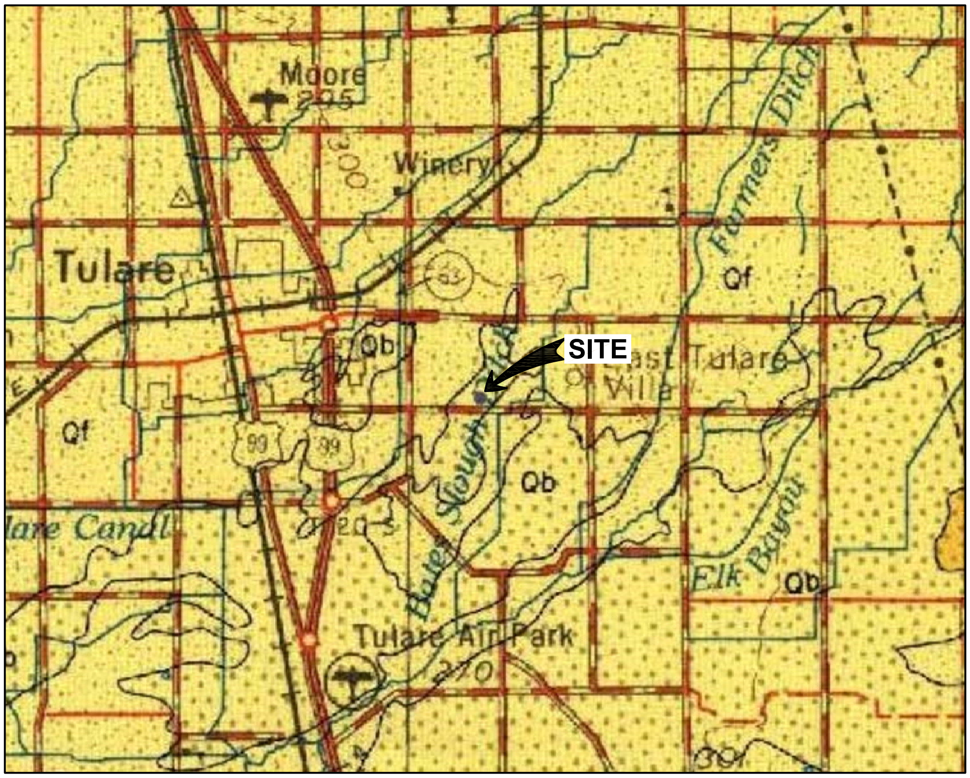


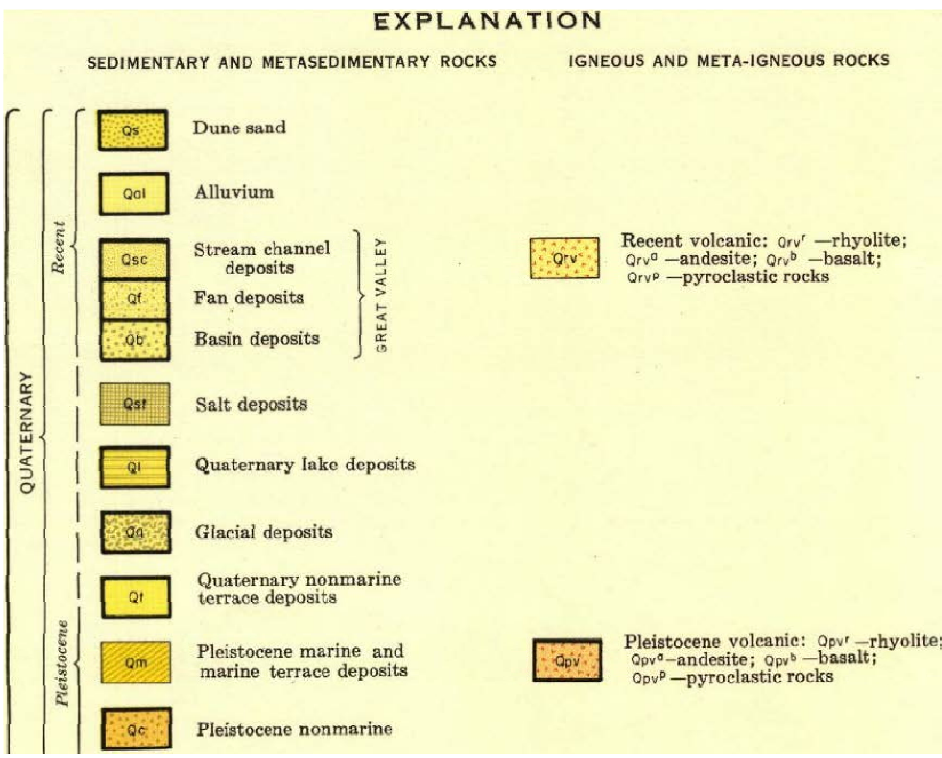
=R-VALUE LOCATIONS

=SOIL BORING LOCATIONS

TECHNICON ENGINEERING SERVICES, INC.

PROJECT: 220239	DATE: 4/26/22	SITE MAP PROPOSED AQUATICS COMPLEX AND CTE BUILDING MISSION OAK HIGH SCHOOL 3442 E. BARDSLEY AVENUE TULARE, CALIFORNIA	FIGURE 2
SOURCE: GOOGLE EARTH	APPROVED BY: YA		

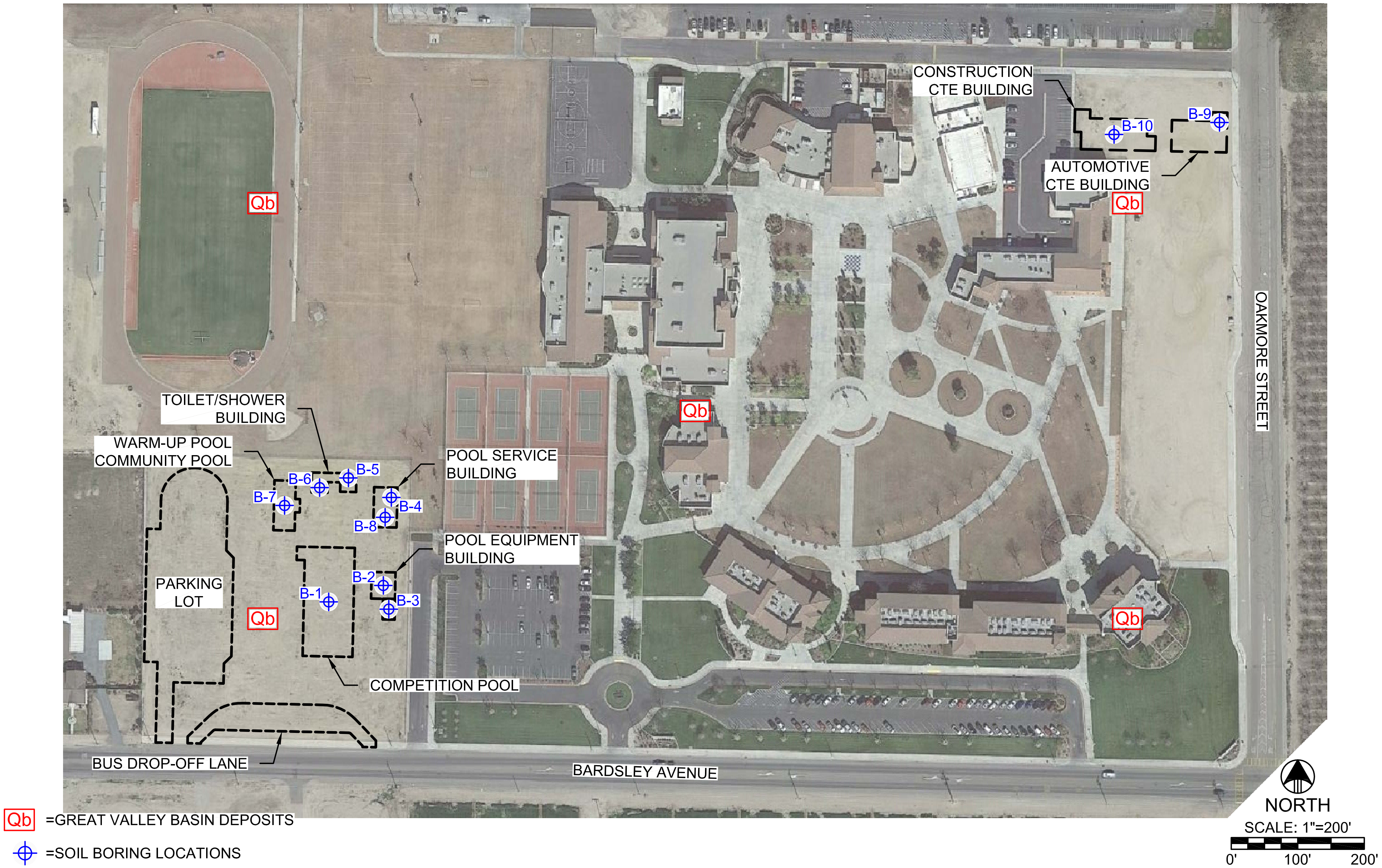




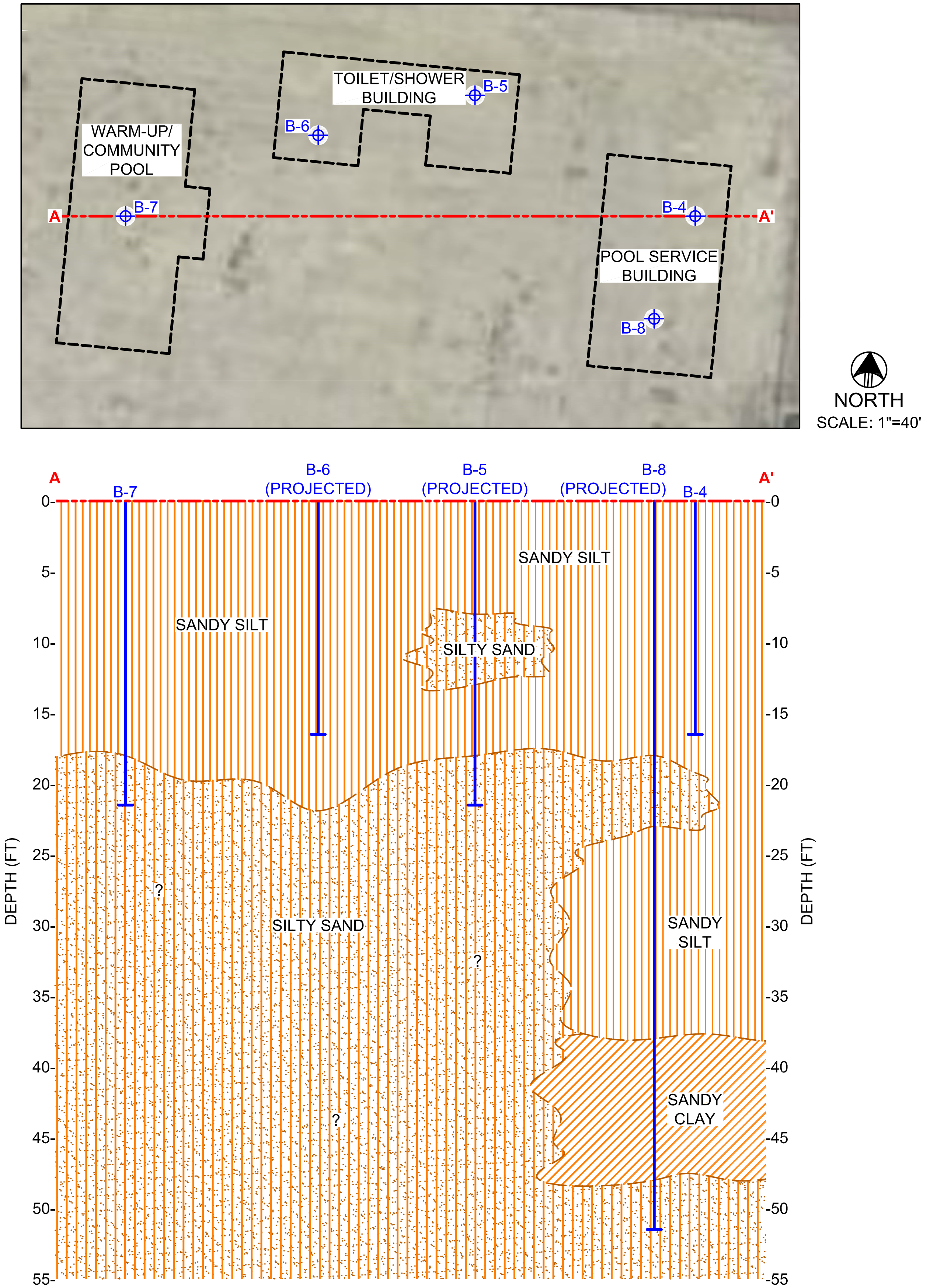
GEOLOGIC MAP OF CALIFORNIA : FRESNO SHEET, SCALE 1:250,000 - 1965

NORTH

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 220239 SOURCE: DIVISION OF MINES AND GEOLOGY	REGIONAL GEOLOGIC MAP PROPOSED AQUATICS COMPLEX AND CTE BUILDING - MISSION OAK HIGH SCHOOL 3442 E. BARDSLEY AVENUE TULARE, CALIFORNIA	FIGURE 3 NTS



TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 220239	DATE: 4/26/22	GEOLOGIC MAP OF SITE PROPOSED AQUATICS COMPLEX AND CTE BUILDING MISSION OAK HIGH SCHOOL 3442 E. BARDSLEY AVENUE TULARE, CALIFORNIA	FIGURE 4
	SOURCE: GOOGLE EARTH	APPROVED BY: YA		



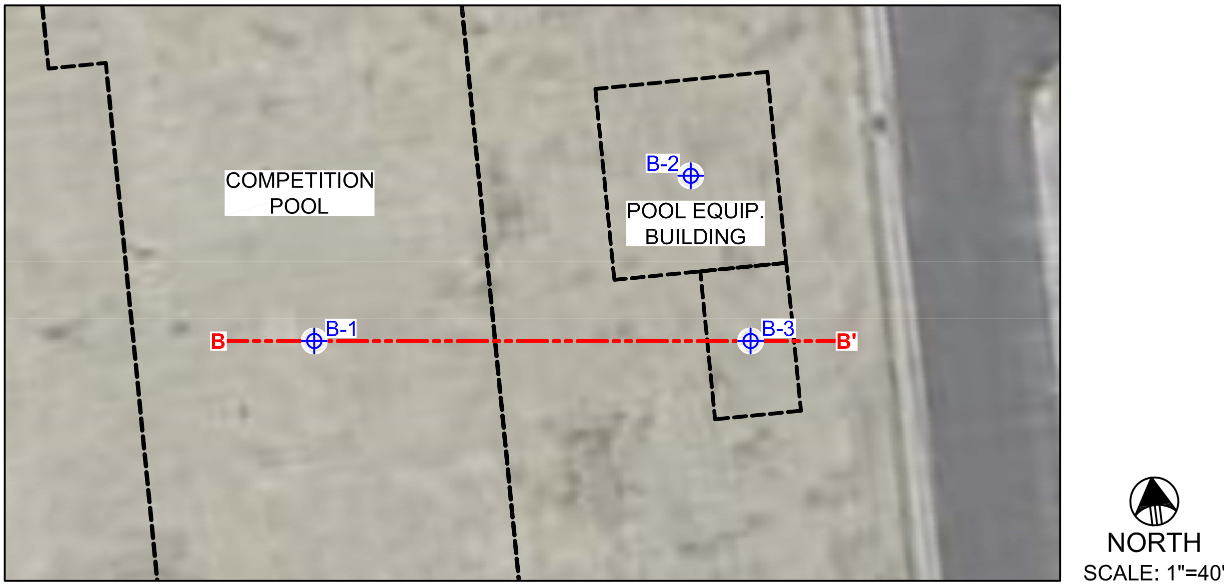
TECHNICON ENGINEERING SERVICES, INC.

