GONZALEZ ARCHITECTS
7545 N. DEL MAR, SUITE 203 FRESNO, CALIFORNIA 93711 TEL: (559) 497 - 1542 FAX: (559) 497 - 1549

ADDENDUM No. 1

DATE: April 24, 2024

MULTI-USE BUILDING
FAIRMEAD ELEMENTARY SCHOOL

DSA FILE NO. 20-10
DSA APPLICATION NO. 02-121993

CHOWCHILLA ELEMENTARY SCHOOL DISTRICT
CHOWCHILLA, CALIFORNIA

G.A. PROJECT NO. 2318

NOTICE TO ALL CONTRACTORS SUBMITTING BIDS FOR THIS WORK AND TO ALL PLAN HOLDERS:
You are hereby notified of the following changes, clarifications or modifications to the original Contract Documents, Project Manual, Drawings, Specifications and subsequent Addenda. This Addendum shall supersede the original Contact Documents, and previous Addenda wherein it contradicts the same and shall take precedence over anything to the contrary therein. All other conditions remain unchanged.

INDEX OF ADDENDA TRANSMITTED HEREWITH
Addendum Item AD1-A01 thru AD1-A05

AD1-A01: BID DATE REVISION:

Refer to Notice Inviting Bids.

Revise Bid Date from May 9, 2024 **to May 16, 2024.**

AD1-A02: FIRE ALARM SUBSTITUTION:

Refer to Spec. Section 28 31 00 & Sheet E2.1.

FIKE Fire Alarm Products are an acceptable substitution product.

AD1-A03: LANDSCAPE & IRRIGATION:

Refer to Sheet C7 & Spec. Section 02-441 + 02-480.

All Landscaping and irrigation will be provided and installed by the District.

AD1-A04: <u>SWPPP REQUIREMENTS:</u>

Refer to attached Spec. Section 02-300 Storm Water Pollution Prevention Plan (SWPPP).

Provide all material, labor, equipment and services necessary to develop, implement and inspect the Storm Water Pollution Prevention Plans (SWPPP) and obtain the Construction General Permit (CGP).

The Owner will pay all related regulatory fees to secure the SWPPP.

AD1-A05: <u>GEOTECH REPORT:</u>

Refer to Spec. Section 02-200 Earthwork.

Incorporate attached Geotechnical Investigation Report and Geological and Seismic Hazards Assessment prepared by Technicon Engineering Services, Inc.; dated 11/13/2023.

END OF ADDENDUM

STORM WATER POLLUTION PREVENTION PLAN

PART 1 – GENERAL

1.01 SUMMARY: SCOPE OF WORK

1. Contractor to provide all services, material, labor and equipment necessary to prepare SWPPP and obtain SWPPP Permit and implement the Storm Water Pollution Prevention Plan (SWPPP) from Madera County.

2. Provide all services, material, labor and equipment necessary to comply with the conditions of the Construction General Permit (CGP) SWPPP.
The District will pay for Construction General Permit Fee.

3. Implement the Best Management Practices (BMP) contained within the SWPPP or implement other practices deemed necessary by the Contractor/Qualified SWPPP Practitioner (QSP) to better accomplish the intent of controlling the quality of runoff water from the Project Site.

1.02 DEFINITIONS:

A. Acronyms:

BMP	Best Management Practices
CARB	California Air Resources Board
CGP	Construction General Permit Order
CSMP	Construction Site Monitoring Program
EPA	Environmental Protection Agency
NOI	Notice of Intent
NOT	Notice of Termination
NPDES	National Pollution Discharge Elimination System
QSD	Qualified SWPPP Developer
QSP	Qualified SWPPP Practitioner
SJVAPCD	San Joaquin Valley Air Pollution Control District
SWPPP	Storm Water Pollution Prevention Plan
SWRCB	State Water Resources Control Board
RWQCB	Regional Water Quality Control Board

1.03 SUBMITTALS:

A. Submit in accordance with Specification Section 01-300.

1. Storm Water Pollution Prevention Plan (SWPPP) and permit.

2. Reports required by the Storm Water Pollution Prevention Plan (SWPPP).

1.04 QUALITY ASSURANCE:

A. Regulatory Requirements:

1. In accordance with Specification Section - REGULATORY REQUIREMENTS, and the following:

a. CARB Materials and equipment used for this Project shall comply with the current applicable regulations of the California Air Resources Board (CARB) and the Environmental Protection Agency (EPA), in the area where the project is located.

b. EPA Environmental Protection Agency

c. SWRCB State Water Resources Control Board

d. RWQCB Regional Water Quality Control Board

e. SJVAPCD San Joaquin Valley Air Pollution Control District

PART 2 - PRODUCTS

2.01 SOURCE QUALITY CONTROL:

A. Storm Water Pollution Prevention Plan (SWPPP):

1. The Contractor shall prepare the SWPPP and obtain the Construction General Permit Order (CGP).

2. The intent of the CGP is to protect the quality of receiving waters of the United States by limiting the quantity of pollutants in rainfall runoff from construction sites of one acre or more in area. In order to accomplish this goal, each construction project is required to prepare a SWPPP that will govern construction activities to lessen the probability that pollutants will be present in rainfall runoff from their site.

3. This site will be covered by the CGP by the time construction begins.

a. All construction activity must comply with the conditions of the CGP.

b. A NOI to be covered by the CGP will be filed by the Contractor with the SWRCB.

4. The BMPs contained in the SWPPP will meet the intent of the CGP.

a. The Owner does not have any responsibility for selecting or implementing the BMPs proposed by the Contractor and QSP to adequately control the quality of runoff from the site.

b. The Contractor and QSP must provide, implement, and carry out the BMPs that comply with the CGP regardless of the BMPs contained in the SWPPP.

c. The Contractor and QSP shall bear full responsibility for reviewing the proposed BMPs, ascertaining their ability to provide adequate controls, and implementing the BMPs or implementing others deemed by the Contractor and QSP to better accomplish the intent of controlling the quality of runoff water from the project site.

PART 3 - EXECUTION

3.01 APPLICATION:

A. General Requirements:

1. The Contractor shall comply with the conditions of the CGP.

2. Under the terms of this Contract, the Contractor is the Operator/Discharger of the Project Site. It is the Contractor's and QSP's responsibility to faithfully and fully implement the BMPs contained in the SWPPP, and other BMPs as required to effectively control the quality of runoff water from the project site.

3. The Contractor shall fully and completely carry out all provisions of the SWPPP and insure that all of the Contractor's forces, including sub-contractors, on the site do the same. The Contractor shall assume full responsibility for the implementation, maintenance and execution of the SWPPP for the life of this project. The Contractor shall be fully liable for penalties, fines, and clean-up costs resulting from the failure of the Contractor's personnel or subcontractor's personnel to comply with the provisions of the SWPPP, and hold the Owner/LRP harmless from the Contractor's failure to implement the SWPPP as required by the SWRCB,RWQCB, CGP, and the local authority having jurisdiction.

4. The Contractor shall be fully aware of the requirements for the full execution of the SWPPP which are contained in the previously mentioned regulations, the requirements of these specifications for implementing, maintaining, and enforcing the provisions of the SWPPP and the impact that the SWPPP will have on the operation, prosecution and cost of the work.

B. Best Management Practices (BMPs):

1. The Contractor's QSP shall conduct inspections weekly and at least once each 24-hour period during extended storm events, to identify and record BMPs that need installation or maintenance to operate effectively. Should the QSP deem the BMPs proposed in the SWPPP are inadequate to meet the requirements of the CGP, or a change occurs in the nature or manner of construction operations not anticipated in the SWPPP, the QSP shall propose alternative BMPs that are equal to or better than those contained in the SWPPP.

3.02 FIELD QUALITY CONTROL:

A. Monitoring of BMPs

1. Monitoring by Contractor's QSP

a. Implement the CSMP (weekly, pre-storm, storm event, post-storm, quarterly inspections) as required by the CGP.
b. Prepare and submit all reports to Owner/LRP and SWRCB as required by the SWPPP and the CGP.

2. The Contractor shall keep a minimum of one copy of the SWPPP and Addenda thereto in the following locations for public inspection:

   a. Contractor's Project Site Field Office.
   b. Contractor's General Business Office.

3.03 CLEANING AND REMOVAL:

A. Completely remove from the Project Site all materials used to construct and maintain the temporary BMPs upon completion and acceptance of the Project.

3.04 RECORD KEEPING:

A. Paper or electronic records of all CSMP inspections, testing, and training reports, including the Annual Report, shall be retained for a period of at least three years. These records shall be available at the project site until construction is completed.

END OF SECTION

November 13, 2023 TES No.230566.001

Mr. Douglas Collins
Chowchilla Elementary School District
355 North Fifth Street
Chowchilla, California 93610
Email: collinsd@chowkids.com

c/o Mr. Art Lopez
Brooks Ransom & Associates
7415 N. Palm Avenue Suite 100
Fresno, California 93711
Email: art@brooksransom.com

Project: Proposed Multi-Purpose Building
Fairmead Elementary School
19421 Avenue 22 ¾
Chowchilla, California

Subject: Geotechnical Investigation and Geologic-Seismic Hazards Evaluation Report

Dear Mr. Collins:

The enclosed report presents the results of a geotechnical investigation and geologic-seismic hazards evaluation for the proposed Multi-Purpose Building in Chowchilla, 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 Chowchilla Elementary 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.

Adam AhTye, PE
Senior Project Engineer

Salvador Alvarez, PE, GE
Geotechnical Engineering Manager

AA:SA:vm

TECHNICON
ENGINEERING SERVICES, INC.

GEOTECHNICAL INVESTIGATION AND GEOLOGIC-SEISMIC HAZARDS EVALUATION REPORT PROPOSED MULTI-PURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL 19421 AVENUE 22 ¾ CHOWCHILLA, CALIFORNIA

Prepared for:

Chowchilla Elementary School District
355 North Fifth Street
Chowchilla, California 93610

November 13, 2023

TES No. 230566.001

GEOTECHNICAL & ENVIRONMENTAL ENGINEERING – CONSTRUCTION TESTING & INSPECTION

TECHNICON
ENGINEERING SERVICES, INC.