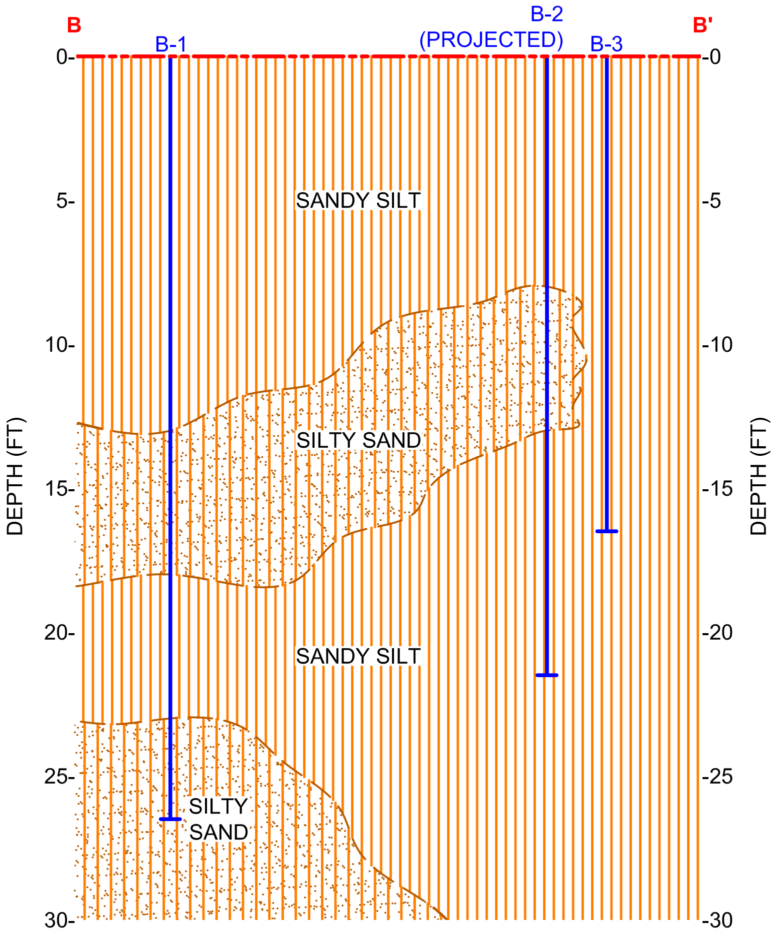
PROJECT: 220239

CAD BY: MH

CROSS SECTION A-A'
PROPOSED AQUATICS COMPLEX AND CTE BUILDING - MISSION OAK HIGH SCHOOL
3442 E. BARDSLEY AVENUE
TULARE, CALIFORNIA

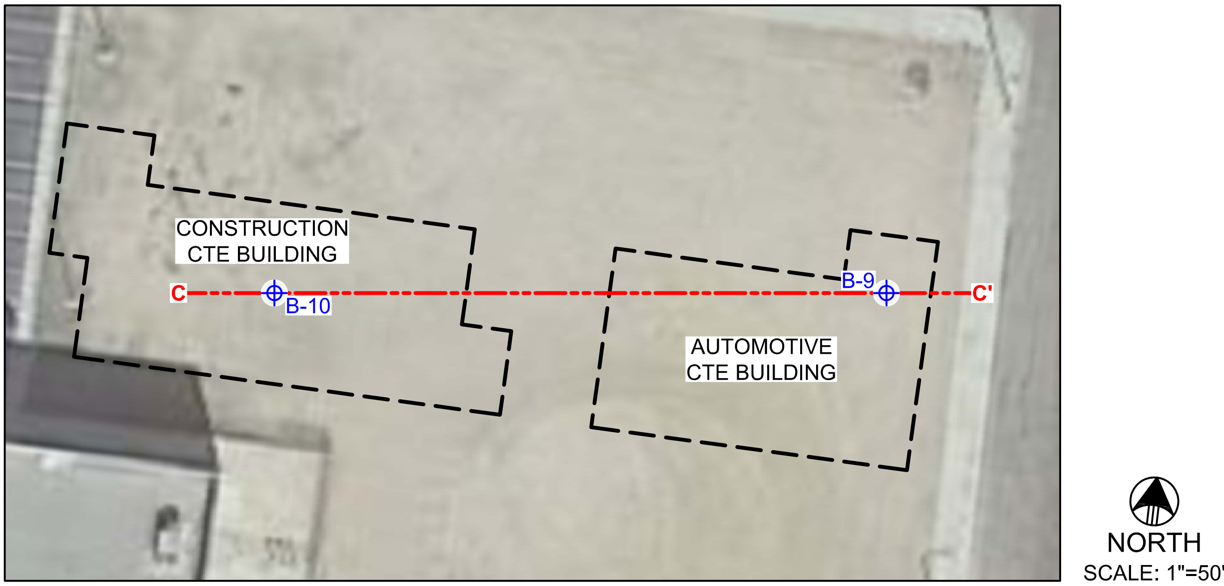
FIGURE 5

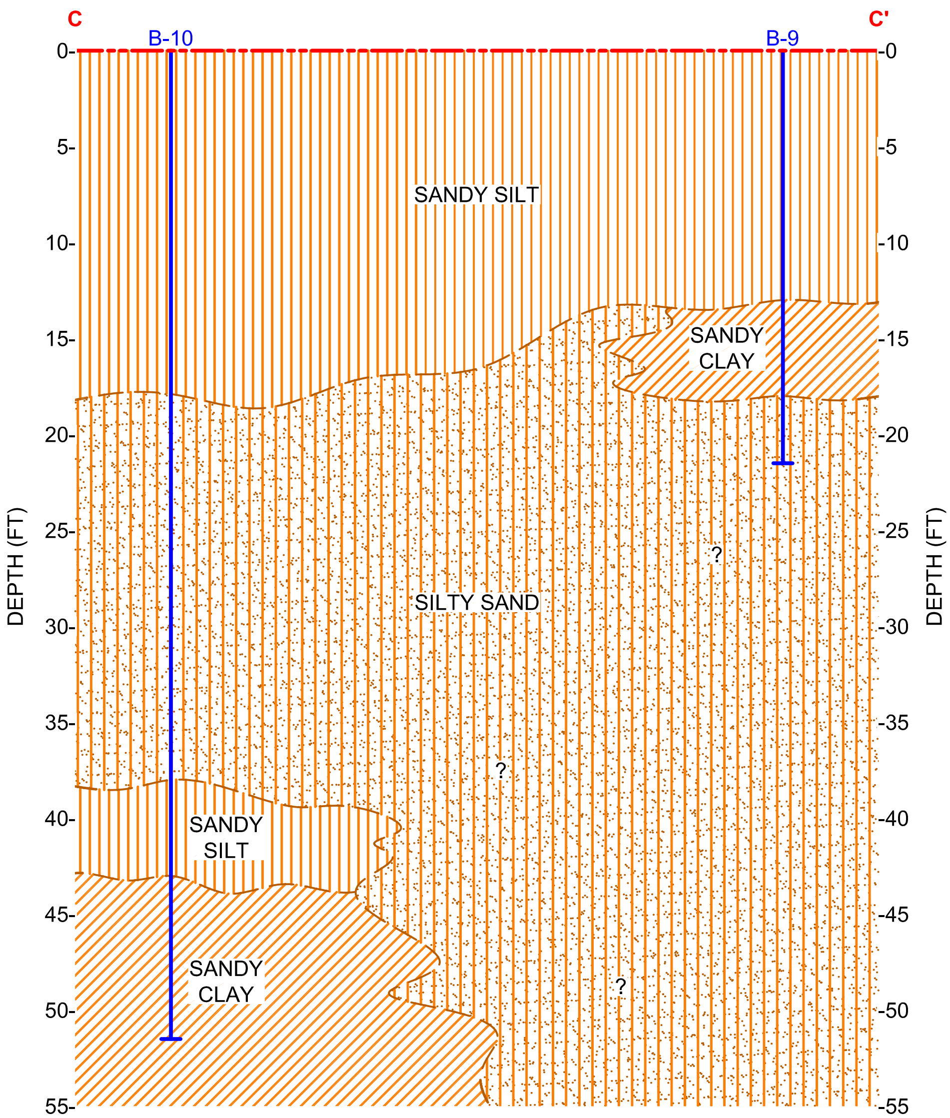




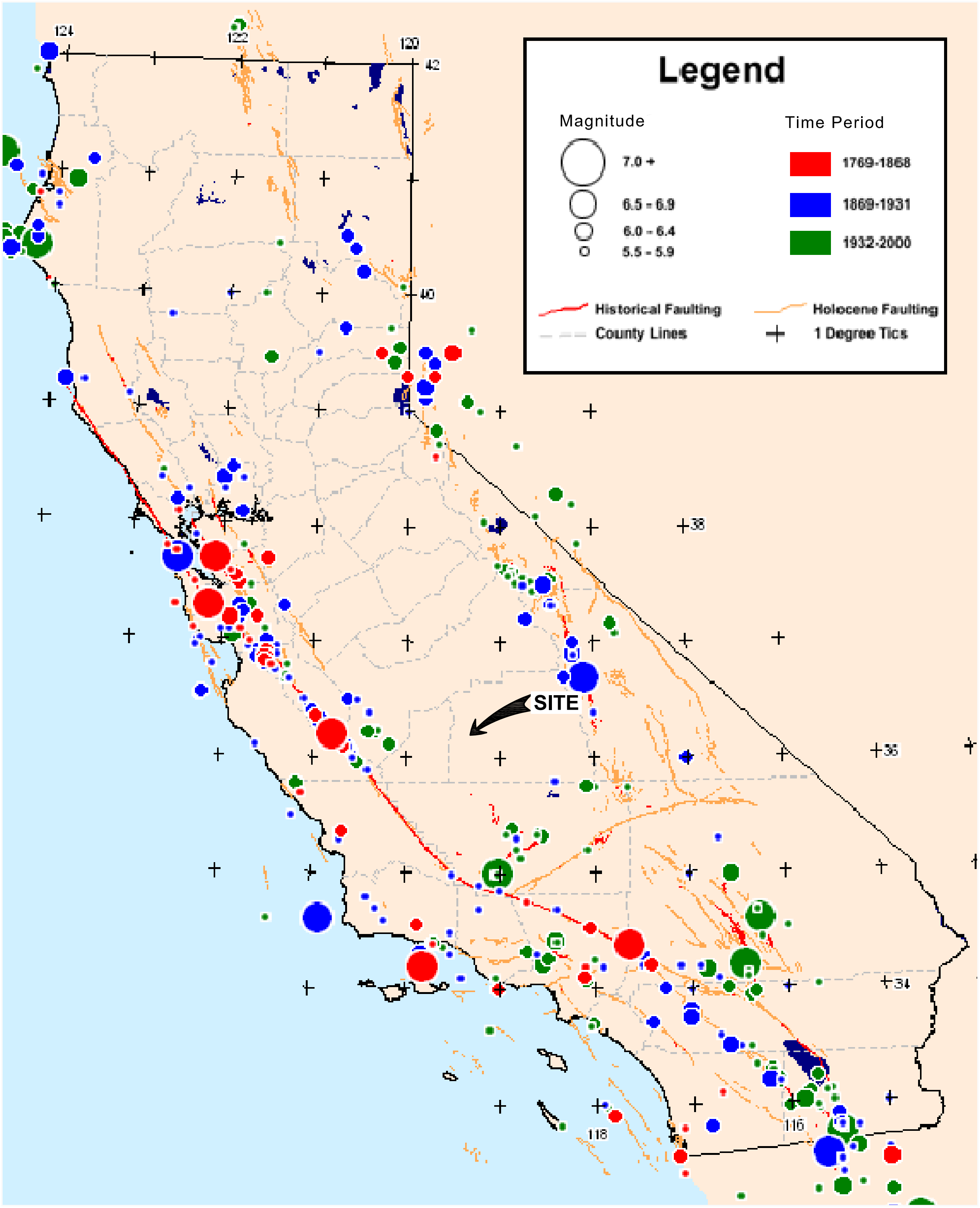
TECHNICON ENGINEERING SERVICES, INC.

PROJECT: 220239	CROSS SECTION B-B' PROPOSED AQUATICS COMPLEX AND CTE BUILDING - MISSION OAK HIGH SCHOOL 3442 E. BARDSLEY AVENUE TULARE, CALIFORNIA	FIGURE	6
CAD BY: MH			

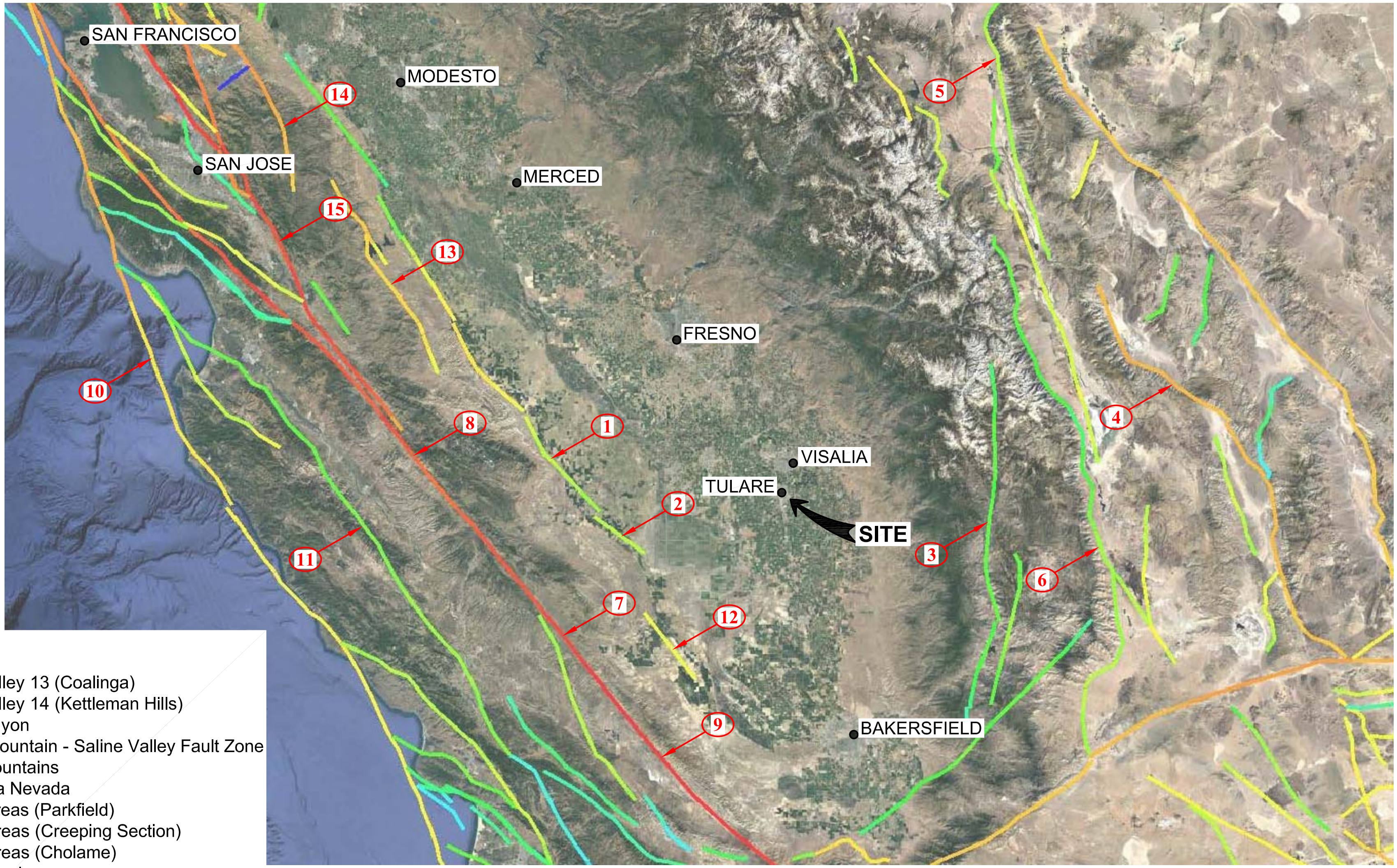




TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 220239	CROSS SECTION C-C' PROPOSED AQUATICS COMPLEX AND CTE BUILDING - MISSION OAK HIGH SCHOOL 3442 E. BARDSLEY AVENUE TULARE, CALIFORNIA	FIGURE 7
	CAD BY: MH		



TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 220239	EPICENTER MAP	FIGURE
	SOURCE: CGS	PROPOSED AQUATICS COMPLEX AND CTE BUILDING - MISSION OAK HIGH SCHOOL 3442 E. BARDSLEY AVENUE TULARE, CALIFORNIA	8 NTS



FAULTS

1. Great Valley 13 (Coalinga)
2. Great Valley 14 (Kettleman Hills)
3. Kern Canyon
4. Hunter Mountain - Saline Valley Fault Zone
5. White Mountains
6. SO Sierra Nevada
7. San Andreas (Parkfield)
8. San Andreas (Creeping Section)
9. San Andreas (Cholame)
10. San Gregorio
11. Reliz
12. Lost Hills
13. Ortigalita
14. Greenville
15. Calaveras

NORTH

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 220239	DATE: 4/26/22	REGIONAL FAULT ACTIVITY MAP PROPOSED AQUATICS COMPLEX AND CTE BUILDING MISSION OAK HIGH SCHOOL 3442 E. BARDSLEY AVENUE TULARE, CALIFORNIA	FIGURE 9
	SOURCE: WGCEP	APPROVED BY: YA		NTS

BORING LOGS AND LOG KEY

APPENDIX A

TECHNICON Engineering Services Inc
4539 N Brawley
Fresno CA 93722
Telephone: 5592769344

KEY TO SYMBOLS

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
DATE OF EXPLORATION 4/4/2022
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
PROJECT NUMBER TES No. 220239

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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BORING B-01

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/4/22 **COMPLETED** 4/4/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 26.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Sandy SILT (ML)** - stiff, brown, moist, with fine sand				
	CAL	3-4-10 (14)			103.2	6.8	S = 30 %	
5	SPT	3-10-13 (23)		Very stiff				
10	CAL	8-10-15 (25)			67.6	76.5	S = 140 %	
				Silty SAND (SM) - medium dense, brown, moist, fine to medium grained				
15	SPT	8-9-13 (22)						
				Sandy SILT (ML) - very stiff, brown, moist, with fine sand, trace clay				
20	CAL	6-12-15 (27)			110.0	17.5	S = 92 %	
				Silty SAND (SM) - medium dense, brown, moist, fine to medium grained				
25	SPT	8-9-11 (20)						

NOTES:
1. Bottom of boring at 26.5 feet.
2. No groundwater encountered.
3. Boring backfilled with .

TECHNICON Engineering Services Inc
4539 N Brawley
Fresno CA 93722
Telephone: 5592769344

BORING B-02

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/4/22 **COMPLETED** 4/4/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 21.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Sandy SILT (ML)** - very stiff, brown, moist, with fine sand				
	CAL	7-8-8 (16)			114.8	7.3	S = 44 %	
5	SPT	8-9-13 (22)						
				Silty SAND (SM) - medium dense, brown, moist, fine to medium grained				
10	CAL	6-6-12 (18)			96.7	5.7	S = 21 %	
				Sandy SILT (ML) - very stiff, brown, moist, with fine sand				
15	SPT	6-6-12 (18)						
20	CAL	10-13-14 (27)			116.7	15.5	S = 99 %	

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

TECHNICON Engineering Services Inc
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Fresno CA 93722
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BORING B-03

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/4/22 **COMPLETED** 4/4/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 16.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Sandy SILT (ML)** - stiff, brown, moist, with fine sand, moderate cementation				
	CAL	7-9-13 (22)			112.4	8.6	S = 48 %	
5	SPT	7-10-13 (23)		Very stiff				
10	CAL	9-15-25 (40)			119.7	5.8	S = 40 %	
15	SPT	3-5-6 (11)		Stiff				

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

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Fresno CA 93722
Telephone: 5592769344

BORING B-04

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/4/22 **COMPLETED** 4/4/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 16.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Sandy SILT (ML)** - stiff, brown, moist, with fine sand				
	CAL	4-8-12 (20)			115.7	11.1	S = 68 %	
5	SPT	6-8-6 (14)						
10	CAL	6-10-13 (23)			110.6	4.1	S = 22 %	
15	SPT	14-30-34 (64)		Hard, strong cementation				

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

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Fresno CA 93722
Telephone: 5592769344

BORING B-05

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/4/22 **COMPLETED** 4/4/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 21.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Sandy SILT (ML)** - stiff, brown, moist, with fine sand				
	CAL	5-8-9 (17)			110.4	9.9	S = 52 %	
5	CAL	8-13-38 (51)			91.2	16.0	S = 52 %	
				Silty SAND (SM) - medium dense, brown, moist, fine grained				
10	SPT	8-6-17 (23)						
				Sandy SILT (ML) - very stiff, brown, moist				
15	CAL	9-16-19 (35)			132.9	18.9	S = 205 %	
				Silty SAND (SM) - medium dense, brown, moist, fine grained				
20	SPT	5-5-9 (14)						

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

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BORING B-06

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/4/22 **COMPLETED** 4/4/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 16.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Sandy SILT (ML)** - very stiff, brown, moist, with fine sand				
	CAL	8-12-14 (26)			100.5	18.5	S = 76 %	
5	SPT	6-12-15 (27)						
10	CAL	7-12-17 (29)			100.4	8.0	S = 33 %	
15	CAL	15-34-50 (84)		Hard	110.2	19.1	S = 101 %	

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

TECHNICON ENGINEERING SERVICES, INC.
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Fresno CA 93722
Telephone: 5592769344

BORING B-07

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/4/22 **COMPLETED** 4/4/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 21.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Sandy SILT (ML)** - stiff, brown, moist, with fine sand				
	CAL	5-10-12 (22)			99.1	12.6	S = 50 %	
5	CAL	7-17-17 (34)		Very stiff	101.8	18.6	S = 79 %	
10	SPT	6-8-9 (17)						
15	CAL	14-15-18 (33)		Hard	113.5	18.0	S = 104 %	
				Silty SAND (SM) - medium dense, brown, moist, fine grained				
20	SPT	5-7-9 (16)						

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

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BORING B-08

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/4/22 **COMPLETED** 4/4/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 51.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Sandy SILT (ML)** - stiff, brown, moist, with fine sand				
	CAL	4-7-7 (14)			107.4	9.0	S = 44 %	
5	SPT	5-8-10 (18)						
10	CAL	6-12-14 (26)		Very stiff	96.6	12.9	S = 48 %	
15	SPT	14-16-10 (26)						
				Silty SAND (SM) - medium dense, brown, moist, fine grained				
20	CAL	7-16-18 (34)			121.6	9.2	S = 68 %	
				Sandy SILT (ML) - stiff, brown, moist, with fine sand				
25	SPT	3-3-6 (9)						
30	CAL	5-12-14 (26)		Very stiff, moist	115.0	16.5	S = 100 %	
35								

(Continued Next Page)

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Fresno CA 93722
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BORING B-08

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/4/22 **COMPLETED** 4/4/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 51.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
35	SPT	3-10-12 (22)		**Sandy SILT (ML)** - stiff, brown, moist, with fine sand *(continued)*				
				Sandy CLAY (CL) - hard, brown, moist, with fine sand				
40	CAL	7-12-29 (41)			113.7	15.5	S = 91 %	
45	SPT	7-10-13 (23)		Very stiff				
				Silty SAND (SM) - medium dense, brown, moist, fine grained				
50	SPT	8-10-12 (22)						

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

TECHNICON Engineering Services Inc
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BORING B-09

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/5/22 **COMPLETED** 4/5/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 21.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Sandy SILT (ML)** - stiff, brown, moist, with fine sand				
	CAL	6-10-15 (25)			102.6	6.8	S = 29 %	
5	SPT	2-3-3 (6)		Medium stiff				
10	CAL	8-12-17 (29)		Stiff	114.3	16.0	S = 95 %	
				Sandy CLAY (CL) - stiff, brown, moist, with fine sand				
15	SPT	3-3-6 (9)						
				Silty SAND (SM) - medium dense, brown, moist, fine grained				
20	CAL	4-5-11 (16)			112.1	16.5	S = 92 %	

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

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Fresno CA 93722
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BORING B-10

PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/5/22 **COMPLETED** 4/5/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 51.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Sandy SILT (ML)** - hard, brown, moist, with fine sand				
	CAL	16-28-29 (57)			110.9	13.7	S = 74 %	
5	SPT	7-8-8 (16)		Stiff				
10	CAL	21-30-30 (60)		Hard	126.1	7.5	S = 64 %	
15	SPT	12-12-30 (42)						
				Silty SAND (SM) - medium dense, brown, moist, fine grained				
20	CAL	8-15-20 (35)		Medium dense				
25	SPT	7-5-9 (14)						
30	CAL	15-21-14 (35)			94.9	16.0	S = 57 %	
35								

(Continued Next Page)

TECHNICON Engineering Services Inc
4539 N Brawley
Fresno CA 93722
Telephone: 5592769344

BORING B-10

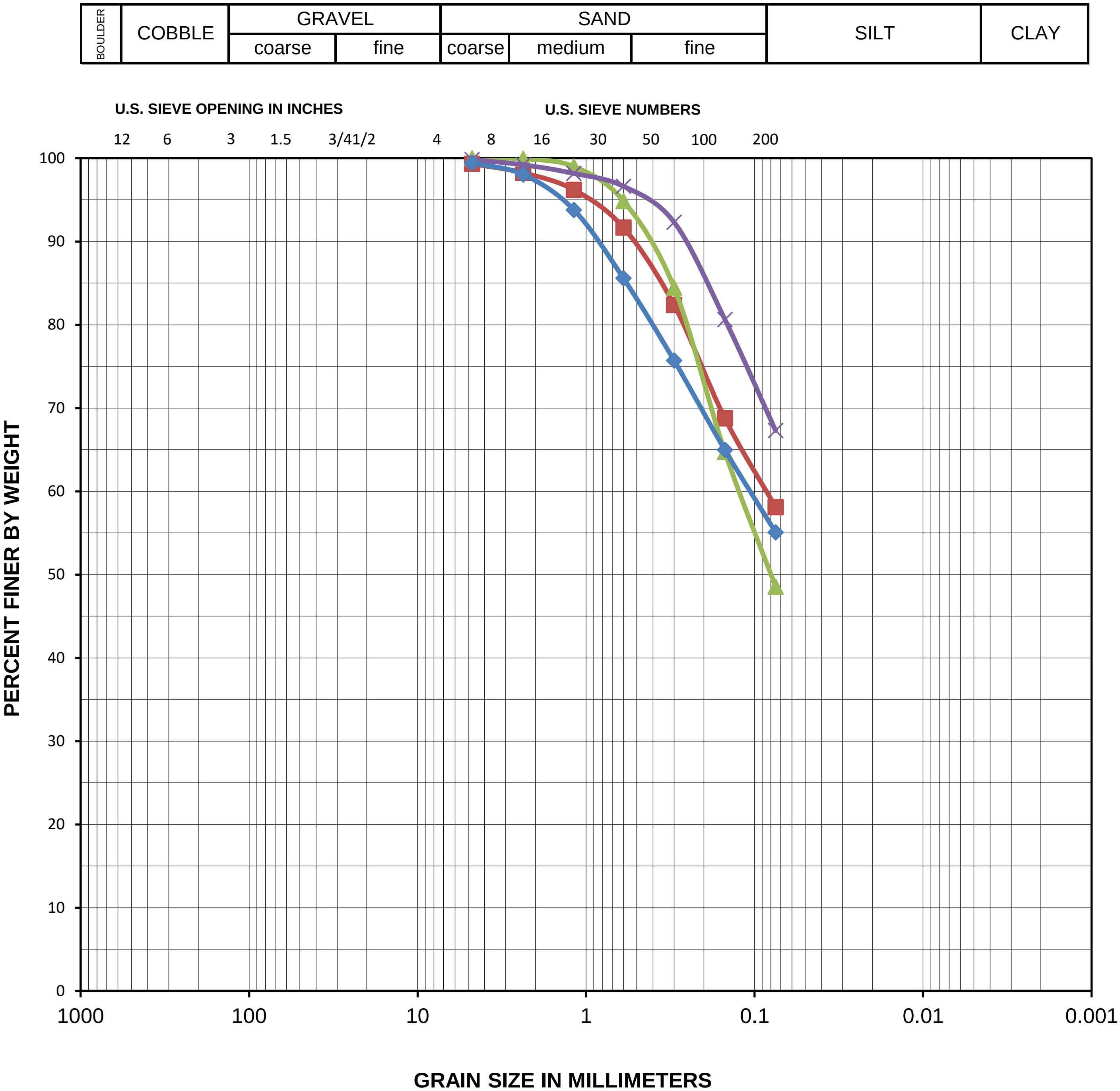
PROJECT NAME Aquatics Complex, CTE Building/Mission Oaks HS
PROJECT NUMBER TES No. 220239
PROJECT LOCATION 3442 E. Bardsley Avenue Tulare, CA
SURFACE DESCRIPTION Flat, vacant, shrub
DATE STARTED 4/5/22 **COMPLETED** 4/5/22
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE CME 45
BORING DEPTH 51.5 ft
DRILLING METHOD 4-inch Solid Flight Auger
LOGGED BY Y. Ashaq **CHECKED BY** S. Alvarez

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
35	SPT	10-11-12 (23)		**Silty SAND (SM)** - medium dense, brown, moist, fine grained *(continued)*				
				Sandy SILT (ML) - very stiff, brown, moist, with fine sand				
40	CAL	13-15-25 (40)			112.4	17.4	S = 98 %	
				Sandy CLAY (CL) - very stiff, brown, moist, with fine sand				
45	SPT	3-6-10 (16)						
50	CAL	10-15-25 (40)			118.0	14.5	S = 96 %	

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

LABORATORY TESTS

APPENDIX B



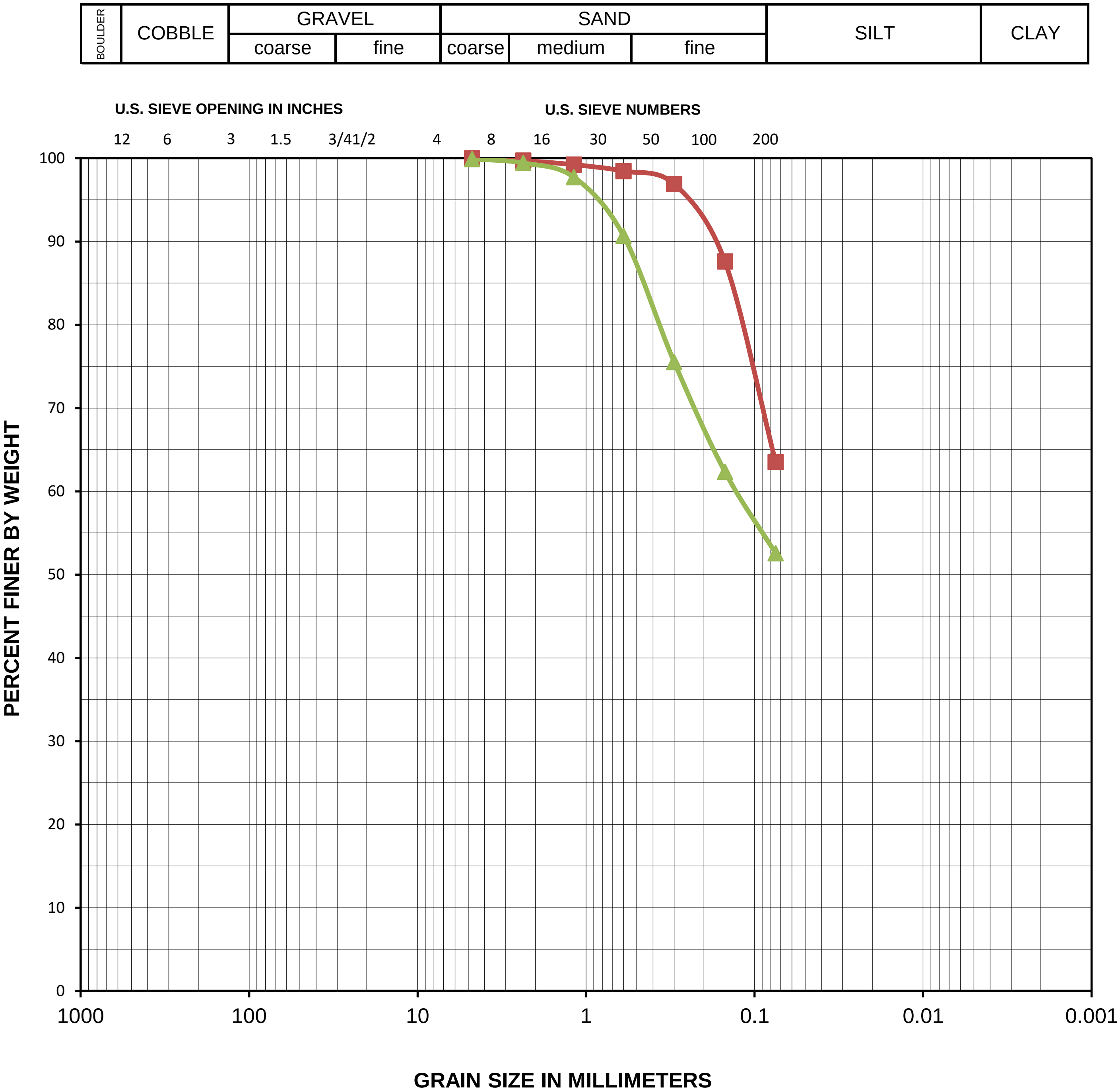
Boring	Depth (ft.)	Sample Description	Passing 3/4"	Passing #4	Passing #200
B-2	0-5	Sandy SILT (ML)	100.0	99.3	58.1
B-5	20	Silty SAND (SM)	100.0	100.0	48.5
B-8	30	Sandy SILT (ML)	100.0	99.8	67.3
B-9	15	Sandy SILT (ML)	100.0	99.4	55.1