1989 ♦ 2022 33 YEARS STRONG

GEOTECHNICAL & ENVIRONMENTAL ENGINEERING – CONSTRUCTION TESTING & INSPECTION

Prepared For:

Chowchilla elementary School District
355 North Fifth Street
Chowchilla, California 93610

GEOTECHNICAL INVESTIGATION AND GEOLOGIC-SEISMIC HAZARDS EVALUATION REPORT
PROPOSED MULTI-PURPOSE BUILDING
FAIRMEAD ELEMENTARY SCHOOL
19421 AVENUE 22 ¾
CHOWCHILLA, CALIFORNIA

TECHNICON PROJECT
TES No. 230566.001

Prepared by:

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CERTIFIED ENGINEERING GEOLOGIST FREDERICK A. MASON No. 2660 Exp. 10/25 STATE OF CALIFORNIA

TECHNICON Engineering Services, Inc.
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November 13, 2023

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GEOTECHNICAL INVESTIGATION AND GEOLOGIC-SEISMIC HAZARDS EVALUATION REPORT PROPOSED MULTI-PURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL 19421 AVENUE 22 ¾ CHOWCHILLA, CALIFORNIA

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1 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation for the proposed Multi-Purpose Building to be constructed at the Fairmead Elementary School located at 19421 Avenue 22 ¾ in Chowchilla, California. The purpose of the investigation was to explore and evaluate the subsurface conditions at the site to develop geotechnical recommendations for project design and construction.

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

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

1.2 LOCATION

The project is located in northwestern Madera County, at 19421 Avenue 22 ¾ in Chowchilla, California. Based on the Berenda, California 7 ½-minute quadrangle topographic map, the site lies within the central area of Section 11, R16E and T10S. The elevation of the site is approximately 251 feet above the Mean Sea Level. Based on the USGS 7½-minute topographic map, the site coordinates are approximately:

Latitude:	37.0819° N
Longitude:	120.1941° W

1.3 PROPOSED CONSTRUCTION

Based on the site plan provided, it is understood that the project involves the design and construction of a new multi-purpose building. The proposed building will consist of an approximately 13,000 square-foot, single-story building supported on shallow concrete foundations and slab-on-grade floors. Maximum wall and column loads are estimated to be less than 3 kips per foot and 30 kips, respectively. Appurtenant improvements will include underground utilities, concrete flatwork, and landscaping. The site appears to be relatively flat, therefore we estimate that cut and fill depths will be less than 2 feet.

1.4 PURPOSE AND SCOPE OF SERVICES

The purpose of the investigation and evaluation was to explore the site subsurface conditions and evaluate pertinent geologic and seismic data to develop recommendations and opinions to aid in project design, approval and construction. The scope of services consisted of a field exploration program, laboratory testing, design analysis, and preparation of this written report as described in **TECHNICON** proposal, dated August 16, 2023 (TES No. GP23-167). This Geotechnical Investigation and Geologic-Seismic Hazards Evaluation Report includes the following:

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

- Design factors for earth retaining structures;
- Design of concrete slabs-on-grade for buildings, including modulus of subgrade reaction;
- 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 September 22, 2023 consisted of drilling three (3) exploratory test borings, and a site reconnaissance by a staff engineer. The test borings were drilled with a SIMCO 2800 truck-mounted drill rig using 4-inch diameter solid flight auger drilling techniques and extended to depths of 16.5, 21.5 and 51.5 feet below existing ground surface (bgs). The approximate locations of the test borings 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)

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 Great Valley is between the Sierra Nevada geomorphic province to the east, and the Coast Rang 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 ago.

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

3.2 AREA AND SITE GEOLOGY

The geology at the site is mapped as Pleistocene aged Riverbank formation (Qr), described as alluvium deposits. The soil subgrade characteristics encountered during the field investigation (i.e. soil type, blow count, etc.) are representative of these sediments. Figure 4 presents a site-specific geologic map of the project.

3.3 SURFACE CONDITIONS

At the time of investigation, the project site consisted of a vacant, landscaped grass play field. The site is generally bounded by existing residences to the north and south, agricultural land and orchards to the east, and the existing Fairmead Elementary School to the west. The overall site topography is relatively flat and approximately level to the surrounding elevation.

3.4 EARTH MATERIALS

The subsurface soils consist of Pleistocene aged Riverbank Formation (Qr). The earth material encountered by the subsurface exploration consisted of clayey sand in the upper 4 to 9 feet, underlain by laterally discontinuous layers of sandy clay, sandy silt, silty sand, and poorly graded sand extending to the maximum depth explored, 51.5 feet bgs. The granular soils generally had a relative density of medium dense to very dense and the fine grained soils had a consistency of 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 section on Figure 5.

3.5 GROUNDWATER CONDITIONS

Groundwater was not encountered within the borings to a depth of 51.5 feet bgs. The California Department of Water Resources “Sustainable Groundwater Management Agency Data Viewer” Spring 2023, indicates the current groundwater depth in the area exceeds approximately 100 feet bgs. Research utilizing the California Department of Water Resources (DWR) website shows a nearby well with recorded data to be approximately 2.2 miles to the west of the project site (Well No. 10S16E09E001M). Based on the groundwater elevation data collected at this well measurements historic high groundwater recorded in 1959 was approximately 49 feet bgs.

Considering the measured groundwater a design groundwater depth of 49 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 1959.

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

4 FAULTING AND SEISMICITY

4.1 HISTORICAL SEISMICITY

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

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

TABLE 4.1-1

SIGNIFICANT REGIONAL EARTHQUAKE EVENTS

Earthquake Name	Year	Distance from Site (miles)	Magnitude (Mw)
Coalinga	1983	59	6.4
Great Fort Tejon	1857	93	7.9
Owens Valley	1872	118	6.5
Ridgecrest	2019	169	7.1

Epicenters of significant earthquakes ($M \geq 5.5$) within the vicinity of the site are shown on Figure 6. Data for earthquakes that occurred from 1800 to 2022 have been obtained from the Significant California Earthquakes website (CGS, 2019) and a composite catalog by the ANSS. The ANSS catalog is a worldwide earthquake catalog which is created by merging the master earthquake catalogs from contributing ANSS member networks and then removing duplicate events, or non-unique solutions from the same event. The ANSS network includes the Northern and Southern California Seismic Networks, the Pacific Northwest Seismic Network, the University of Nevada, Reno Seismic Network, the University of Utah Seismographic Stations, and the United States National Earthquake Information Service. The earthquake database also consists of earthquake records between 1800 and 1900 from Seeburger and Bolt (1976) and 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 active and late Quaternary faults in the area with respect to the subject site are shown on Figure 7, 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	32	7.1
Ortigalita	Right Lateral/ Strike Slip	40	7.1
Calaveras	Strike Slip	59	7.0
San Andreas	Right Lateral/ Strike Slip	63	8.0

4.3 SITE CLASS

Based on the field exploration, the site soil is classified as Site Class D as presented in ASCE 7-16 based on the average Standard Penetration Tests (N value) at the project site. Site Class D is defined as a stiff soil profile with shear wave velocities between 600 feet/sec and 1,200 feet/sec, or Standard Penetration Resistance (N) between 15 to 50 blows/foot, or undrained shear strength (S_u) between 1,000 to 2,000 psf for the upper 100 feet.

4.4 GENERAL PROCEDURE SEISMIC DESIGN CRITERIA

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

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

Seismic Item	Design Value	Seismic Item	Design Value
Site Class	D	Seismic Design Category	D
S_S	0.571	S_{MS}	0.767
S_1	0.230	S_{M1}	0.492
Site Coefficient, F_v	2.140*	S_{DS}	0.511
Site Coefficient, F_a	1.343	S_{D1}	0.328
T_S	0.642		

*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 0.348g and a weighted magnitude of Mw = 6.23.

4.5 SITE SPECIFIC SEISMIC DESIGN CRITERIA

In accordance with ASCE 7-16 11.4.8, since the project is in a site class D and the S_1 value is greater than 0.2 (0.230g) a site-specific ground motion hazard analysis was performed. The analysis followed the requirements of ASCE 7-16, Sections 21.2 through 21.5, as well as ASCE 7-16, Supplement No. 1 and No. 3, and 2022 CBC 1830A.6.

The following steps were utilized for determining the site-specific ground motion parameters: Seismic design parameters were obtained for the project site utilizing a Site Class D, and site coordinates from the Structural Engineers Association of California (SEAOC) website (http://seismicmaps.org). The USGS Unified Hazard Tool and the Risk-Targeted Ground Motion calculator was used to calculate the probabilistic ground motion response spectrum in accordance with ASCE 7-16 Section 21.2.1.2 Method 2. The 2014 NGA West2 – GMPEs worksheet from the Pacific Earthquake Engineering Research Center was then used to calculate deterministic spectral response acceleration as an 84th-percentile 5% damped spectral response acceleration in the maximum horizontal direction by using fault parameters and magnitude area relationships given by the USGS Unified Hazard Tool in accordance with ASCE 7-16 Section 21.2.2. Supplement No. 3 indicates that projects located in Site Class D should increase S_{M1} by 50 percent in Equation 11.4-2. This increase results in a 50 percent increase of S_{D1} in Equation 11.4-4. These increased values are to be used for all applications and formulation of the design response spectrum. The Site-Specific MCE_R was then calculated by a single factor such that the maximum response spectral acceleration equals $1.5F_a$, with F_a determined using Table 11.4.1 in the ASCE 7-16. In accordance with ASCE 7-16 Section 21.3, the design spectral response had to be checked that no period shall be taken as less than 80% of S_a determined in accordance with Section 11.4.6, where F_a is determined using Table 11.4.1 and F_V is taken as 2.4 for $S_1 < 0.2$ or 2.5 for S_1 > or equal to 0.2. After checking design spectrum is greater than 80% of code-based spectrum for all periods, using the design spectrum graph, design acceleration parameters such as S_{DS} is taken as 90% of max S_a between periods T=0.2 and 5 seconds and parameter S_{D1} taken as the maximum value of the product, TS_a, for periods from 1 to 5 seconds for sites with $V_s < 365.76$ m/s in accordance with ASCE 7-16 Section 21.4. The parameters S_{MS} and S_{M1} are then taken as 1.5 times S_{DS} and S_{D1}, respectively. Lastly, the maximum considered earthquake geometric mean peak ground acceleration is taken by comparing deterministic peak ground acceleration from 84th spectral acceleration at T=0.01 seconds to $0.5F_{PGA}$, following with the greater of those two values being compared to the probabilistic peak ground acceleration, with the lesser of the two values being the site-specific peak ground acceleration (0.348) in accordance with ASCE 7-16, Section 21.5. Based on this analysis, a peak ground acceleration of 0.348g is recommended for the evaluation of liquefaction. The site-specific ground motion analysis is attached.