PROJECT NO.:	220239
LAB TECH:	JD
INPUT BY:	YA
CHECKED BY:	SA
DATE:	5/5/2022
REVISED:	-

SIEVE ANALYSIS

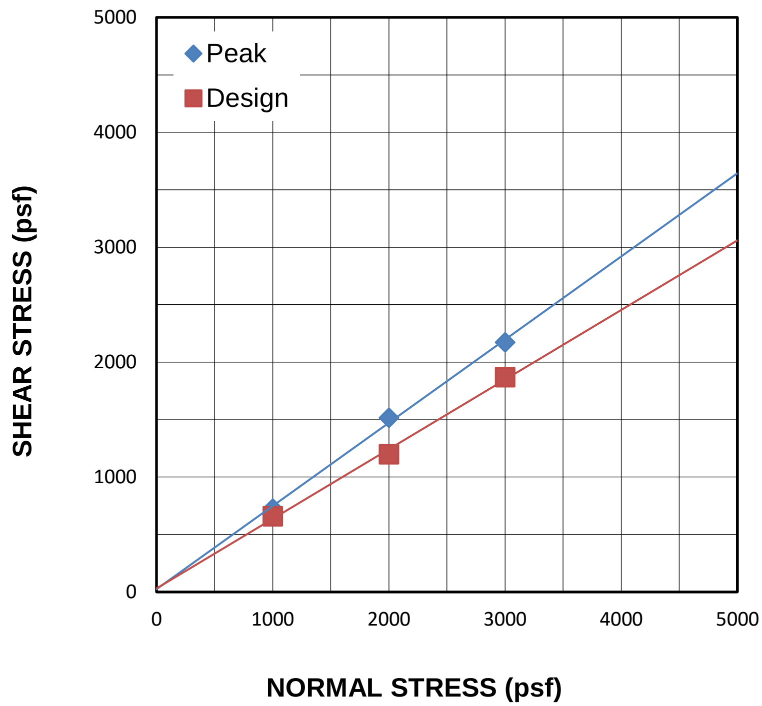
AQUATICS COMPLEX AND CTE BLDGS
3442 E. BARDSLEY AVE./MISSION OAK H.S.
TULARE, CA

TECHNICON
ENGINEERING SERVICES, INC.



Boring	Depth (ft.)	Sample Description	Passing 3/4"	Passing #4	Passing #200
B-7	10	Sandy SILT (ML)	100.0	100.0	63.5
B-10	0-5	Sandy SILT (ML)	100.0	99.9	52.6

PROJECT NO.:	220239	SIEVE ANALYSIS	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:	JD	AQUATICS COMPLEX AND CTE BLDGS 3442 E. BARDSLEY AVE./MISSION OAK H.S. TULARE, CA	
INPUT BY:	YA		
CHECKED BY:	SA		
DATE:	5/5/2022		
REVISED:	-		



Depth (ft.)		Sample Description
B-9	1	SANDY SILT (ML)

	Specimen No.	Dry Unit Weight (pcf)	Water Content (%)	Saturation (%)	Area (in^2)	Height (in)
Initial	1	102.6	6.8	29.5	4.60	1.00
	2	102.6	6.8	29.5	4.60	1.00
	3	102.6	6.8	29.5	4.60	1.00
At Test	Specimen No.	Dry Unit Weight (pcf)	Water Content (%)	Saturation (%)	Area (in^2)	Height (in)
	1	104.8	23.5	107.6	4.60	0.979
	2	105.5	25.3	118.1	4.60	0.972
	3	104.4	17.2	78.0	4.60	0.983

Specimen No.	Peak Shear Stress (psf)	Design Shear Stress (psf)	Normal Stress (psf)	Strain Rate (in/min)
1	725.8	657.4	1000	0.005
2	1515.7	1199.1	2000	0.005
3	2173.5	1869.0	3000	0.005

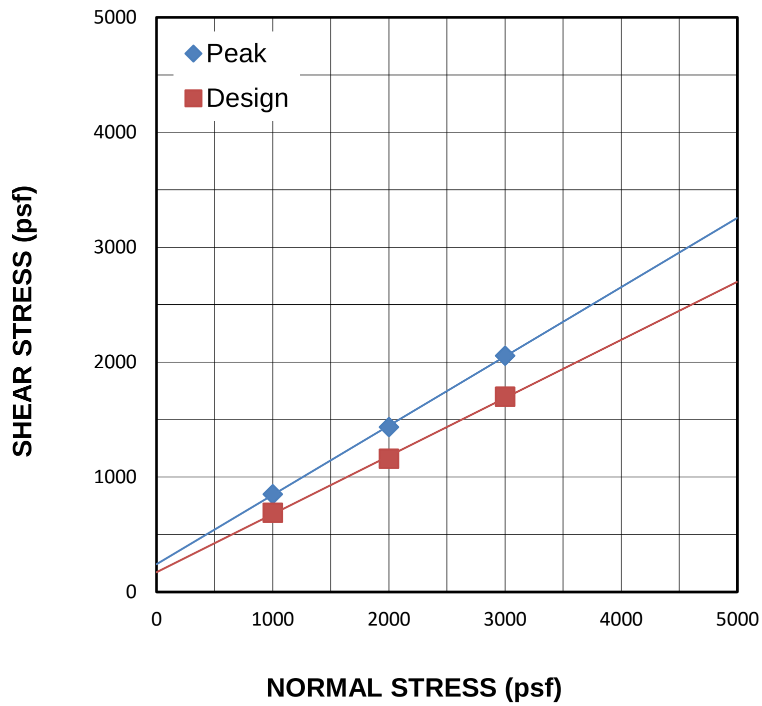
Results	Cohesion (psf)	Friction ϕ (deg)
Peak	24	35.9
Design	30	31.2

PROJECT NO	220239
LAB TECH:	
INPUT BY:	YA
CHECKED BY	SA
DATE:	4/29/2022
REVISED:	-

DIRECT SHEAR

AQUATICS COMPLEX AND CTE BLDGS
3442 E. BARDSLEY AVE./MISSION OAK H.S.
TULARE, CA

TECHNICON
ENGINEERING SERVICES, INC.



	Depth (ft.)	Sample Description
B-7	5	SANDY SILT (ML)

	Specimen No.	Dry Unit Weight (pcf)	Water Content (%)	Saturation (%)	Area (in^2)	Height (in)
Initial	1	101.8	18.6	78.9	4.60	1.00
	2	101.8	18.6	78.9	4.60	1.00
	3	101.8	18.6	78.9	4.60	1.00
At Test	Specimen No.	Dry Unit Weight (pcf)	Water Content (%)	Saturation (%)	Area (in^2)	Height (in)
	1	113.3	27	155.7	4.60	0.887
	2	103.4	25.5	112.6	4.60	0.985
	3	105.4	26.7	124.5	4.60	0.964

Specimen No.	Peak Shear Stress (psf)	Design Shear Stress (psf)	Normal Stress (psf)	Strain Rate (in/min)
1	849.9	688.8	1000	0.005
2	1434.2	1158.4	2000	0.005
3	2055.5	1700.0	3000	0.005

Results	Cohesion (psf)	Friction ϕ (deg)
Peak	241	31.1
Design	171	26.8

PROJECT NO	220239	DIRECT SHEAR	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:		AQUATICS COMPLEX AND CTE BLDGS 3442 E. BARDSLEY AVE./MISSION OAK H.S. TULARE, CA	
INPUT BY:	YA		
CHECKED BY	SA		
DATE:	4/29/2022		
REVISED:	-		

Boring	Depth (ft.)	Sample Description
B-2	0-5	SANDY SILT (ML)

Moisture		
Wet Weight (g)	Dry Weight (g)	Water Content (%)
200.0	180.71	10.7

Soil Specimen		
Mold Weight (g)	Soil + Mold Weight (g)	Soil Weight (g)
363.9	750.7	386.7
Mold Diameter (in)	Mold Height (in)	Mold Volume (ft^3)
4.0	1.0	12.57
Moist Density (pcf)	Dry Density (pcf)	Saturation (%)
116.6	105.4	48.1

Expansion		
Initial Reading (in)	Final Reading (in)	Expansion (in)
0.0000	0.0280	0.0280

Expansion Index, EI	
$EI_{measured}$	EI_{50}
28	27.0

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	220239	EXPANSION INDEX	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:		AQUATICS COMPLEX AND CTE BLDGS 3442 E. BARDSLEY AVE./MISSION OAK H.S. TULARE, CA	
INPUT BY:	YA		
CHECKED BY	SA		
DATE:	4/29/2022		
REVISED:	-		

Boring	Depth (ft.)	Sample Description
B-10	0-5	SANDY SILT (ML)

Moisture		
Wet Weight (g)	Dry Weight (g)	Water Content (%)
200.0	184.52	8.4

Soil Specimen		
Mold Weight (g)	Soil + Mold Weight (g)	Soil Weight (g)
367.6	786.9	419.3
Mold Diameter (in)	Mold Height (in)	Mold Volume (ft^3)
4.0	1.0	12.57
Moist Density (pcf)	Dry Density (pcf)	Saturation (%)
126.5	116.7	51.0

Expansion		
Initial Reading (in)	Final Reading (in)	Expansion (in)
0.0000	0.0222	0.0222

Expansion Index, EI	
$EI_{measured}$	EI_{50}
22.2	22.7

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	220239	EXPANSION INDEX	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:		AQUATICS COMPLEX AND CTE BLDGS 3442 E. BARDSLEY AVE./MISSION OAK H.S. TULARE, CA	
INPUT BY:	YA		
CHECKED BY	SA		
DATE:	4/29/2022		
REVISED:	-		

Boring	Depth (ft)	Sample Description
B-2	0-5	Sandy SILT (ML)

MINIMUM RESISTIVITY

Water Added (ml)	0	150	250	350	450				
Resistance (ohm)	500,000	3,200	820	690	720				
Resistivity (ohm-cm)*	532,500	3,408	873	735	767				

Box Constant=1.065

Minimum Resistivity (ohm-cm)	735
pH	7.97

Years to perforation*	22

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

CHEMICAL ANALYSIS

	Soluble Sulfate SO_4-S	Soluble Chloride Cl
	0.4 mg/kg	7.1 mg/kg
	0.4 mg/kg	5.3 mg/kg
	0.4 mg/kg	7.1 mg/kg
Average	0.4 mg/kg	6.5 mg/kg

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

PROJECT NO.: 220239	CORROSIVITY TESTS	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:		
INPUT BY: YA	AQUATICS COMPLEX AND CTE BLDGS 3442 E. BARDSLEY AVE./MISSION OAK H.S. TULARE, CA	
CHECKED BY: SA		
DATE: 4/29/2022		
REVISED: -		

Boring	Depth (ft)	Sample Description
B-10	0-5	Sandy SILT (ML)

MINIMUM RESISTIVITY

Water Added (ml)	0	150	250	350	450	550			
Resistance (ohm)	1,000,000	2,800	1,400	1,400	1,100	1,300			
Resistivity (ohm-cm)*	1,065,000	2,982	1,491	1,491	1,172	1,385			

Box Constant=1.065

Minimum Resistivity (ohm-cm)	1,172
pH	7.98

Years to perforation*	26

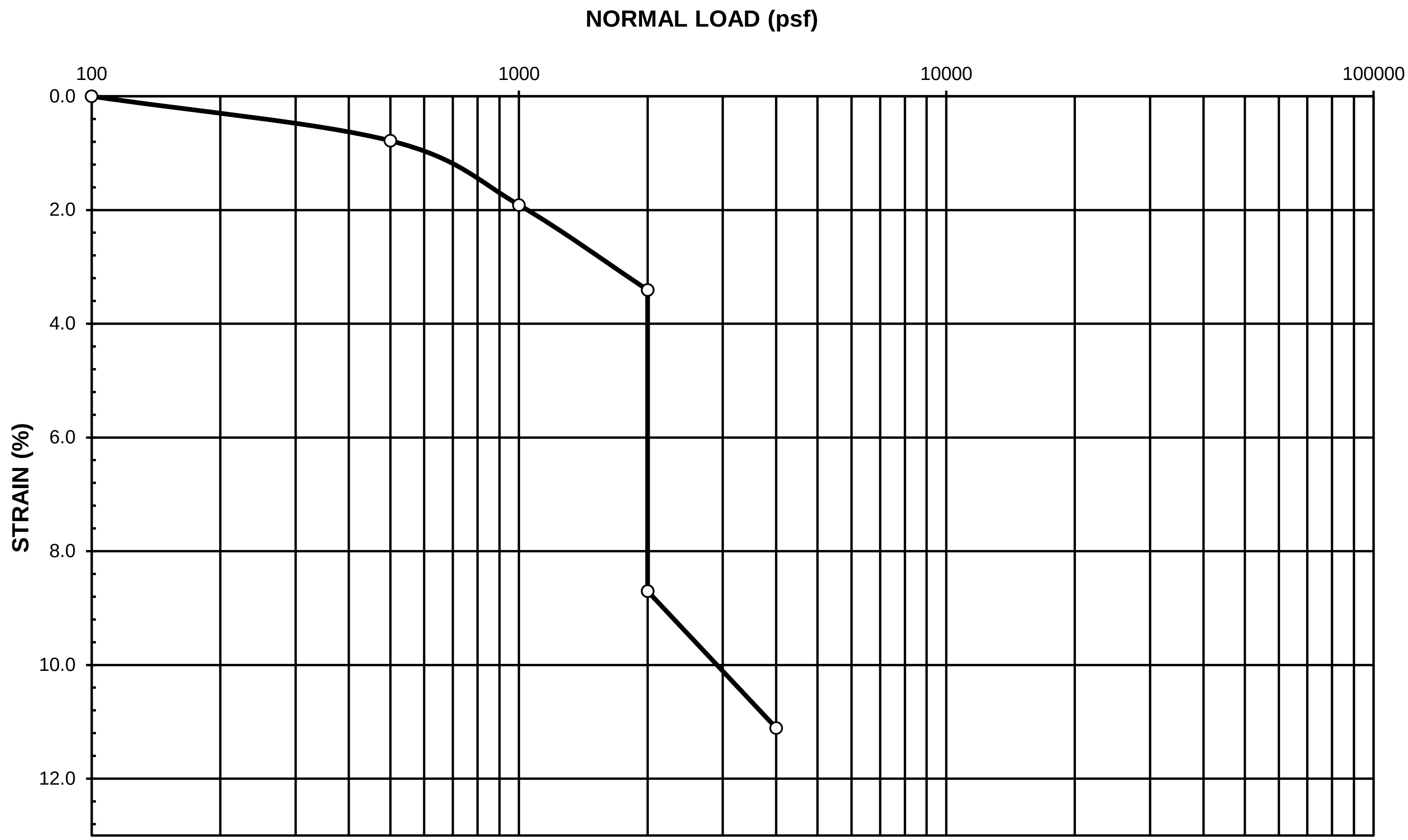
* Caltrans California Test 643 - Method for Estimating the Service Life of Steel Culverts

CHEMICAL ANALYSIS

	Soluble Sulfate SO_4-S	Soluble Chloride Cl
	0.4 mg/kg	1.8 mg/kg
	0.4 mg/kg	1.8 mg/kg
	0.4 mg/kg	1.8 mg/kg
Average	0.4 mg/kg	1.8 mg/kg

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

PROJECT NO.: 220239	CORROSIVITY TESTS	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:		
INPUT BY: YA	AQUATICS COMPLEX AND CTE BLDGS	
CHECKED BY: SA	3442 E. BARDSLEY AVE./MISSION OAK H.S.	
DATE: 4/29/2022	TULARE, CA	
REVISED: -		



Boring	Depth (ft)	Sample Description
B-5	1.0	Sandy SILT (ML)

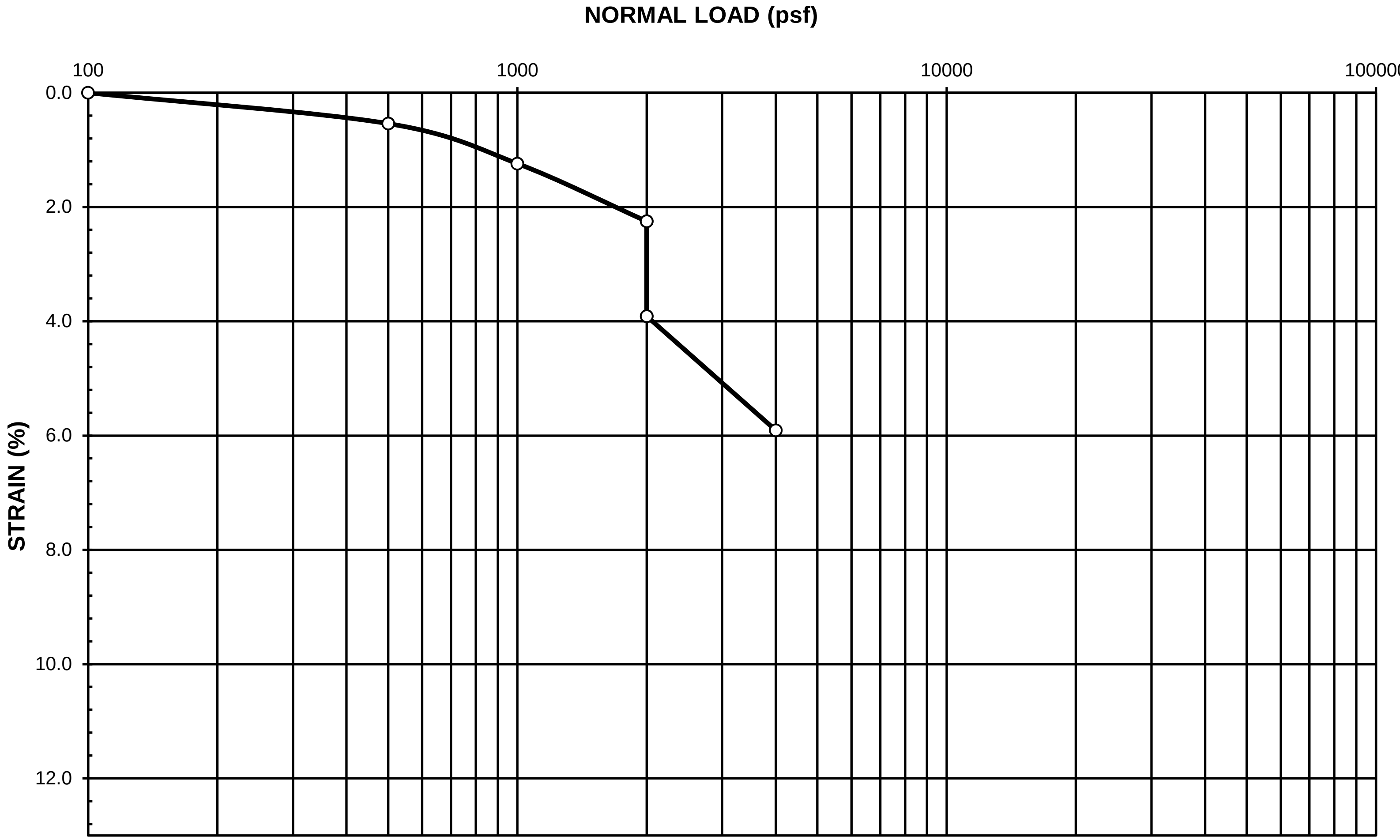
	Sample Diameter (in)	Sample Height (in)	Moisture Content (%)	Dry Density (pcf)
Initial	2.42	1.0100	9.9	108.9
Final	2.42	0.8978	18.0	121.3

PROJECT NO.:	220239
LAB TECH:	RJ
INPUT BY:	YA
CHECKED BY:	SA
DATE:	4/29/2022
REVISED:	-

COLLAPSE POTENTIAL

AQUATICS COMPLEX AND CTE BLDGS
3442 E. BARDSLEY AVE./MISSION OAK H.S.
TULARE, CA

TECHNICON
ENGINEERING SERVICES, INC.



Boring	Depth (ft)	Sample Description
B-9	10.0	Sandy SILT (ML)

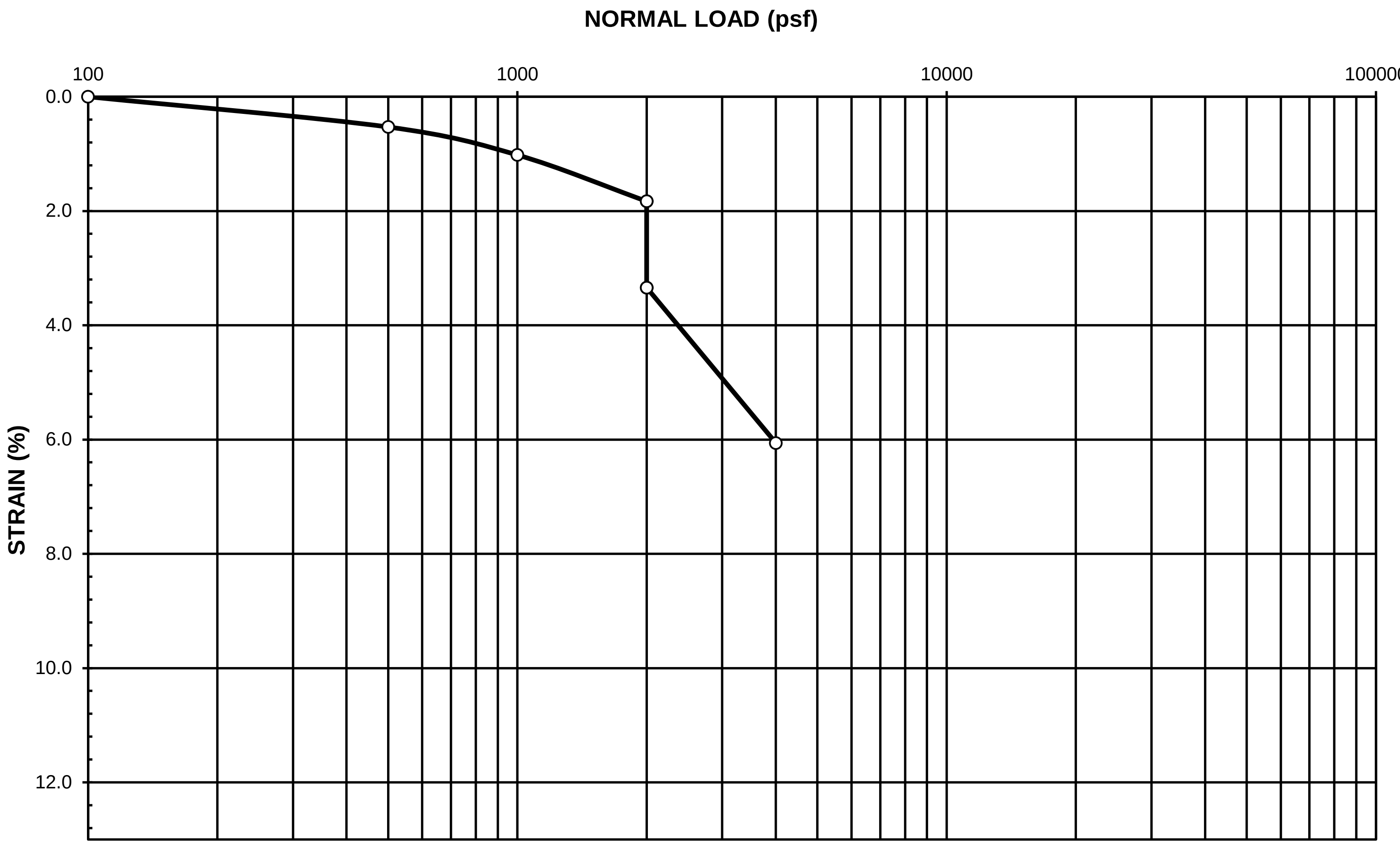
	Sample Diameter (in)	Sample Height (in)	Moisture Content (%)	Dry Density (pcf)
Initial	2.42	1.0000	16.0	113.8
Final	2.42	0.9409	17.8	121.0

PROJECT NO.:	220239
LAB TECH:	RJ
INPUT BY:	YA
CHECKED BY:	SA
DATE:	4/29/2022
REVISED:	-

COLLAPSE POTENTIAL

AQUATICS COMPLEX AND CTE BLDGS
3442 E. BARDSLEY AVE./MISSION OAK H.S.
TULARE, CA

TECHNICON
ENGINEERING SERVICES, INC.



Boring	Depth (ft)	Sample Description
B-5	5.0	Sandy SILT (ML)

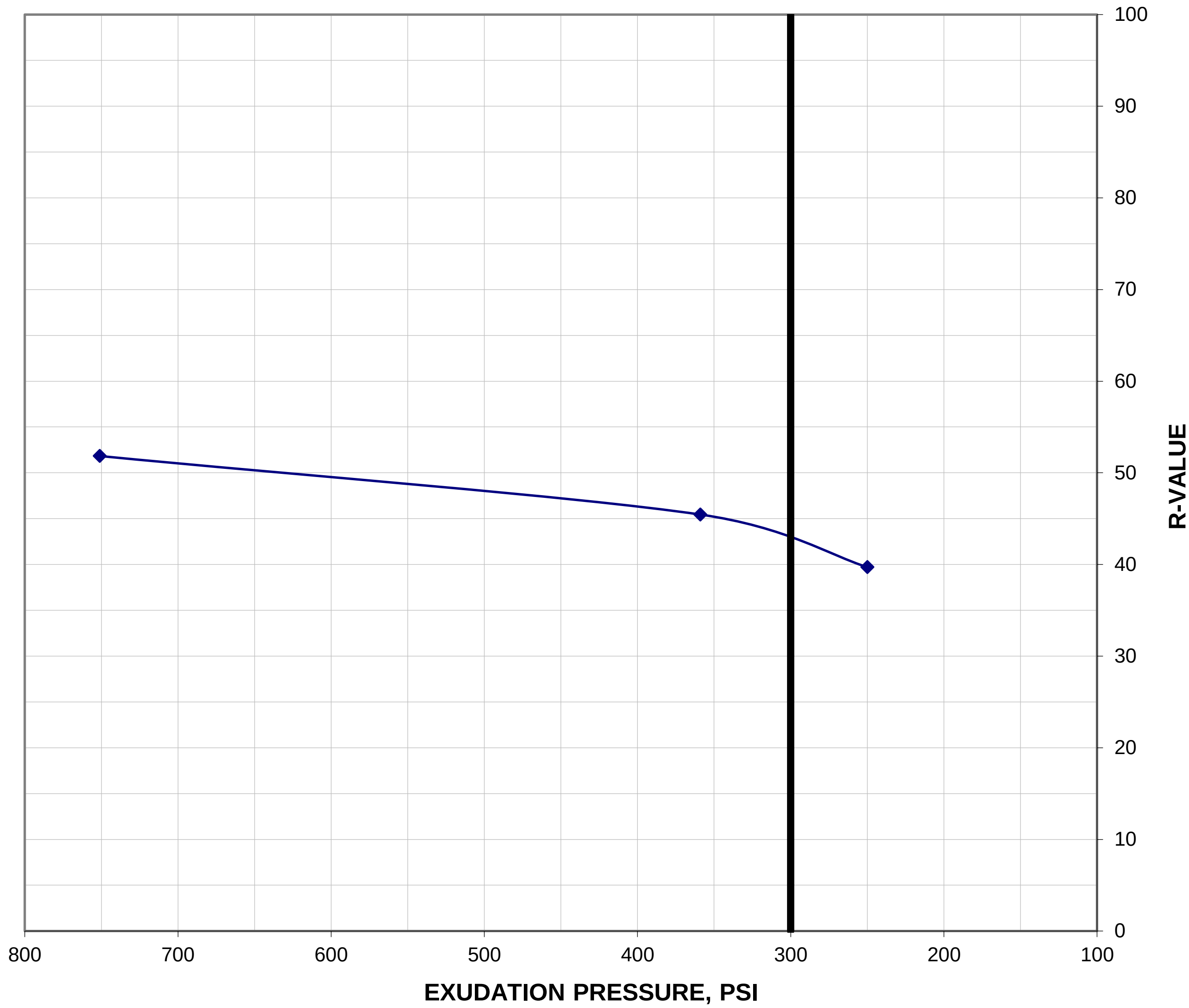
	Sample Diameter (in)	Sample Height (in)	Moisture Content (%)	Dry Density (pcf)
Initial	2.42	1.0000	16.0	106.6
Final	2.42	0.9394	27.1	113.5

PROJECT NO.:	220239
LAB TECH:	RJ
INPUT BY:	YA
CHECKED BY:	SA
DATE:	4/29/2022
REVISED:	-

COLLAPSE POTENTIAL

AQUATICS COMPLEX AND CTE BLDGS
3442 E. BARDSLEY AVE./MISSION OAK H.S.
TULARE, CA

TECHNICON
ENGINEERING SERVICES, INC.

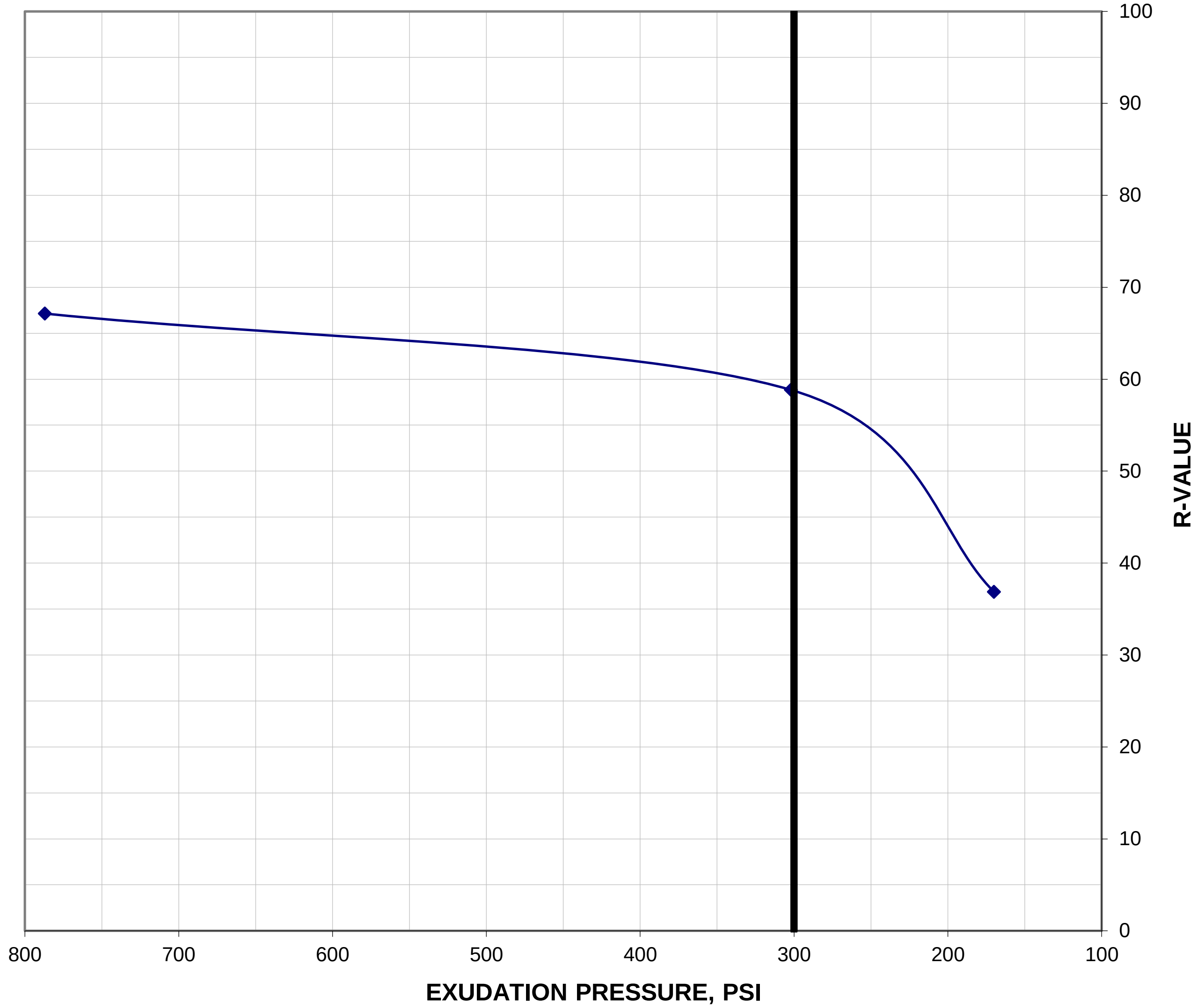


Boring	Depth (ft)	Sample Description
RV-1	0-5	Sandy Silt (ML)