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

Seismic Item	Design Value	Seismic Item	Design Value
Site Class	D	Seismic Design Category	D
S_S	0.571	S_{MS}	0.898
S_1	0.230	S_{M1}	0.698
Site Coefficient, F_v	2.500	S_{DS}	0.599
Site Coefficient, F_a	1.343	S_{D1}	0.465
T_S	1.125		

5 GEOLOGIC AND SEISMIC HAZARDS

5.1 GENERAL

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

5.2 SURFACE FAULT RUPTURE

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

5.3 SEISMICALLY INDUCED GROUND FAILURE

5.3.1 Liquefaction

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

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

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

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

Youd et. al. (2001) and considered the relative density and fines content of the granular sediments. The analysis considered a historical high design groundwater depth of 49 feet bgs and measured groundwater depth of greater than 50 feet bgs, ground acceleration (PGA_M) of 0.348g, and earthquake moment magnitude, Mw = 6.23

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

According to the City of Chowchilla General Plan (CGP, 2040), given the relatively level topography of the Central Valley and the project site, the risk for ground failure and landslides within the 2040 General Plan Planning Area is extremely remote and limited to areas outside of the project site.

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 CGP indicates that earthquake-induced seiches are not considered a risk in the City of Chowchilla.

5.4.2 Potential for Inundation Due to Dam Failure

According to the Chowchilla General Plan, two dams could cause substantial flooding in the City of Chowchilla in the event of a failure: Buchanan Dam and Berenda Slough Dam. Therefore, mitigation measures such as preparing an emergency evacuation plan and route are recommended.

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 06039C0900E, dated September 26, 2008) indicating the area is determined to be outside the 0.2 percent annual chance floodplain. The civil engineer should plan site grades accordingly.

5.5 EXPANSIVE SOILS

One (1) Expansion Index (EI) test was performed on a soil sample collected from the near surface soils of the site. The test indicated the near surface soils have a very low potential for expansion as indicated by an EI of 11. These soils are not susceptible to volume changes associated with changes in soil moisture content. The potential for future differential movement of structures resulting from these soils is negligible.

5.6 HYDROCOMPACTION (SOIL COLLAPSE)

Our experience has found that some of the alluvial soils in the San Joaquin Valley are subject to hydrocompaction. Hydrocompactive soil has a relatively loose skeletal structure, which is weakly cemented by soluble salts or a slight clay mineral content. Moisture increase breaks down the inter-particle cementation causing a collapse of the skeletal structure. The significant loss in soil volume can result in settlement of overlying structures. The geotechnical exploration and laboratory testing identified that hydrocompactive characteristics were minimum. Based on the laboratory testing, post saturation of soil samples obtained from the site indicated moderate collapse potential upon inundation. Analysis indicates that settlement due to hydrocomapction is negligible; therefore no mitigation is required.

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 CGP does not identify subsidence within the proposed project area. 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 stadium improvements as currently envisioned. The use of spread and continuous reinforced concrete footings bearing on engineered fill are considered appropriate for structure support provided that the recommendations presented in this report are incorporated into the project design and construction.

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

6.2 SITE PREPARATION

6.2.1 Stripping

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

6.2.2 Disturbed Soil, Undocumented Fill and Subsurface Obstructions

Initial site grading should include a reasonable search to locate disturbed soil, undocumented fill soils, debris, abandoned underground structures, and/or existing utilities that may exist within the area of construction. All underground utilities should be rerouted beyond the perimeter of the proposed improvements and all previous trench backfill and any loose soils generated by the utility removal should be removed to expose undisturbed native soil. If any areas or pockets of soft or loose soils or void spaces made by burrowing animals, undocumented fill, or other disturbed soil are encountered, they should be excavated to expose approved undisturbed native soil. Excavations for removal of the above items should be dish-shaped and backfilled with engineered fill (see Section 6.3).

6.2.3 Over-Excavation

After performing the removals described in Sections 6.2.1 and 6.2.2, the proposed project site should be over-excavated a minimum depth of 18 inches below existing ground surface to remove any loose soils from the project area. The bottom of the excavation should be processed in accordance with Section 6.2.4 and the scarified soil should be recompacted according to Table 6.3-2. The lateral limits of the over-excavation should extend at least 5 feet beyond the perimeter of the proposed improvements.

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 12 inches below exposed subgrade elevation. The subgrade soil should be uniformly moisture conditioned to at least optimum moisture, proof rolled to detect soft or pliant areas, and compacted to the requirements for engineered fill. Soft or pliant areas should be mitigated in accordance with Sections 6.2.2.

6.2.5 Construction Considerations

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

6.3 ENGINEERED FILL

6.3.1 Materials

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

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

TABLE 6.3-1
IMPORT FILL CRITERIA

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

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

6.4 TEMPORARY EXCAVATIONS

6.4.1 General

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

6.4.2 Excavations and Slopes

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

6.4.3 Construction Considerations

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

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

TRENCH BACKFILL

6.4.4 Materials

Pipe zone backfill (i.e., material beneath and in the immediate vicinity of the pipe), should consist of soil compatible with design requirements for the specific types of pipes. It is recommended 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. It should be noted that the native clayey material may require significant effort to achieve compaction within narrow trenches. If granular import is used for backfill, a native clay soil or lean concrete slurry dike should be provided in the upper 4 feet where trenches cross beneath the perimeter of the structures. This dike is intended to minimize the lateral migration of subsurface water into clay soil under the buildings. If granular import material is used for pipe or trench zone backfill, it should have a piping ratio compatible with the adjacent soil, or a geofabric separator should be utilized.

6.4.5 Compaction Criteria

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

7 DESIGN RECOMMENDATION

7.1 GENERAL

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

7.2 SPREAD FOOTINGS

7.2.1 Vertical Bearing Pressures and Settlements – Strip and Spread Foundations

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

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

TABLE 7.2-1
BEARING CAPACITY

	Bearing Capacity (psf)
Static Loading	535 B + 1,140 D
Total Combined Loading	805 B + 1,710 D
Unfactored Ultimate Bearing	1,610 B + 3,420 D

Note: 1) B is the footing width (ft), D is the footing depth (ft)

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.

The foundation soil is anticipated to have a low expansive potential. Therefore, foundation embedment and reinforcement should be consistent with structural or architectural considerations and the 2022 CBC.

Analysis, based on methods by Schmertmann, determined the following estimated static settlement based on a range of assumed design bearing and estimated structural loads. Settlement is expected to occur rapidly with load application. 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	3 kips/ft	1,500	0.25
Square	30 kips	1,500	0.30

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	0.42	0.50	0.62
Passive Pressure (psf/ft)	340	455	680
Lateral Translation Needed to Develop Passive Pressure	0.004 D	0.007 D	0.018 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)	41
At-Rest Pressure (psf/ft of depth)	63
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 frictional resistance given in Table 7.2-3 and the bearing capacities given in Table 7.2-1. When utilizing the bearing capacities of Table 7.2-1, the static loading value represents the average bearing for the footing and the total combined loading value presents the allowable maximum toe pressure.

7.4 SLABS-ON-GRADE

7.4.1 Subgrade Preparation

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

7.4.2 Capillary and Moisture/Vapor Break

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

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

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

According to American Concrete Institute (ACI) 302.2R-06, concrete could be placed directly on the vapor retarding membrane to minimize the potential for developing a reservoir of moisture in the sand layer, which could lead to future moisture entrapment and potential moisture and flooring problems. If concrete is placed directly on the membrane, care should be taken to not damage the membrane and special concrete curing methods implemented to minimize potential slab curing problems. If the protective sand layer is not used, the building designer should be in agreement. Many slab designers feel the sand cushion is important to proper concrete curing as well as minimizing slab curling issues.

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

7.4.3 Conventional Slab Design

There are no geotechnical considerations (e.g., expansive soil), which would require special design of slabs. Therefore, the thickness and reinforcement of slabs-on-grade should be determined by structural considerations and should be designed by the project structural engineer or building designer. A modulus of subgrade reaction, K_p (B_p = 1 foot), of 250 pci may be used for elastic analysis of slabs on properly compacted subgrade. Slab concrete should have good density, a low water/cement ratio, and proper curing to promote a low porosity and reduce moisture vapor transmission.

7.5 PIER FOUNDATIONS

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

TABLE 7.5-1
ALLOWABLE AXIAL CAPACITY

	Frictional Resistance for Vertical Loads in Compression (lbs)
Static Loading	60 DL[2]
Total Combined Loading	80 DL[2]
Unfactored Ultimate Capacity	120 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 195 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

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

The pH of the soil tested was 6.66 and the minimum electrical resistivity was 3,302 ohm-cm. These values are generally representative of an environment that could be moderately corrosive to buried unprotected metals. Utilizing methods provided in Caltrans California Test 643, “Method for Estimating the Service Life of Steel Culverts”, an 18-gauge steel zinc-coated culvert is estimated to have a maintenance-free service life (years to perforation) exceeding 20 years. Therefore, if project improvements will involve metal that comes into contact with the on-site soil, the design should consider this potential soil corrosiveness described.