Specimen	1	2	3
Exudation Pressure (psi)	250	359	751
Moisture Content at Test (%)	15.0	13.8	13.1
Dry Density (pcf)	113.5	113.7	113.8
Expansion Pressure (psf)	130	178	307
R-Value by Stabilometer	40	45	52
R-Value by Expansion Pressure (TI = 4.5)	31		
R-Value at 300 psi Exudation Pressure	43		

Controlling R-Value	31

PROJECT NO: 220239	RESISTANCE VALUE	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH: FM		
INPUT BY: YA	AQUATICS COMPLEX AND CTE BLDGS	
CHECKED BY: SA	3442 E. BARDSLEY AVE./MISSION OAK H.S.	
DATE: 4/29/2022	TULARE, CA	
REVISED: -		

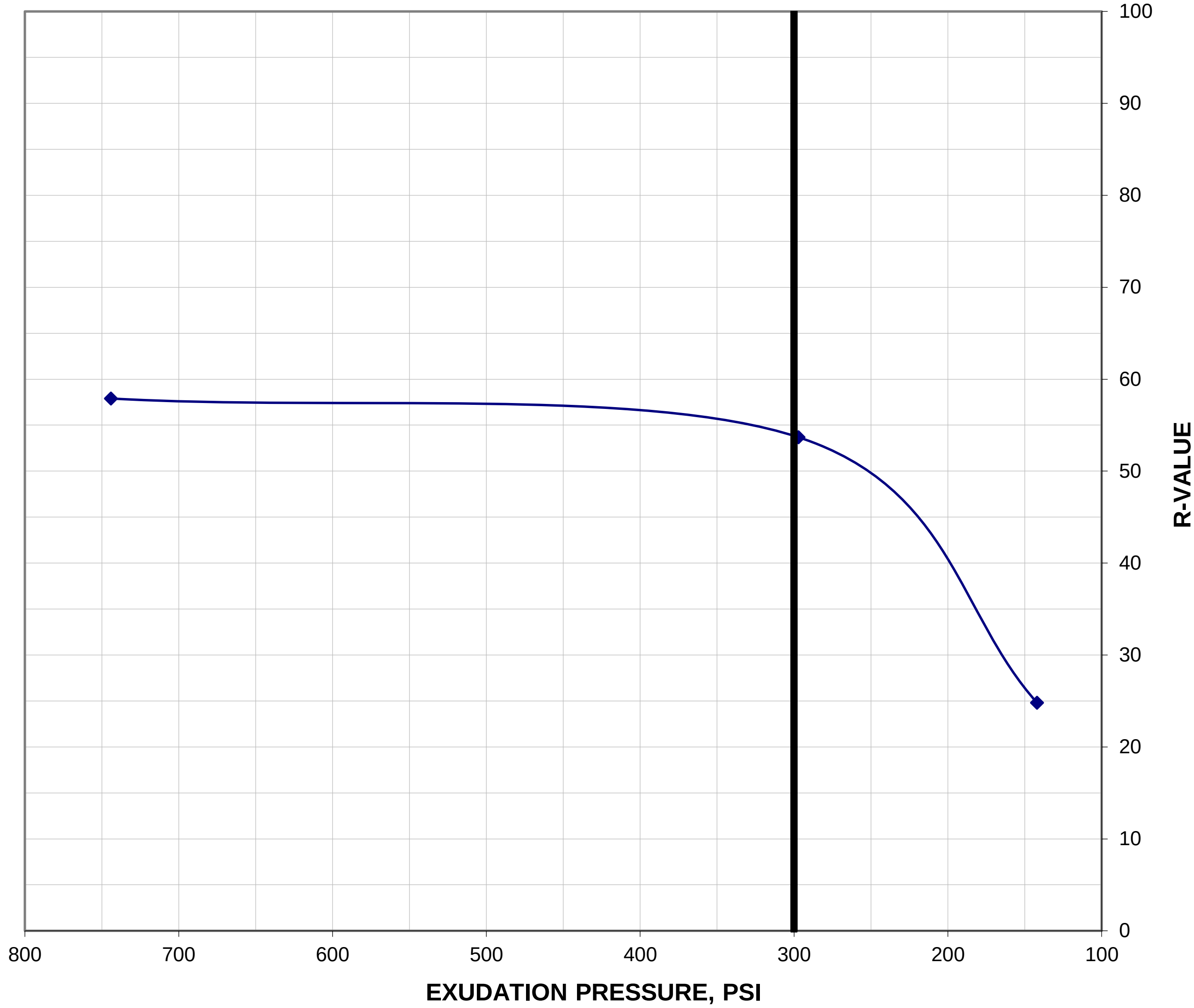


Boring	Depth (ft)	Sample Description
RV-2	0-5	Sandy Silt (ML)

Specimen	1	2	3
Exudation Pressure (psi)	170	302	787
Moisture Content at Test (%)	16.1	15.5	14.8
Dry Density (pcf)	111.6	113.1	112.2
Expansion Pressure (psf)	165	247	381
R-Value by Stabilometer	37	59	67
R-Value by Expansion Pressure (TI = 4.5)	11		
R-Value at 300 psi Exudation Pressure	59		

Controlling R-Value	11

PROJECT NO:	220239	RESISTANCE VALUE	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:	FM		
INPUT BY:	YA	AQUATICS COMPLEX AND CTE BLDGS	
CHECKED BY:	SA	3442 E. BARDSLEY AVE./MISSION OAK H.S.	
DATE:	4/29/2022	TULARE, CA	
REVISED:	-		

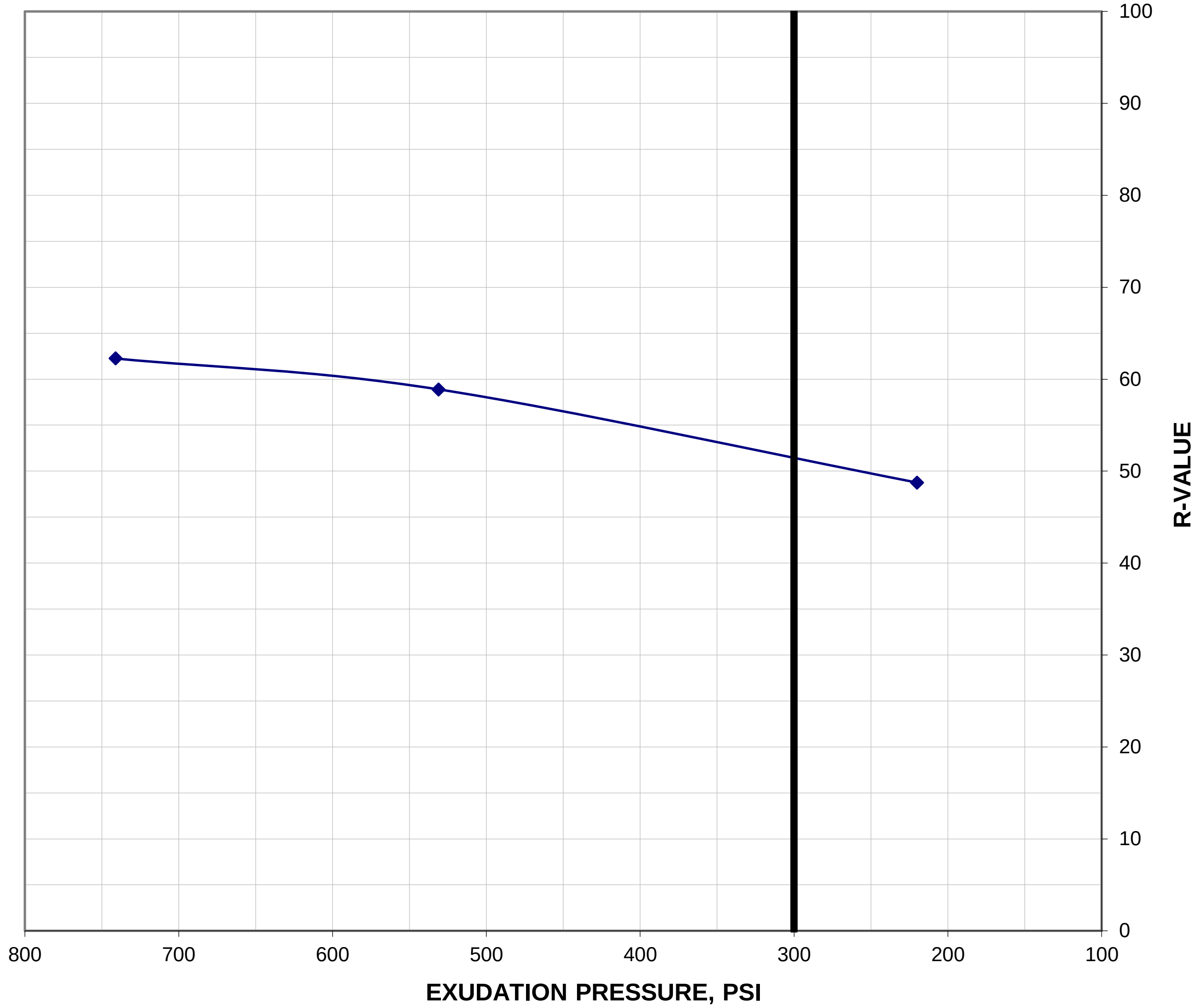


Boring	Depth (ft)	Sample Description
RV-3	0-5	Sandy Silt (ML)

Specimen	1	2	3
Exudation Pressure (psi)	142	297	744
Moisture Content at Test (%)	15.8	15.3	14.9
Dry Density (pcf)	112.0	112.6	112.8
Expansion Pressure (psf)	78	113	182
R-Value by Stabilometer	25	54	58
R-Value by Expansion Pressure (TI = 4.5)	28		
R-Value at 300 psi Exudation Pressure	54		

Controlling R-Value	28

PROJECT NO:	220239	RESISTANCE VALUE	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:	FM		
INPUT BY:	YA	AQUATICS COMPLEX AND CTE BLDGS	
CHECKED BY:	SA	3442 E. BARDSLEY AVE./MISSION OAK H.S.	
DATE:	4/29/2022	TULARE, CA	
REVISED:	-		



Boring	Depth (ft)	Sample Description
RV-4	0-5	Sandy Silt (ML)

Specimen	1	2	3
Exudation Pressure (psi)	220	531	741
Moisture Content at Test (%)	14.6	14.0	13.5
Dry Density (pcf)	111.0	114.0	113.9
Expansion Pressure (psf)	126	134	225
R-Value by Stabilometer	49	59	62
R-Value by Expansion Pressure (TI = 4.5)	11		
R-Value at 300 psi Exudation Pressure	52		

Controlling R-Value	11

PROJECT NO:	220239	RESISTANCE VALUE	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:	FM		
INPUT BY:	YA	AQUATICS COMPLEX AND CTE BLDGS	
CHECKED BY:	SA	3442 E. BARDSLEY AVE./MISSION OAK H.S.	
DATE:	4/29/2022	TULARE, CA	
REVISED:	-		

USGS DEAGGREGATION SUMMARIES

APPENDIX C

U.S. Geological Survey - Earthquake Hazards Program

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.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (v4.2.0)

Latitude

Decimal degrees

36.19789218

Longitude

Decimal degrees, negative values for western longitudes

-119.29886481

Site Class

259 m/s (Site class D)