Test results suggest that low levels of soluble sulfates (6 ppm) and low levels of soluble chlorides (< 5 ppm) are present in on-site soils. Normal cement (Type II) and normal reinforcement cover should be adequate for foundation concrete that comes in contact with the foundation soils.

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

7.7 SITE DRAINAGE

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

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

The maintenance personnel must maintain the established drainage by not blocking or obstructing gradients away from structures without providing some alternative drainage means (e.g., area drains and subsurface pipes). If planter or landscape areas are established near the structures, it is important to prevent surface run-off from entering the planter and care must be taken not to over irrigate and to maintain a leak-free sprinkler piping system. Consideration should be given to use of low volume emitter irrigation systems for planters. Well-maintained low-volume emitter irrigation (drip system) is best suited for planters adjacent to structures. Watering practices must strive to use only sufficient water to sustain and promote plant growth.

8 ADDITIONAL SERVICES

8.1 DESIGN REVIEW AND CONSULTATION

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

8.2 CONSTRUCTION OBSERVATION AND TESTING

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

9 LIMITATIONS

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

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

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

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

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

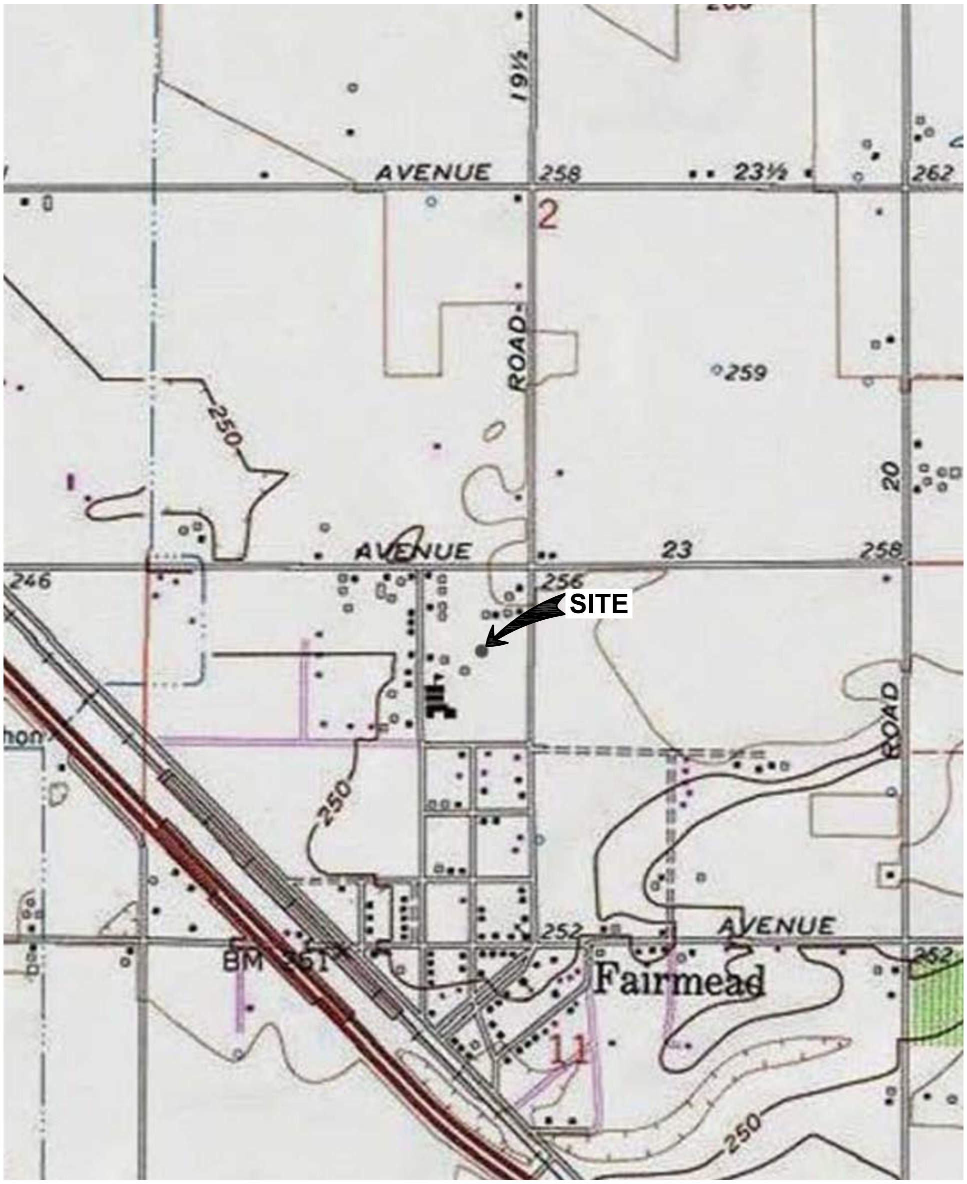
10 REFERENCES

- ASCE 7 Hazard Tool, http://asce7hazardtool.online/
- California Building Code, (2022), Vol. 2, California Building Standards Commission.
- California Department of Transportation, California Test Method No. 643, Method for Estimated the Service Life of Steel Culverts (1999)
- California Department of Water Resources, Sustainable Groundwater Management Agency Data Viewer (Spring 2021)
- California Department of Water Resources (http://www.water.ca.gov/waterdatalibrary/)
- Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J. (2003), The Revised 2002 California Probabilistic Seismic Hazards Maps, California Geological Survey, June 2003
- CGS, Alguist-Priolo Earthquake Fault Zone (2000)
- CGS, (1986), Guidelines to Geologic/Seismic Reports: Note 42
- CGS, (1986), Guidelines for Preparing Engineering Geologic Reports: Note 44
- CGS (2002), California Geomorphic Provinces: Note 36
- CGS, (2022), Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings: CGS Note 48
- CGS, Fault Activity Map of California (2010), Compiled by Charles W. Jennings, William A. Bryant, George Saucedo
- CGS, Fault-Rupture Hazard Zones in California, Special Publication 42
- CGS, (2008), Guidelines for Evaluating and Mitigating Seismic Hazards, Special Publication 117A
- CGS, (2019), Significant California Earthquakes
- City of Chowchilla County General Plan (2040)
- Croft, M.G., 1972, Subsurface Geology of the late Tertiary and Quaternary Water Bearing Deposits of the Southern part of the San Joaquin Valley, California, USGS Water – Supply Paper 1999 – H
- Federal Emergency Management Agency, Flood Insurance Rate Maps, Madera County, California: No. 06039C0900E (September 26, 2008)
- Field, E.H., and 2014 Working Group on California Earthquake Probabilities, 2015, UCERF3: A new earthquake forecast for California's complex fault system: U.S. Geological Survey 2015-3009, 6 p., https://dx.doi.org/10.3133/fs20153009
- Seed, el al. 2003, Recent Advances in Soil Liquefaction Engineering, A Unified and Consistent Framework
- Seeburger and Bolt (1976)
- Structural Engineers Association of California "Seismic Design Maps", http://seismicmaps.org

- Toppozada et al. (1978, 1981)
- USGS, (2015), Berenda, Quadrangle 7½-Minute Series (Topographic)
- 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

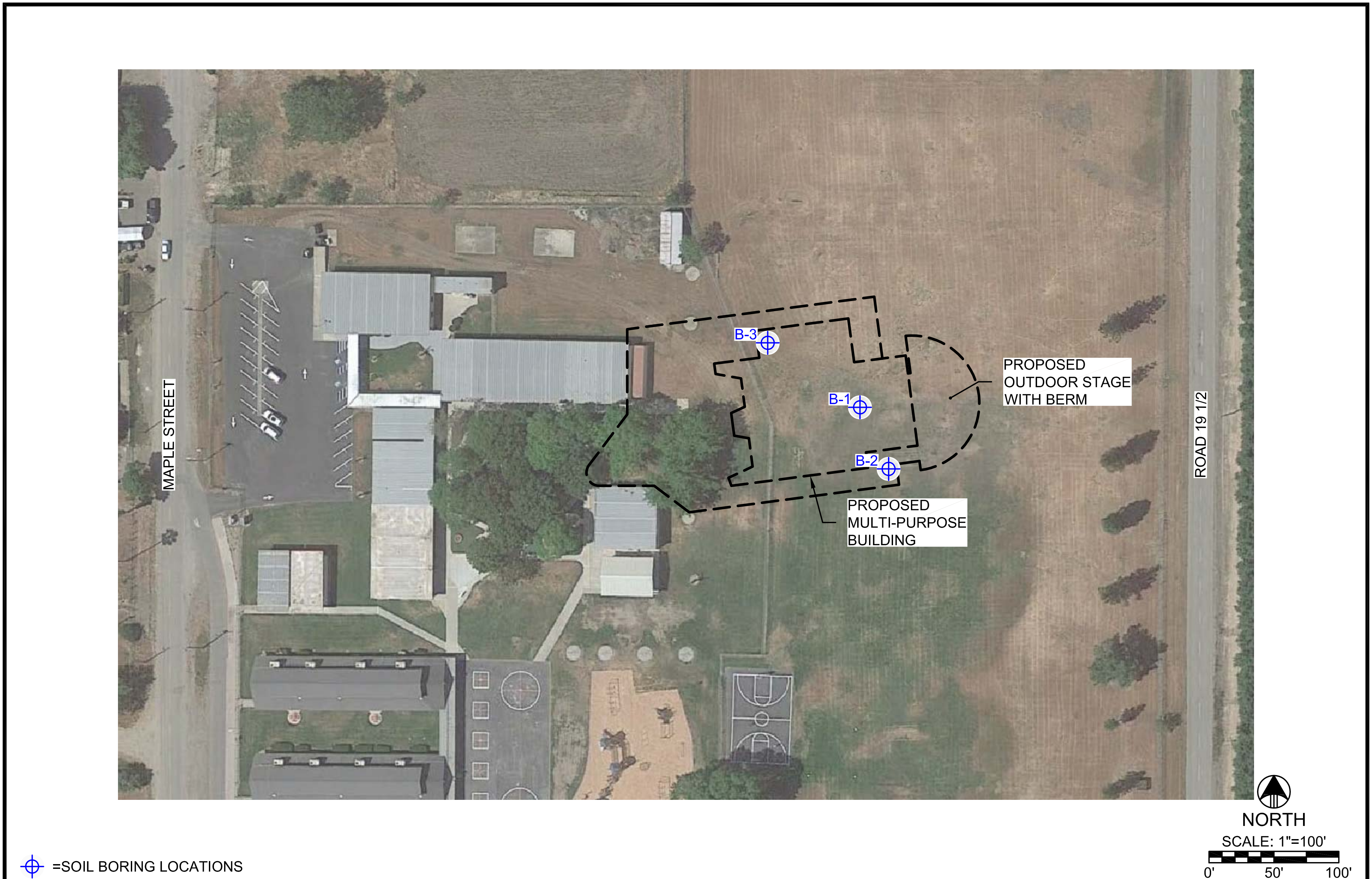
1 through 7



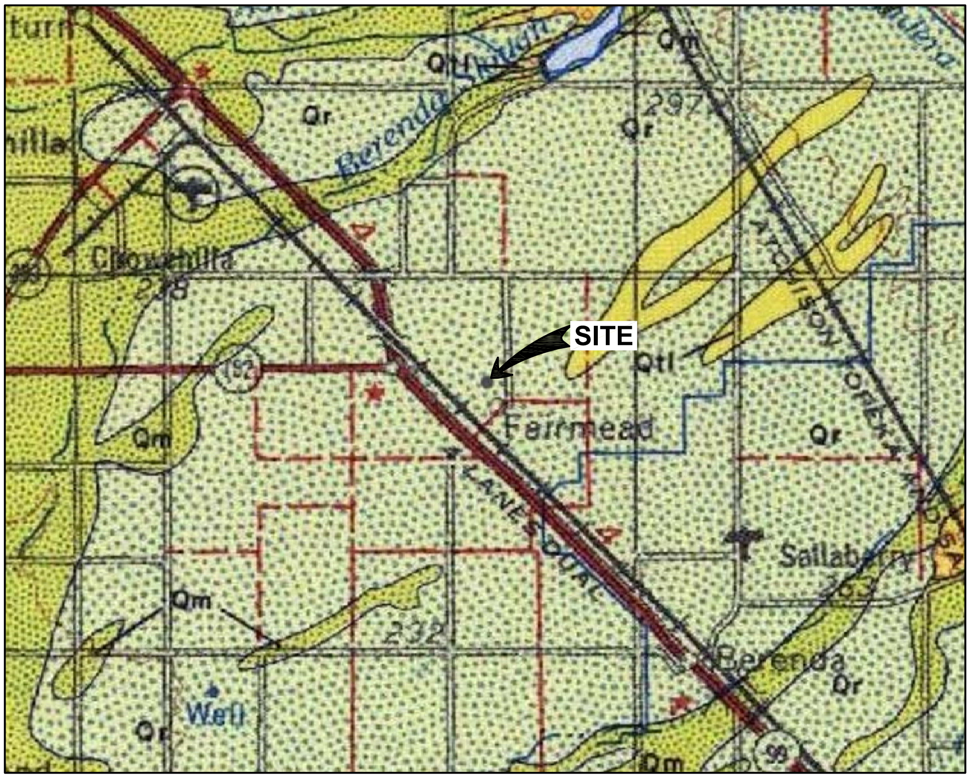
LAT.: 37.0819°N, LONG.: 120.1941°W, 11-T10S-R16E, MDB&M

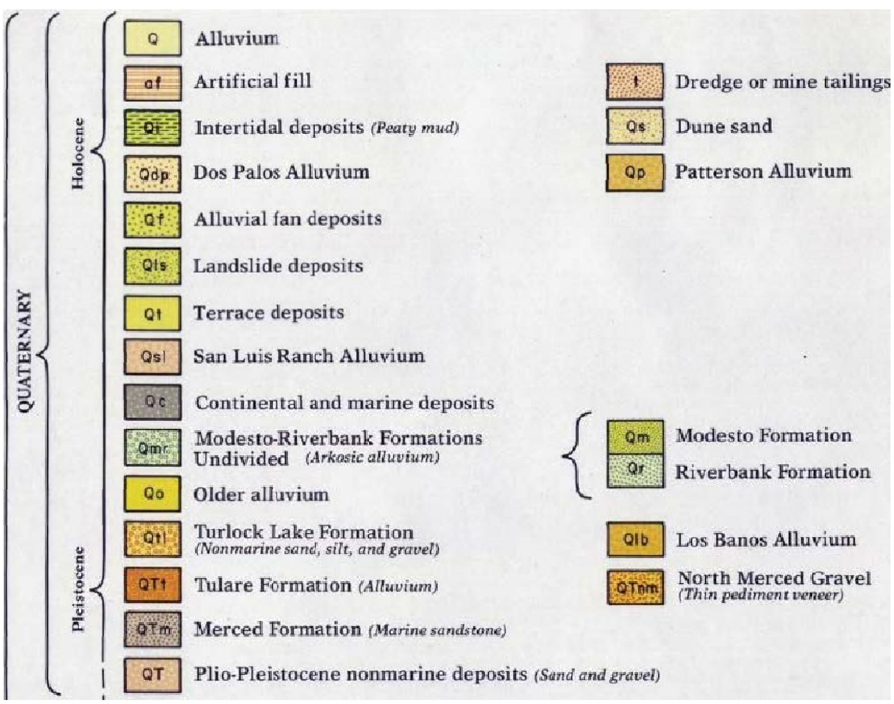
NORTH

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 230556 SOURCE: USGS TOPOGRAPHIC MAPS	VICINITY MAP PROPOSED MULTI-PURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL 19421 AVENUE 22 3/4 CHOWCHILLA, CALIFORNIA	FIGURE 1 NTS



TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 230556	DATE: 10/5/23	SITE MAP PROPOSED MULTI-PURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL 19421 AVENUE 22 3/4 CHOWCHILLA, CALIFORNIA	FIGURE 2
	SOURCE: GOOGLE EARTH	APPROVED BY: AA		

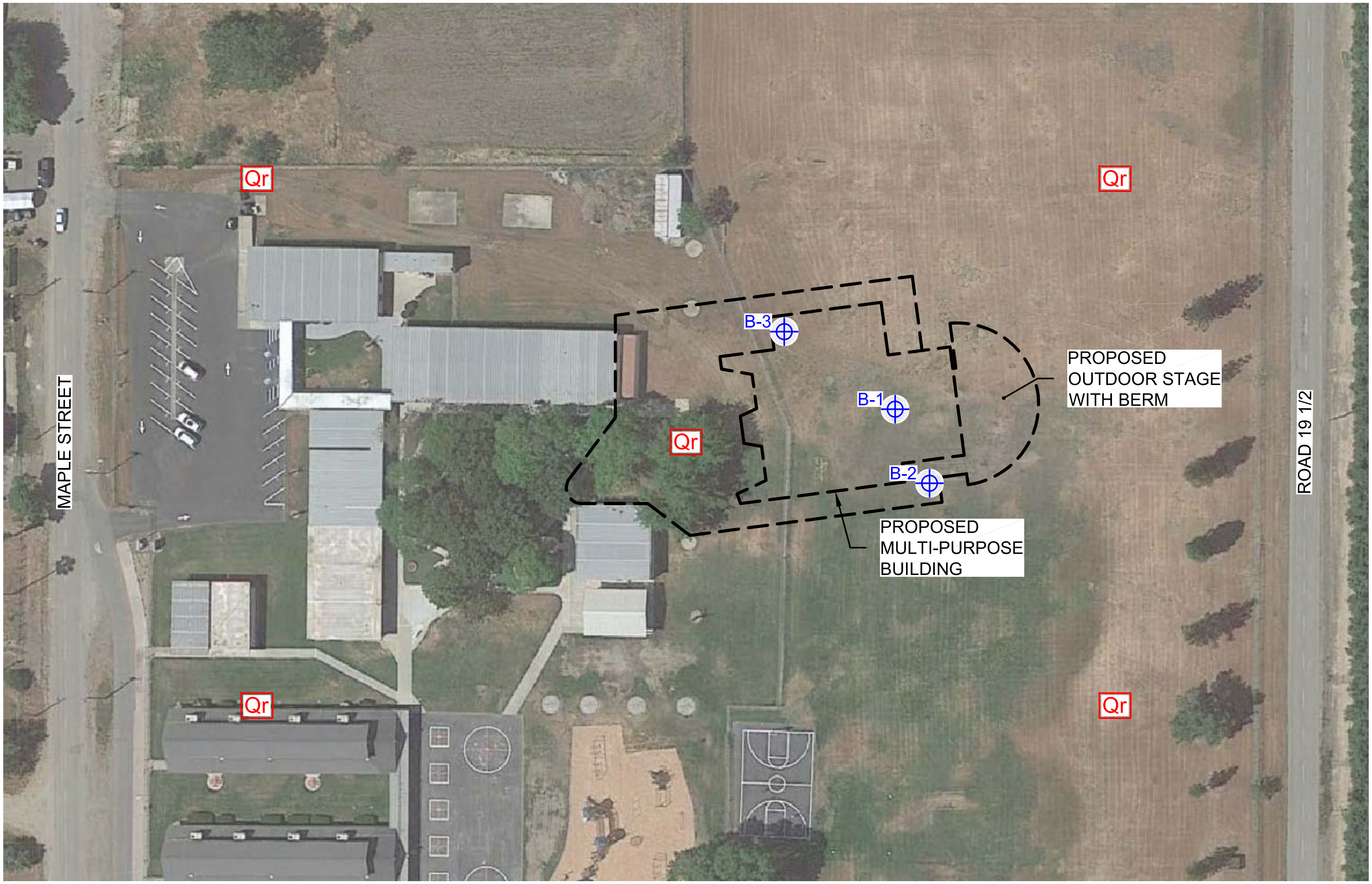




NORTH

GEOLOGIC MAP OF THE SAN FRANCISCO - SAN JOSE QUADRANGLE, CALIFORNIA, SCALE 1:250,000 - 1991

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 230556 SOURCE: DIVISION OF MINES AND GEOLOGY	REGIONAL GEOLOGIC MAP PROPOSED MULTI-PURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL 19421 AVENUE 22 3/4 CHOWCHILLA, CALIFORNIA	FIGURE 3 NTS