Spectral Period

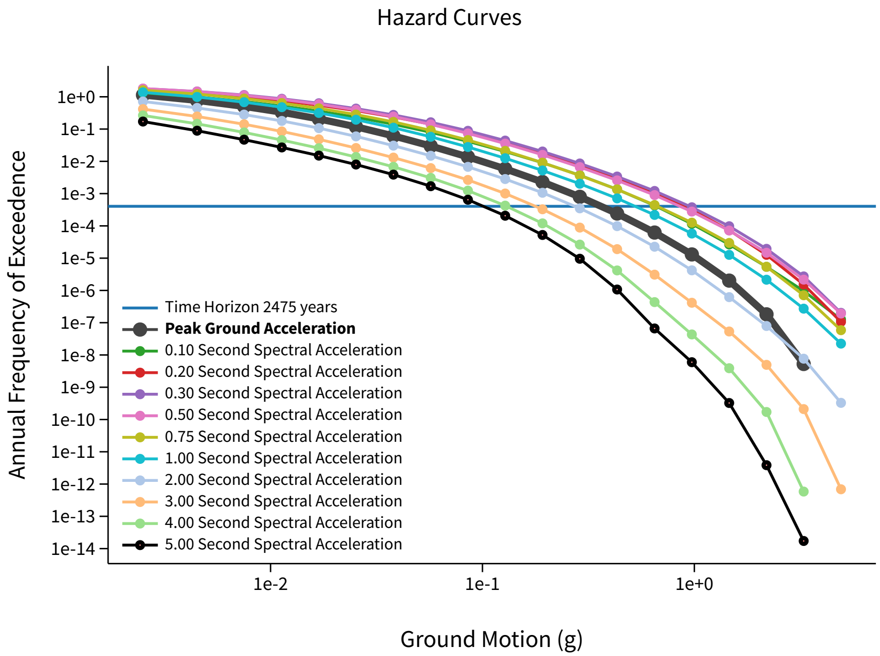
Peak Ground Acceleration

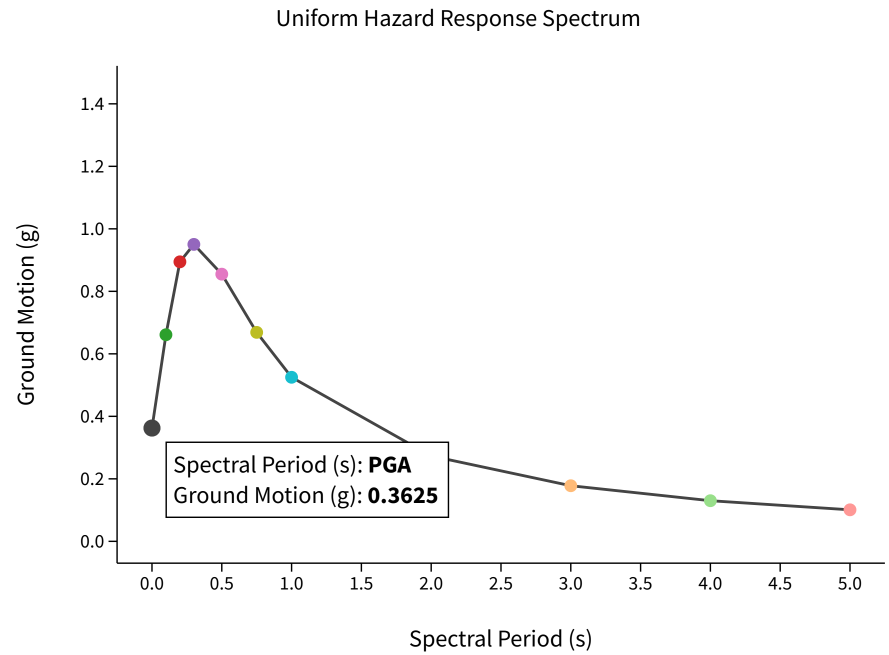
Time Horizon

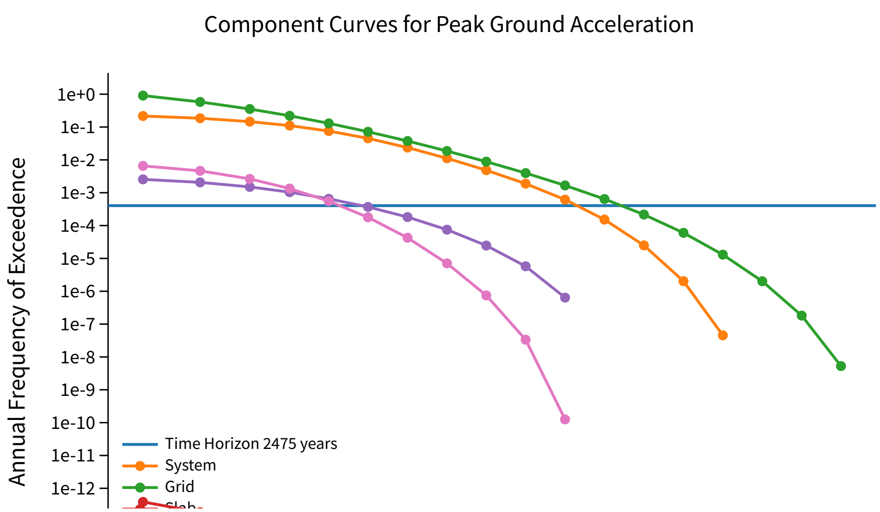
Return period in years

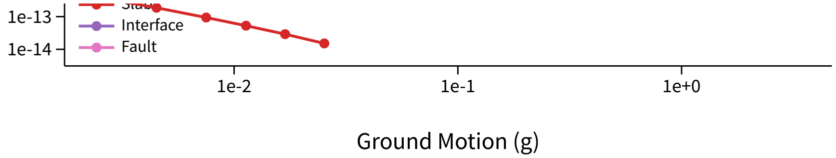
2475

Hazard Curve







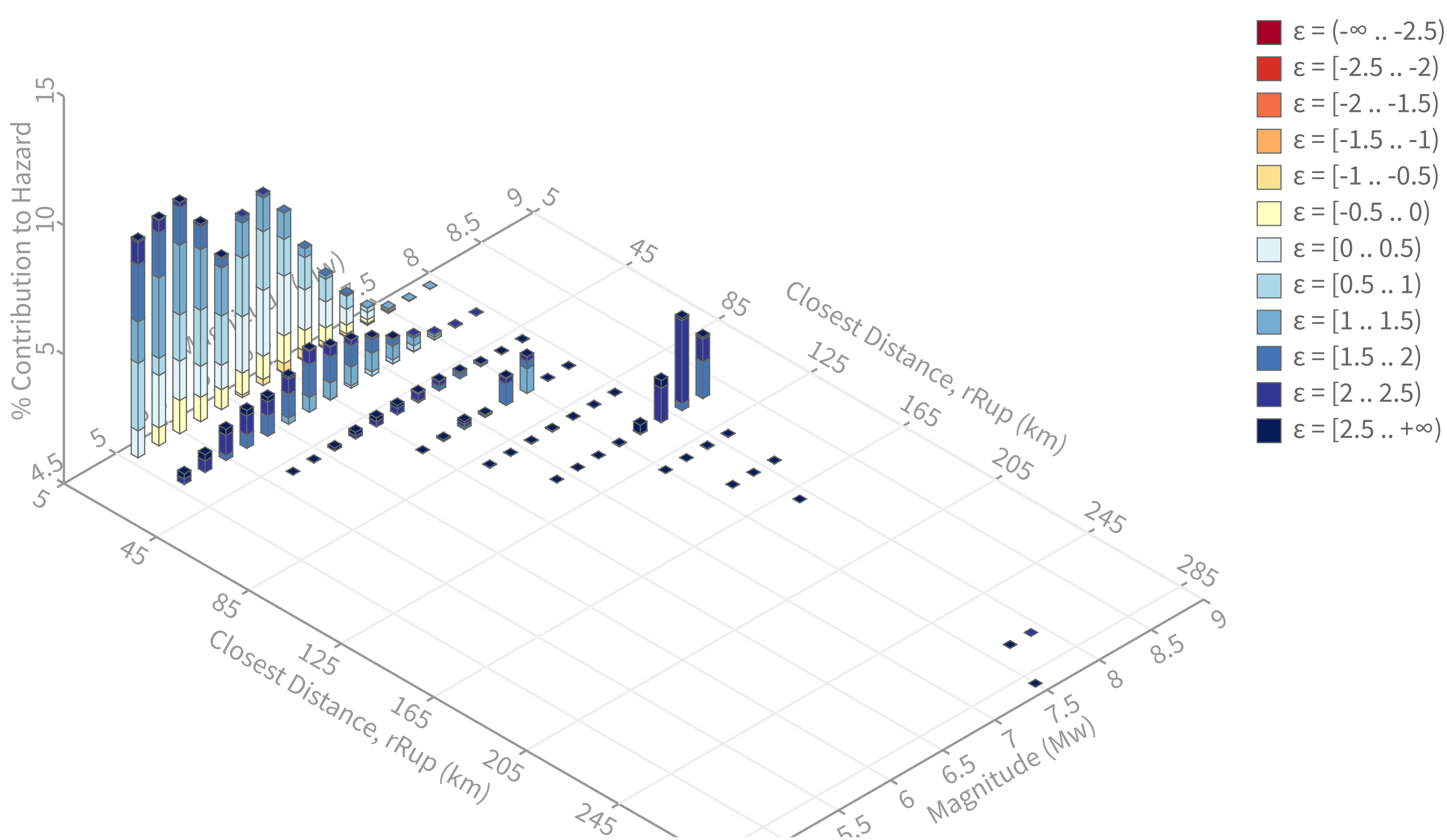


View Raw Data

Deaggregation

Component

Total




285
4.5
5

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.36250568 g

Recovered targets

Return period: 2723.5222 yrs
Exceedance rate: 0.0003671716 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.18 %

Mean (over all sources)

m: 6.21
r: 22.96 km
ε_0: 1.09 σ

Mode (largest m-r bin)

m: 5.5
r: 10.13 km
ε_0: 0.88 σ
Contribution: 8.95 %

Mode (largest m-r-ε_0 bin)

m: 8.1
r: 101.67 km
ε_0: 2.17 σ
Contribution: 3.23 %

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							44.62
PointSourceFinite: -119.299, 36.202		4.90	5.63	0.14	119.299°W	36.202°N	0.00	4.79
PointSourceFinite: -119.299, 36.202		4.90	5.63	0.14	119.299°W	36.202°N	0.00	4.74
PointSourceFinite: -119.299, 36.328		13.63	6.00	1.08	119.299°W	36.328°N	0.00	3.42
PointSourceFinite: -119.299, 36.328		13.63	6.00	1.08	119.299°W	36.328°N	0.00	3.37
PointSourceFinite: -119.299, 36.283		9.96	5.84	0.78	119.299°W	36.283°N	0.00	2.10
PointSourceFinite: -119.299, 36.283		9.96	5.84	0.78	119.299°W	36.283°N	0.00	2.09
PointSourceFinite: -119.299, 36.274		9.26	5.81	0.71	119.299°W	36.274°N	0.00	1.74
PointSourceFinite: -119.299, 36.292		10.68	5.87	0.84	119.299°W	36.292°N	0.00	1.71
PointSourceFinite: -119.299, 36.274		9.26	5.81	0.71	119.299°W	36.274°N	0.00	1.70
PointSourceFinite: -119.299, 36.292		10.68	5.87	0.84	119.299°W	36.292°N	0.00	1.70
PointSourceFinite: -119.299, 36.301		11.40	5.90	0.91	119.299°W	36.301°N	0.00	1.12
PointSourceFinite: -119.299, 36.301		11.40	5.90	0.91	119.299°W	36.301°N	0.00	1.10
PointSourceFinite: -119.299, 36.391		19.00	6.19	1.40	119.299°W	36.391°N	0.00	1.07
PointSourceFinite: -119.299, 36.391		19.00	6.19	1.40	119.299°W	36.391°N	0.00	1.05
UC33brAvg_FM32 (opt)	Grid							44.56
PointSourceFinite: -119.299, 36.202		4.90	5.63	0.15	119.299°W	36.202°N	0.00	4.78
PointSourceFinite: -119.299, 36.202		4.90	5.63	0.15	119.299°W	36.202°N	0.00	4.73
PointSourceFinite: -119.299, 36.328		13.63	5.99	1.08	119.299°W	36.328°N	0.00	3.41
PointSourceFinite: -119.299, 36.328		13.63	5.99	1.08	119.299°W	36.328°N	0.00	3.37
PointSourceFinite: -119.299, 36.283		9.96	5.84	0.78	119.299°W	36.283°N	0.00	2.09
PointSourceFinite: -119.299, 36.283		9.96	5.84	0.78	119.299°W	36.283°N	0.00	2.09
PointSourceFinite: -119.299, 36.274		9.26	5.81	0.71	119.299°W	36.274°N	0.00	1.74
PointSourceFinite: -119.299, 36.292		10.68	5.87	0.84	119.299°W	36.292°N	0.00	1.70
PointSourceFinite: -119.299, 36.274		9.26	5.81	0.71	119.299°W	36.274°N	0.00	1.70
PointSourceFinite: -119.299, 36.292		10.68	5.87	0.84	119.299°W	36.292°N	0.00	1.69
PointSourceFinite: -119.299, 36.301		11.40	5.90	0.91	119.299°W	36.301°N	0.00	1.12
PointSourceFinite: -119.299, 36.301		11.40	5.90	0.91	119.299°W	36.301°N	0.00	1.10
PointSourceFinite: -119.299, 36.391		19.00	6.19	1.41	119.299°W	36.391°N	0.00	1.07
PointSourceFinite: -119.299, 36.391		19.00	6.19	1.41	119.299°W	36.391°N	0.00	1.05
UC33brAvg_FM32	System							5.42

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
San Andreas (Cholame) rev [7]		101.47	8.10	2.18	120.183°W	35.632°N	231.92	3.77
Great Valley 14 (Kettleman Hills) [1]		63.51	7.42	1.66	119.944°W	35.981°N	247.56	1.23
UC33brAvg_FM31	System							5.41
San Andreas (Cholame) rev [7]		101.47	8.10	2.18	120.183°W	35.632°N	231.92	3.78
Great Valley 14 (Kettleman Hills) [1]		63.51	7.42	1.66	119.944°W	35.981°N	247.56	1.20

SITE SPECIFIC GROUND MOTION ANALYSIS

APPENDIX D

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

Technicon Engineering Services, Inc.

Project:	**Mission Oak High School/Aquatics Complex and CTE Bldg.**
Job #:	**TES No. 220239**
Date:	**4/27/2022**
Checked by:	**SA**

TECHNICON ENGINEERING SERVICES, INC.

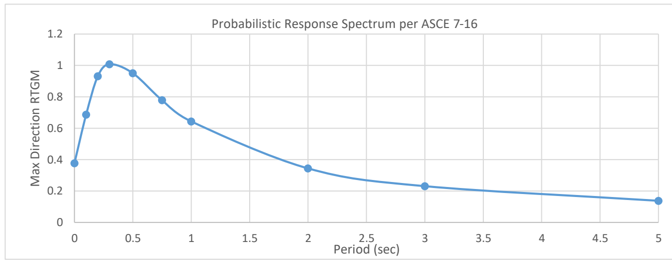
INPUT
OUTPUT
ANALYSIS

https://seismicmaps.org/ ** Values input from OSHPD seismic design map

S_S	0.587
S_1	0.229
S_{DS}	0.521
PGA_M	0.344
F_a	1.33

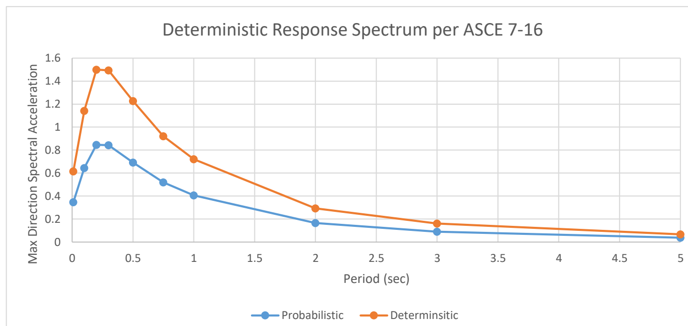
1. Use Unified Hazard Tool "raw data" from Hazard Curve & Risk-Targeted Ground Motion Calculator to get "UHGM & RTGM" values
   a. Plot time vs. adjusted RTGM
2. Input M_w and R_{rup} into NGAW2 Excel worksheet. M_w & R_{rup} can be found with deagg sheet (unified hazard tool) "Mean (over all sources)".
   a. PS_a 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	0.363	0.343	1.1	0.3773
0.1	0.661	0.624	1.1	0.6864
0.2	0.895	0.847	1.1	0.9317
0.3	0.951	0.896	1.125	1.008
0.5	0.855	0.809	1.175	0.950575
0.75	0.669	0.629	1.2375	0.7783875
1	0.525	0.495	1.3	0.6435
2	0.273	0.255	1.35	0.34425
3	0.178	0.165	1.4	0.231
5	0.101	0.092	1.5	0.138



Scaling Factor: **1.772798185**

	*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	0.3153	1.1	0.34683	0.614859594
0.1	0.58551	1.1	0.644061	1.141790172
0.2	0.7692	1.1	0.84612	1.5
0.3	0.749	1.125	0.842625	1.49380407
0.5	0.5892	1.175	0.69231	1.227325911
0.75	0.41988	1.2375	0.5196015	0.921148596
1	0.31303	1.3	0.406939	0.72142072
2	0.1225	1.35	0.165375	0.2931765
3	0.06506	1.4	0.091084	0.16147355
5	0.02533	1.5	0.037995	0.067357467



- ASCE 7-16 Section 21.2.2

If Largest Deterministic Spectral acceleration < 1.5, then scaling by a factor of F_a1.5.

Table **11.4.1** : Site Class D @ $S_S \geq 1.5$ → F_a = 1.33

F_a1.5 → F_a = **1.995**

- Section 21.3

F_v is taken as 2.4 for $S_1 < 0.2$ or 2.5 for $S_1 > 0.2$

Fv → 2.5

- Section 11.4.6 - Design Response Spectrum

$$T_0 = 0.2\left(\frac{S_{D1}}{S_{DS}}\right) \qquad T_S = \left(\frac{S_{D1}}{S_{DS}}\right)$$

equ. 11.4-2: $S_{M1} = S_1 * F_V$ → 0.5725

equ. 11.4-4: $S_{D1} = \left(\frac{2}{3}\right) S_{M1}$ → 0.382

T_0 → 0.147

T_S → 0.733

S_S	0.587
S_1	0.229
S_{DS} * from seismic design map	0.521
S_{D1} * from section 11.4.6	0.382
T_0	0.147
T_S	0.733

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

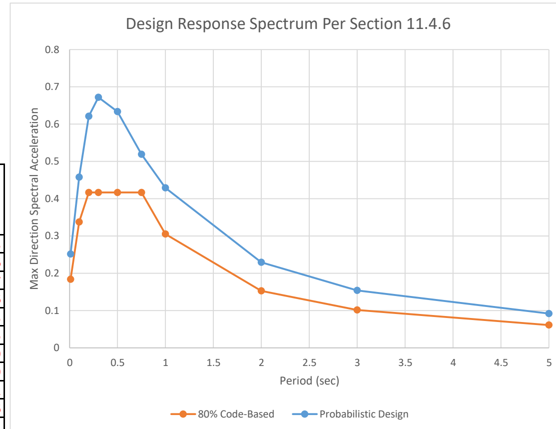
Technicon Engineering Services, Inc.	
Project:	Mission Oak High School/Aquatics Complex and CTE Bldg.
Job #:	TES No. 220239
Date:	4/27/2022
Checked by:	SA
S_S	0.587
S_1	0.229
S_{DS}	0.521
PGA_M	0.344
F_a	1.33

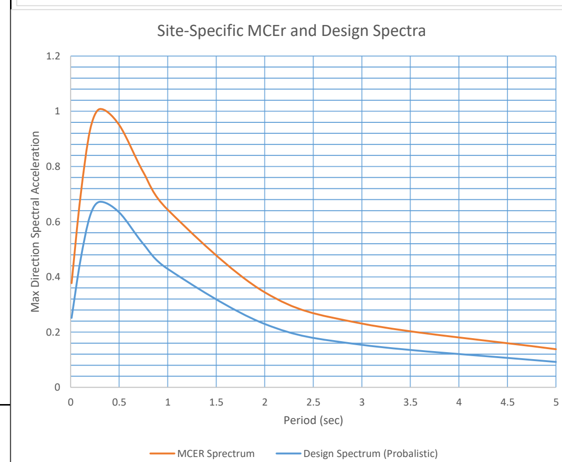
TECHNICON
ENGINEERING SERVICES, INC.