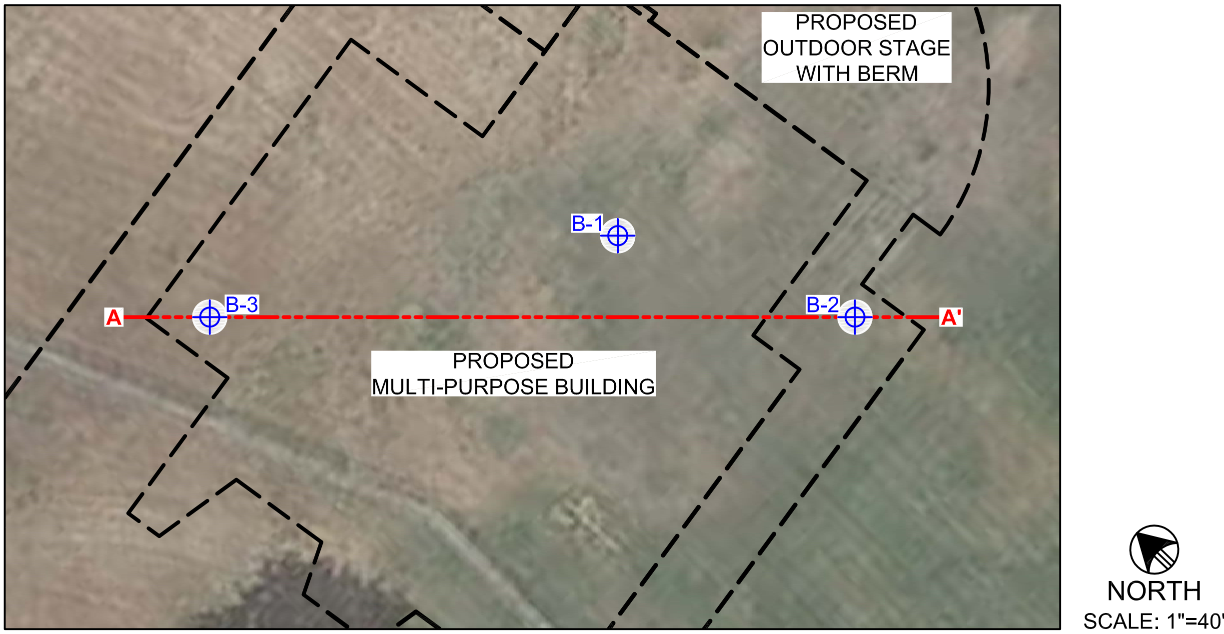


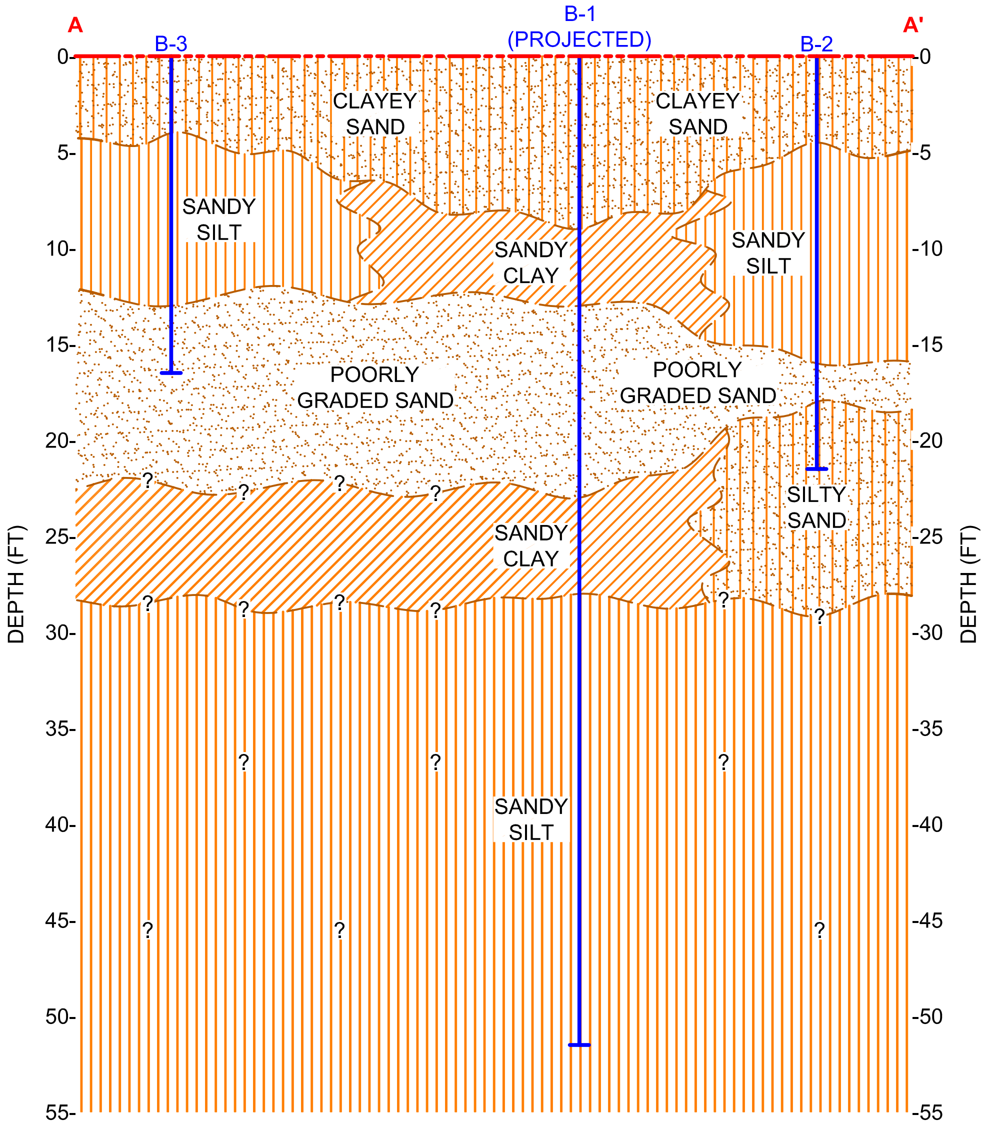
Qr =RIVERBANK FORMATION

=SOIL BORING LOCATIONS

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

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 230556	DATE: 10/31/23	GEOLOGIC MAP OF SITE PROPOSED MULTI-PURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL 19421 AVENUE 22 3/4 CHOWCHILLA, CALIFORNIA	FIGURE 4
	SOURCE: GOOGLE EARTH	APPROVED BY: AA		





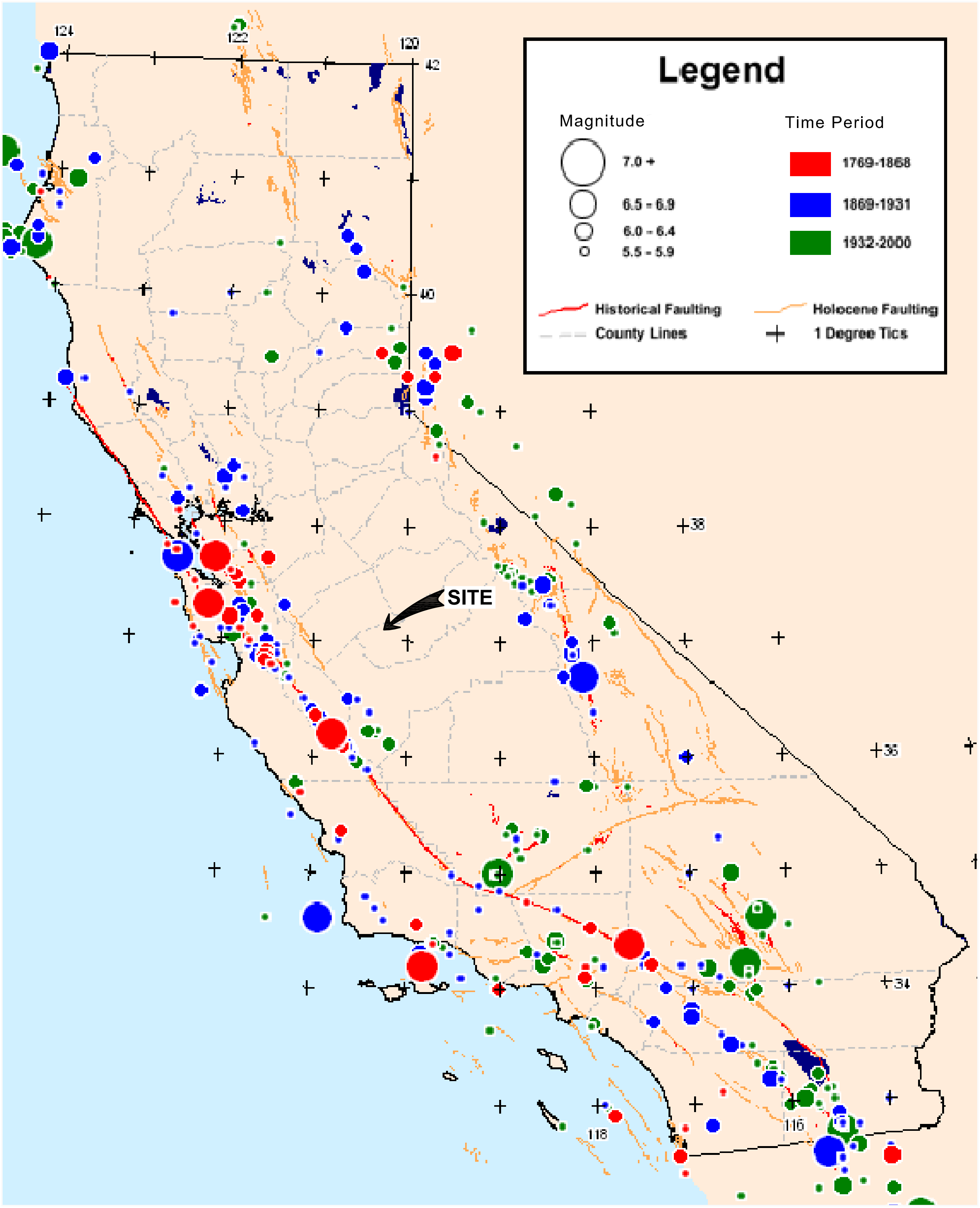
TECHNICON
ENGINEERING SERVICES, INC.

PROJECT:
230556

CAD BY:
MH

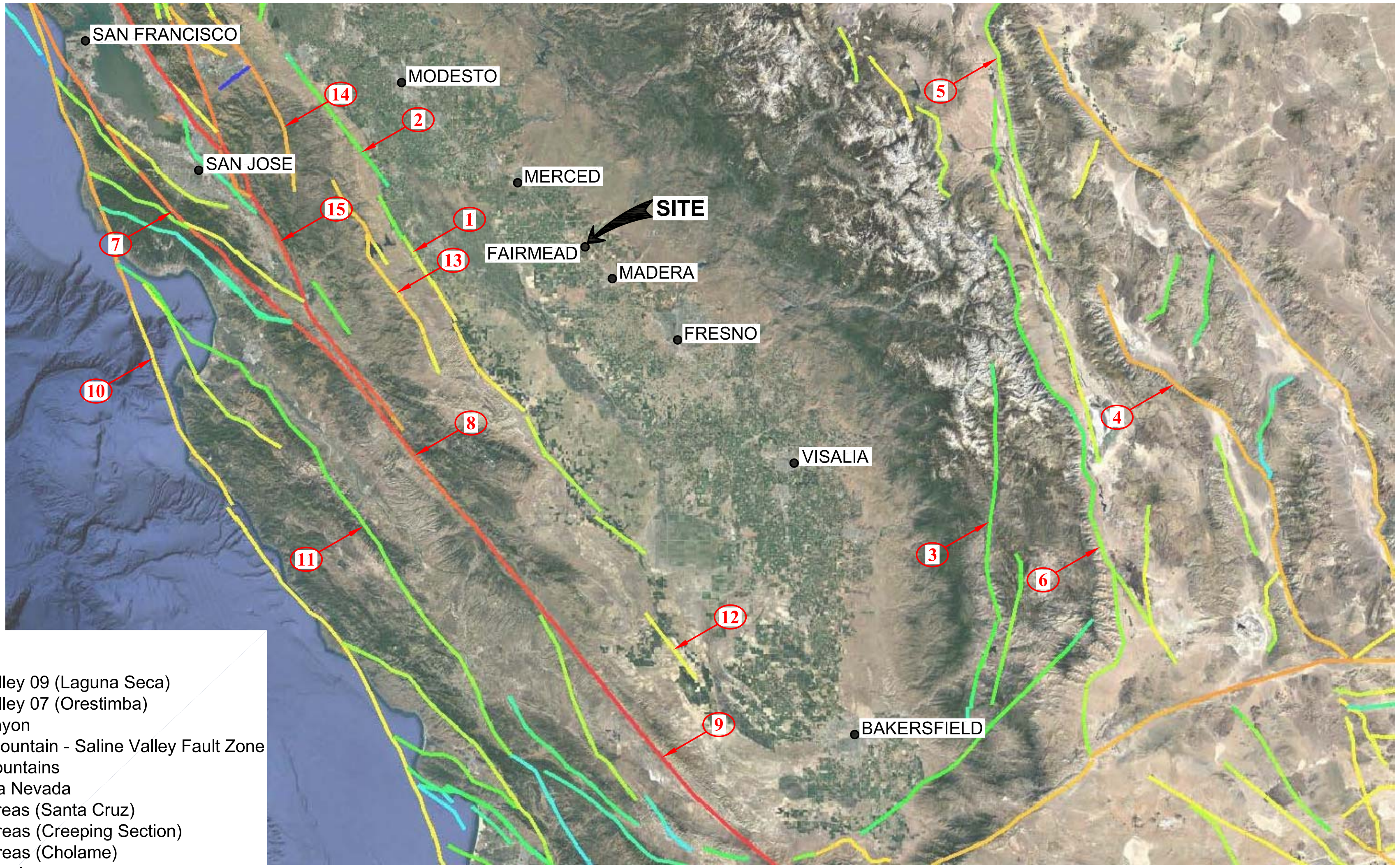
CROSS SECTION A-A'
PROPOSED MULTI-PURPOSE BUILDING
FAIRMEAD ELEMENTARY SCHOOL
19421 AVENUE 22 3/4
CHOWCHILLA, CALIFORNIA

FIGURE
5



TECHNICON
ENGINEERING SERVICES, INC.

PROJECT: 230556	EPICENTER MAP PROPOSED MULTI-PURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL 19421 AVENUE 22 3/4 CHOWCHILLA, CALIFORNIA	FIGURE 6 NTS
SOURCE: CGS		



FAULTS

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

TECHNICON
ENGINEERING SERVICES, INC.

PROJECT: 230556	DATE: 10/31/23	REGIONAL FAULT ACTIVITY MAP PROPOSED MULTI-PURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL 19421 AVENUE 22 3/4 CHOWCHILLA, CALIFORNIA	FIGURE 7 NTS
SOURCE: WGCEP	APPROVED BY: AA		

BORING LOGS AND LOG KEY

APPENDIX A

TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, California 93722
Telephone: 559.276.9311

KEY TO SYMBOLS

PROJECT NAME Proposed Multi-Purpose Building
DATE OF EXPLORATION 9/22/2023
PROJECT LOCATION Chowchilla, California
PROJECT NUMBER 230556

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

KEY TO SYMBOLS 2 - TECHNICON.GDT - 11/1/23 08:11 - Z:\TESDATA\PROJECTS\230500-230599\230556 FAIRMEAD ES - MULTI PURPOSE BLDG\REPORTS\230556 - GINT.GPJ

TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, California 93722
Telephone: 559.276.9311

BORING B-1

PROJECT NAME Proposed Multi-Purpose Building
PROJECT NUMBER 230556
PROJECT LOCATION Chowchilla, California
SURFACE DESCRIPTION grass
DATE STARTED 9/22/23 **COMPLETED** 9/22/23
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE SIMCO
BORING DEPTH 51.5 ft
DRILLING METHOD Solid Flight Auger
LOGGED BY C. Odneal **CHECKED BY** A. AhTye

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Clayey SAND (SC)** - medium dense, brown, moist, fine to coarse grained				
	CAL	2-2-3 (5)			117.9	10.3	S = 67 %	
	GB							
5	CAL	17-50/5"		Very dense	127.3	9.7	S = 86 %	
10				**Sandy CLAY (CL)** - hard, brown, moist, low plasticity				
	SPT	9-20-50/3"						
				Poorly Graded SAND (SP) - dense, light brown, moist, fine to coarse grained, trace clay				
15	CAL	10-21-22 (43)			111.6	7.2	S = 40 %	
20	CAL	14-25-24 (49)			114.1	6.2	S = 36 %	
				Sandy CLAY (CL) - hard, brown, moist, low plasticity				
25	CAL	5-17-27 (44)			120.9	12.9	S = 93 %	
				Sandy SILT (ML) - very stiff, brown, moist				
30	SPT	6-8-12 (20)						
35								

(Continued Next Page)

TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, California 93722
Telephone: 559.276.9311

BORING B-1

PROJECT NAME Proposed Multi-Purpose Building
PROJECT NUMBER 230556
PROJECT LOCATION Chowchilla, California
SURFACE DESCRIPTION grass
DATE STARTED 9/22/23 **COMPLETED** 9/22/23
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE SIMCO
BORING DEPTH 51.5 ft
DRILLING METHOD Solid Flight Auger
LOGGED BY C. Odneal **CHECKED BY** A. AhTye

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
35	CAL	8-15-18 (33)		**Sandy SILT (ML)** - very stiff, brown, moist *(continued)* Hard	108.9	17.1	S = 87 %	
40	SPT	8-10-11 (21)		Very stiff				
45	CAL	19-34-35 (69)		Hard	126.4	10.1	S = 87 %	
50	SPT	7-12-23 (35)						

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

TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, California 93722
Telephone: 559.276.9311

BORING B-2

PROJECT NAME Proposed Multi-Purpose Building
PROJECT NUMBER 230556
PROJECT LOCATION Chowchilla, California
SURFACE DESCRIPTION grass
DATE STARTED 9/22/23 **COMPLETED** 9/22/23
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE SIMCO
BORING DEPTH 21.5 ft
DRILLING METHOD Solid Flight Auger
LOGGED BY C. Odneal **CHECKED BY** A. AhTye