INPUT
OUTPUT
ANALYSIS

Site-Specfic 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*S_a
0.01	0.229735974	T less than T_o	0.183788779	0.251533333	0.002515333
0.1	0.421759738		0.33740779	0.4576	0.04576
0.2	0.521	T_0 < T < T_S; T = S_{DS}	0.4168	0.621133333	0.124226667
0.3	0.521		0.4168	0.672	0.2016
0.5	0.521		0.4168	0.633716667	0.316858333
0.75	0.521		0.4168	0.518925	0.38919375
1	0.381666667	T > T_S; S_a = S_{D1}/T	0.305333333	0.429	0.429
2	0.190833333		0.152666667	0.2295	0.459
3	0.127222222		0.101777778	0.154	0.462
5	0.076333333		0.061066667	0.092	0.46





- Section 21.4 Design Acceleration Parameters

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

$S_{a\ max}$ → 0.672

$S_{DS} = 90\% * S_{a\ max}$ → 0.605

$S_{MS} = 1.5 * S_{DS}$ → 0.907

V_{s30} < 365 m/s

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

Max S_a between T=1-5 sec → 0.462

S_{D1} → 0.462

$S_{M1} = 1.5 * S_{D1}$ S_{M1} → 0.693

- Section 21.5.1 - Probabilistic MCE_G Peak Ground Acceleration

Probablisitc PGA from UHGM @ T=0 sec

$PGA_{prob.}$ → 0.363

- Section 21.5.2 - Determinsitic MCE_G Peak Ground Acceleration

Determinsitic PGA from 84th Spectral Acceleration @ T=0.01 sec

PGA → 0.315

Table **11.8-1**: Site Class D @ PGA = 0.5 → F_{PGA} = 1.344

$0.5F_{PGA}$ = 0.672

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

Therefore; $PGA_{det.}$ → 0.672

- Section 21.5.3 - Site Specific MCE_G Peak Ground Acceleration

$PGA_{prob.}$ → 0.363

*Take the lesser

→ PGA_{SS} 0.363

$PGA_{det.}$ → 0.672

PGA CHECK

From Seismic Design Map: PGA_M → 0.344

80 % of PGA_M → 0.275

*Take the greater

Site-Specific PGA → 0.363

Final Seismic Design Values	
S_S	0.587
S_1	0.229
S_{MS}	0.907
S_{DS}	0.605
S_{D1}	0.462
S_{M1}	0.693
F_A	1.330
F_V	2.500
PGA_M	0.363

LIQUEFACTION ANALYSIS AND SEISMICALLY INDUCED SETTLEMENT CALCULATIONS

APPENDIX E

Project Aquatics Complex and CTE Buildings
DSA File
DSA App No.

Calc by AA Date 5/9/22
Checked by SA Date 5/9/22

Project No: 220239
Boring: B-8 and B-10

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*(exp(-0.4605*Mw))
******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 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.

Factor of Safety, F_L is:
F_L = CRR / CSR = Uniform CSR necessary to trigger liquefaction/Equivalent, Uniform, earthquake induced CSR

Hammer Efficiencies - Technicon Drilling Rigs	
CME 45	87.3%
CME 55	74.3%
CME 75	72.5%

Rod Length = **1.22** meters above grounds surface
Hammer Efficiency = **87%** Emean/E60 = Energy Ratio to correct to standard 60% Energy
Ring Sampler Corr. = **0.65**

Surcharge = Any surcharge on top of the ground (psf)
[1]$C_N = 2.2/(1.2+s'_0/P_a)$Youd and Idriss 2001 Formula (10)

Emean/E60= 1.455 Sur.= 0 psf Measured Ground Water Depth = 100 feet Design Ground Water Depth = 14.8 feet acc. max = 0.362 g Earthq. Mw = 6.21

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 (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_B	C_R	C_s	$C_BC_RC_s$	Corrected Blow Count $(N_1)_{60}$	$(N_1)_{60cs}$	$CSR_{7.5}$ Induced	$CRR_{7.5}$ (Resist. - c.sand)	Factor of Safety F_L	Will It Liquefy?
3	4	ML	3	0.09	0.09	0.09	0.5	1.70	117	117	2	14	5.000	1.200	0.997	1.81	52.0	1.0	0.75	1.00	0.75	16.9	25.3	0.129	0.297	2.29	ABOVE
8	4	ML	5	0.32	0.32	0.32	1.7	1.47	117	117	1	6	5.000	1.200	0.987	1.81	52.0	1.0	0.75	1.20	0.90	11.6	18.9	0.128	0.202	1.57	ABOVE
13	4	ML	5	0.61	0.61	0.61	3.2	1.25	117	117	1	17	5.000	1.200	0.976	1.81	52.0	1.0	0.85	1.20	1.02	31.4	42.7	0.127	LARGE	LARGE	ABOVE
14.8	4	ML	1.8	0.81	0.81	0.81	4.2	1.13	117	117	1	11	5.000	1.200	0.968	1.81	55.0	1.0	0.85	1.20	1.02	18.4	27.1	0.126	0.341	2.72	ABOVE
18	4	ML	3.2	0.96	0.96	0.91	5.0	1.06	117	122.5	1	11	5.000	1.200	0.962	1.81	55.0	1.0	0.95	1.20	1.14	19.3	28.1	0.131	0.374	2.85	NO
23	4	SM	5	1.22	1.22	1.06	6.2	0.95	132	138.1	2	16	5.000	1.200	0.952	1.81	48.0	1.0	0.95	1.00	0.95	13.6	21.3	0.143	0.233	1.63	NO
28	4	SM	5	1.55	1.55	1.25	7.8	0.84	132	138.1	1	9	5.000	1.200	0.941	1.81	48.0	1.0	0.95	1.20	1.14	12.5	20.0	0.152	0.215	1.42	NO
33	4	SM	5	1.88	1.88	1.43	9.3	0.75	132	134	2	26	5.000	1.200	0.926	1.81	48.0	1.0	1.00	1.00	1.00	18.5	27.2	0.158	0.343	2.17	NO
38	4	SM	5	2.18	2.18	1.61	10.8	0.69	110	134	1	22	5.000	1.200	0.885	1.81	48.0	1.0	1.00	1.20	1.20	26.3	36.6	0.156	LARGE	LARGE	NO
43	4	ML	5	2.48	2.48	1.79	12.3	0.63	132	133.2	2	40	5.000	1.200	0.844	1.81	55.0	1.0	1.00	1.00	1.00	23.9	33.6	0.152	LARGE	LARGE	NO
48	4	CL	5	2.81	2.81	1.96	13.9	0.58	132	133.2	1	16	5.000	1.200	0.804	1.81	70.0	1.0	1.00	1.20	1.20	16.2	24.4	0.150	0.281	1.88	NO
51.5	4	SM	3.5	3.09	3.09	2.11	15.2	0.40	132	133.2	1	22	5.000	1.200	0.769	1.81	45.0	1.0	1.00	1.20	1.20	15.4	23.4	0.146	0.264	1.81	NO

Project Aquatics Complex and CTE Buildings | Calc by AA | Date 5/9/22
DSA File | Checked by SA | Date 5/9/22
DSA App No.

Project No: 220239
Boring: B-8 and B-10

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*(exp(-0.4605*Mw))

******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   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 = **87%** Emean/E60 = Energy Ratio to correct to standard 60% Energy

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

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

Ring Sampler Corr. = **0.65**

Emean/E60= 1.455 | Sur.= 0 psf | Measured Ground Water Depth = 100 feet | Design Ground Water Depth = 14.8 feet | acc. max = 0.362 g | Earthq. Mw = 6.21

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)	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	Stress Reduct. Coeff. r_d	MSF	Est. % Fines	$C_BC_RC_S$	Corrected Blow Count $(N_1)_{60}$	ΔN	$(N_1)_{60cs}$	$CSR_{7.5}$ Induced	Factor of Safety F_L	ε (Only if FS<1.3) (%)	Settlement, inches
3	4	ML	3	0.09	0.09	0.09	0.5	1.70	117	117	2	14	0.997	1.81	52.0	0.75	16.9	4.2	21.0	0.129	2.29	-	ABOVE
8	4	ML	5	0.32	0.32	0.32	1.7	1.47	117	117	1	6	0.987	1.81	52.0	0.90	11.6	4.2	15.7	0.128	1.57	-	ABOVE
13	4	ML	5	0.61	0.61	0.61	3.2	1.25	117	117	1	17	0.976	1.81	52.0	1.02	31.4	4.2	35.6	0.127	LARGE	-	ABOVE
14.8	4	ML	1.8	0.81	0.81	0.81	4.2	1.13	117	117	1	11	0.968	1.81	55.0	1.02	18.4	4.4	22.8	0.126	2.72	-	ABOVE
18	4	ML	3.2	0.96	0.96	0.91	5.0	1.06	117	122.5	1	11	0.962	1.81	55.0	1.14	19.3	4.4	23.6	0.131	2.85	-	NONE
23	4	SM	5	1.22	1.22	1.06	6.2	0.95	132	138.1	2	16	0.952	1.81	48.0	0.95	13.6	3.8	17.5	0.143	1.63	-	NONE
28	4	SM	5	1.55	1.55	1.25	7.8	0.84	132	138.1	1	9	0.941	1.81	48.0	1.14	12.5	3.8	16.3	0.152	1.42	-	NONE
33	4	SM	5	1.88	1.88	1.43	9.3	0.75	132	134	2	26	0.926	1.81	48.0	1.00	18.5	3.8	22.3	0.158	2.17	-	NONE
38	4	SM	5	2.18	2.18	1.61	10.8	0.69	110	134	1	22	0.885	1.81	48.0	1.20	26.3	3.8	30.2	0.156	LARGE	-	NONE
43	4	ML	5	2.48	2.48	1.79	12.3	0.63	132	133.2	2	40	0.844	1.81	55.0	1.00	23.9	4.4	28.2	0.152	LARGE	-	NONE
48	4	CL	5	2.81	2.81	1.96	13.9	0.58	132	133.2	1	16	0.804	1.81	70.0	1.20	16.2	5.6	21.8	0.150	1.88	-	NONE
51.5	4	SM	3.5	3.09	3.09	2.11	15.2	0.40	132	133.2	1	22	0.769	1.81	45.0	1.20	15.4	3.6	19.0	0.146	1.81	-	NONE

Total Settlement 0.0

May be off by 0.1 inches due to rounding

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Checked by SA Date 5/9/22

Project No: 220239
Boring: B-8 and B-10

Dynamic Dry Sand Settlement

g_{cyc} = [(tav)/s'vo]/Gmax = 0.65 (a_{max} /g) s_o r_d / G_{max}

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

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 = 0.362 g Earthq. Mw = 6.21

Elev. Base of Layer (ft)	Elev. Top of Layer (ft)	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. rd	$(N_1)_{60cs}$	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	ML	3	0.5	117	175.5	2	14	0.997	25.3	6.57E-05	0.06		0.00E+00		0.6841	0.0000	0.0000	N/A
8	4	ML	5	1.7	117	643.5	1	6	0.987	18.9	1.37E-04	0.21		0.00E+00		0.6841	0.0000	0.0000	N/A
13	4	ML	5	3.2	117	1228.5	1	17	0.976	42.7	1.43E-04	0.40		0.00E+00		0.6841	0.0000	0.0000	N/A
14.8	4	ML	1.8	4.2	117	1626.3	1	11	0.968	27.1	1.90E-04	0.53		0.00E+00		0.6841	0.0000	0.0000	N/A
18	4	ML	3.2	5.0	117	1918.8	1	11	0.962	28.1	2.02E-04	0.62		0.00E+00		0.6841	0.0000	0.0000	N/A
23	4	SM	5	6.2	132	2436.0	2	16	0.952	**21.3**	**2.47E-04**	**0.79**	**5.00E-04**	**5.00E-02**	**5.00E-02**	**0.6841**	**0.0342**	**0.0684**	**0.0410**
28	4	SM	5	7.8	132	3096.0	1	9	0.941	**20.0**	**2.81E-04**	**1.01**	**6.00E-04**	**6.00E-02**	**7.00E-02**	**0.6841**	**0.0479**	**0.0958**	**0.0575**
33	4	SM	5	9.3	132	3756.0	2	26	0.926	**27.2**	**2.76E-04**	**1.22**	**5.50E-04**	**5.50E-02**	**3.00E-02**	**0.6841**	**0.0205**	**0.0410**	**0.0246**
38	4	SM	5	10.8	110	4361.0	1	22	0.885	**36.6**	**2.57E-04**	**1.42**	**5.20E-04**	**5.20E-02**	**2.50E-02**	**0.6841**	**0.0171**	**0.0342**	**0.0205**
43	4	ML	5	12.3	132	4966.0	2	40	0.844	33.6	2.69E-04	1.61		0.00E+00		0.6841	0.0000	0.0000	N/A
48	4	CL	5	13.9	132	5626.0	1	16	0.804	24.4	3.03E-04	1.83		0.00E+00		0.6841	0.0000	0.0000	N/A
51.5	4	SM	3.5	15.2	132	6187.0	1	22	0.769	**23.4**	**3.09E-04**	**2.01**	**5.00E-04**	**5.00E-02**	**5.00E-02**	**0.6841**	**0.0342**	**0.0684**	**0.0287**
																		Total Settlement	**0.14**

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Dynamic Dry Sand Settlement

g_{cyc} = [(tav)/s'vo]/Gmax = 0.65 (a_{max} /g) s_o r_d / G_{max}

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

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 | Design Ground Water Depth = 14.8 feet | acc. max = 0.362 g | Earthq. Mw = 6.21

Elev. Base of Layer (ft)	Elev. Top of Layer (ft)	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. rd	$(N_1)_{60cs}$	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	ML	3	0.5	117	175.5	2	14	0.997	25.3	6.57E-05	0.06	0.00E+00	0.00E+00	0.00E+00	0.6841	0.0000	0.0000	N/A
8	4	ML	5	1.7	117	643.5	1	6	0.987	18.9	1.37E-04	0.21	0.00E+00	0.00E+00	0.00E+00	0.6841	0.0000	0.0000	N/A
13	4	ML	5	3.2	117	1228.5	1	17	0.976	42.7	1.43E-04	0.40	0.00E+00	0.00E+00	0.00E+00	0.6841	0.0000	0.0000	N/A
14.8	4	ML	1.8	4.2	117	1626.3	1	11	0.968	27.1	1.90E-04	0.53	0.00E+00	0.00E+00	0.00E+00	0.6841	0.0000	0.0000	N/A
18	4	ML	3.2	5.0	117	1918.8	1	11	0.962	28.1	2.02E-04	0.62	0.00E+00	0.00E+00	0.00E+00	0.6841	0.0000	0.0000	N/A
23	4	SM	5	6.2	132	2436.0	2	16	0.952	21.3	2.47E-04	0.79	5.00E-04	5.00E-02	5.00E-02	0.6841	0.0342	0.0684	N/A
28	4	SM	5	7.8	132	3096.0	1	9	0.941	20.0	2.81E-04	1.01	6.00E-04	6.00E-02	7.00E-02	0.6841	0.0479	0.0958	N/A
33	4	SM	5	9.3	132	3756.0	2	26	0.926	27.2	2.76E-04	1.22	5.50E-04	5.50E-02	3.00E-02	0.6841	0.0205	0.0410	N/A
38	4	SM	5	10.8	110	4361.0	1	22	0.885	36.6	2.57E-04	1.42	5.20E-04	5.20E-02	2.50E-02	0.6841	0.0171	0.0342	N/A
43	4	ML	5	12.3	132	4966.0	2	40	0.844	33.6	2.69E-04	1.61	0.00E+00	0.00E+00	0.00E+00	0.6841	0.0000	0.0000	N/A
48	4	CL	5	13.9	132	5626.0	1	16	0.804	24.4	3.03E-04	1.83	0.00E+00	0.00E+00	0.00E+00	0.6841	0.0000	0.0000	N/A
51.5	4	SM	3.5	15.2	132	6187.0	1	22	0.769	23.4	3.09E-04	2.01	5.00E-04	5.00E-02	5.00E-02	0.6841	0.0342	0.0684	N/A
																		Total Settlement	0.00