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Clayey SAND (SC)** - medium dense, brown, moist, fine to medium grained				
	CAL	12-12-12 (24)			103.1	18.0	S = 79 %	
5	SPT	19-16-6 (22)		**Sandy SILT (ML)** - stiff, brown, moist Very stiff				
10	CAL	6-13-17 (30)			123.3	12.7	S = 99 %	
15	SPT	8-9-12 (21)		**Poorly Graded SAND (SP)** - medium dense, light brown, moist, fine to coarse grained				
				Silty SAND (SM) - medium dense, light brown, moist, fine to medium grained				
20	SPT	2-9-15 (24)						

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

TECHNICON Engineering Services, Inc.
4539 N. Brawley Avenue #108
Fresno, California 93722
Telephone: 559.276.9311

BORING B-3

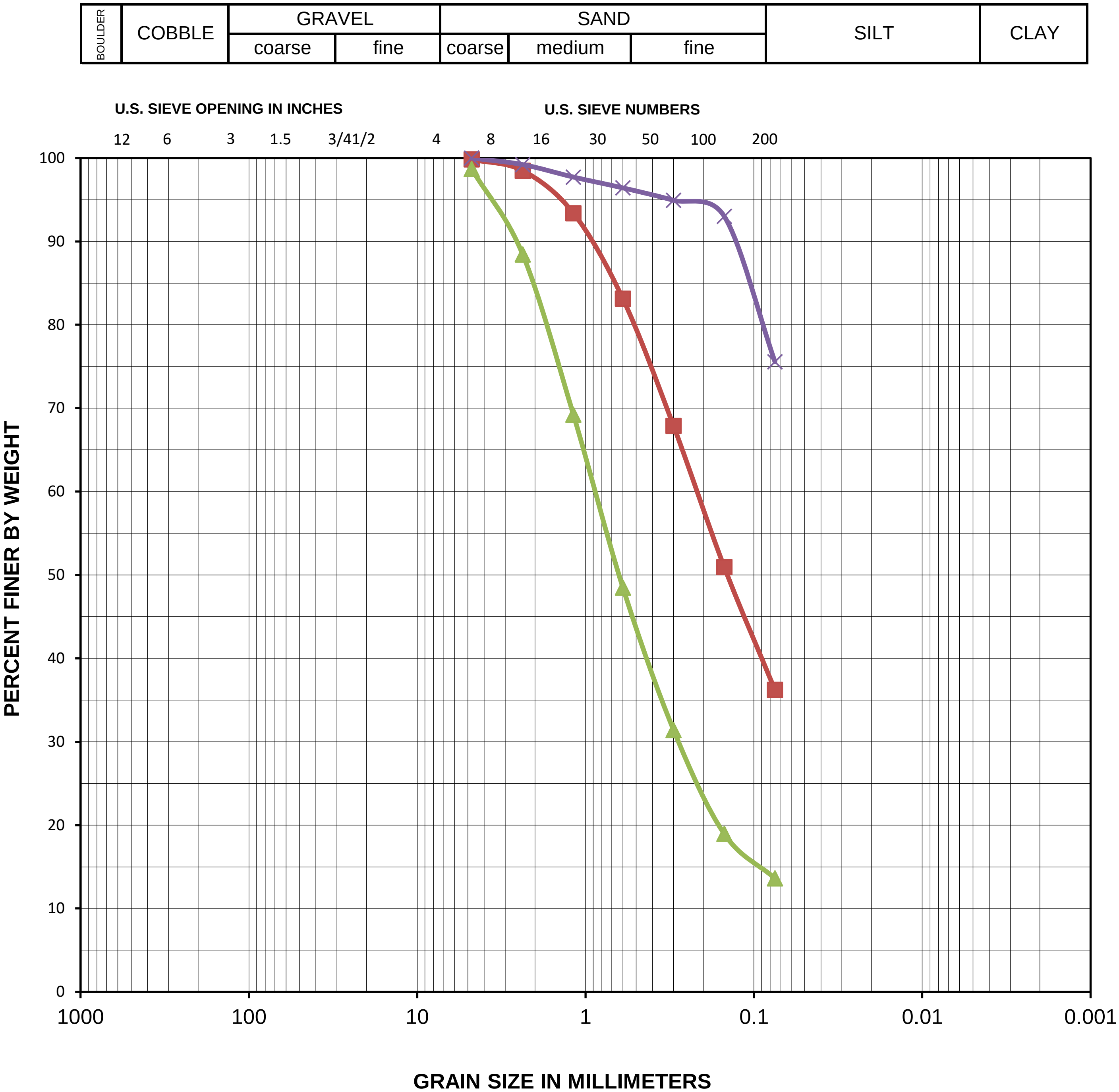
PROJECT NAME Proposed Multi-Purpose Building
PROJECT NUMBER 230556
PROJECT LOCATION Chowchilla, California
SURFACE DESCRIPTION grass
DATE STARTED 9/22/23 **COMPLETED** 9/22/23
GROUND ELEVATION 0 ft
DRILLING CONTRACTOR TECHNICON Engineering Services, Inc.
GROUND WATER LEVEL No groundwater encountered.
DRILL RIG TYPE SIMCO
BORING DEPTH 16.5 ft
DRILLING METHOD Solid Flight Auger
LOGGED BY C. Odneal **CHECKED BY** A. AhTye

DEPTH (ft)	SAMPLE TYPE	BLOWS/ft	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	REMARKS
0				**Clayey SAND (SC)** - dense, brown, moist, fine to coarse grained				
	GB CAL	7-10-16 (26)			123.5	6.3	S = 49 %	
5				**Sandy SILT (ML)** - very stiff, brown, moist				
	CAL	10-29-33 (62)			126.0	9.1	S = 77 %	
10	SPT	6-12-16 (28)		Stiff				
				Poorly Graded SAND (SP) - dense, light brown, moist, fine to coarse grained				
15	CAL	15-21-28 (49)			109.0	4.9	S = 25 %	

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

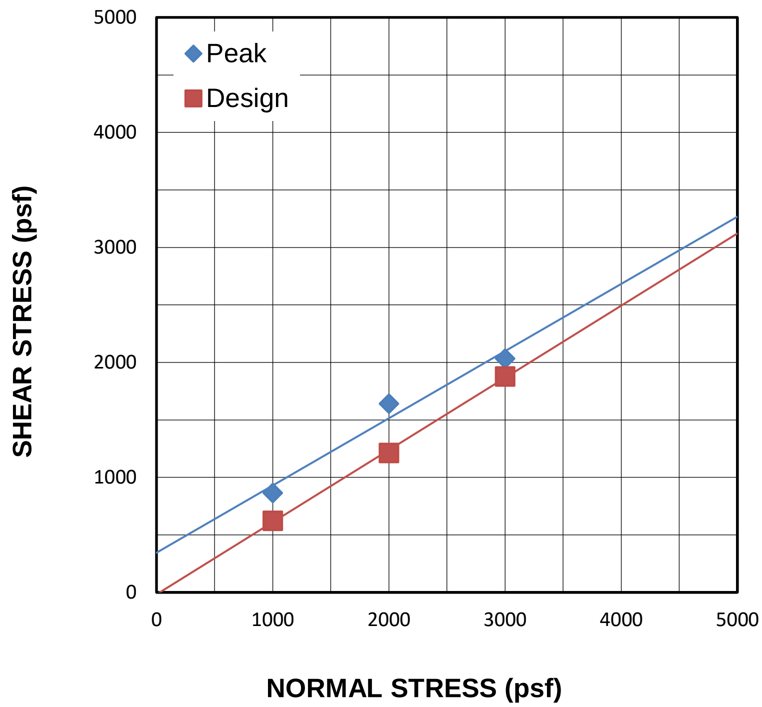
LABORATORY TESTS

APPENDIX B



Boring	Depth (ft.)	Sample Description	Passing 3/4"	Passing #4	Passing #200
B-3	0-5	Clayey SAND (SC)	100.0	99.8	36.2
B-2	20	Silty SAND (SM)	100.0	98.7	13.6
B-1	30	Sandy SILT (ML)	100.0	100.0	75.6

PROJECT NO.:	230556	SIEVE ANALYSIS	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:	SA	PROPOSED MULTIPURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL CHOWCHILLA, CALIFORNIA	
INPUT BY:	SA		
CHECKED BY:	SA		
DATE:	10/31/2023		
REVISED:	-		



	Depth (ft.)	Sample Description
B-3	2	Clayey SAND (SC)

	Specimen No.	Dry Unit Weight (pcf)	Water Content (%)	Saturation (%)	Area (in^2)	Height (in)
Initial	1	123.5	6.3	49.3	4.60	1.00
	2	123.5	6.3	49.3	4.60	1.00
	3	123.5	6.3	49.3	4.60	1.00
At Test	Specimen No.	Dry Unit Weight (pcf)	Water Content (%)	Saturation (%)	Area (in^2)	Height (in)
	1	123.8	12.5	98.5	4.60	0.998
	2	123.0	12.2	93.9	4.60	1.004
	3	124.1	12.5	99.8	4.60	0.995

Specimen No.	Peak Shear Stress (psf)	Design Shear Stress (psf)	Normal Stress (psf)	Strain Rate (in/min)
1	864.9	623.0	1000	0.002
2	1641.6	1211.6	2000	0.002
3	2033.6	1878.4	3000	0.002

Results	Cohesion (psf)	Friction ϕ (deg)
Peak	345	30.3
Design	0	32.1

PROJECT NO	230556	DIRECT SHEAR	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:		PROPOSED MULTIPURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL CHOWCHILLA, CALIFORNIA	
INPUT BY:	AA		
CHECKED BY	SA		
DATE:	10/31/2023		
REVISED:	-		

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

Moisture		
Wet Weight (g)	Dry Weight (g)	Water Content (%)
100.0	93.1	7.4

Soil Specimen		
Mold Weight (g)	Soil + Mold Weight (g)	Soil Weight (g)
363.6	787.7	424.1
Mold Diameter (in)	Mold Height (in)	Mold Volume (ft^3)
4.0	1.0	12.57
Moist Density (pcf)	Dry Density (pcf)	Saturation (%)
127.9	119.1	48.2

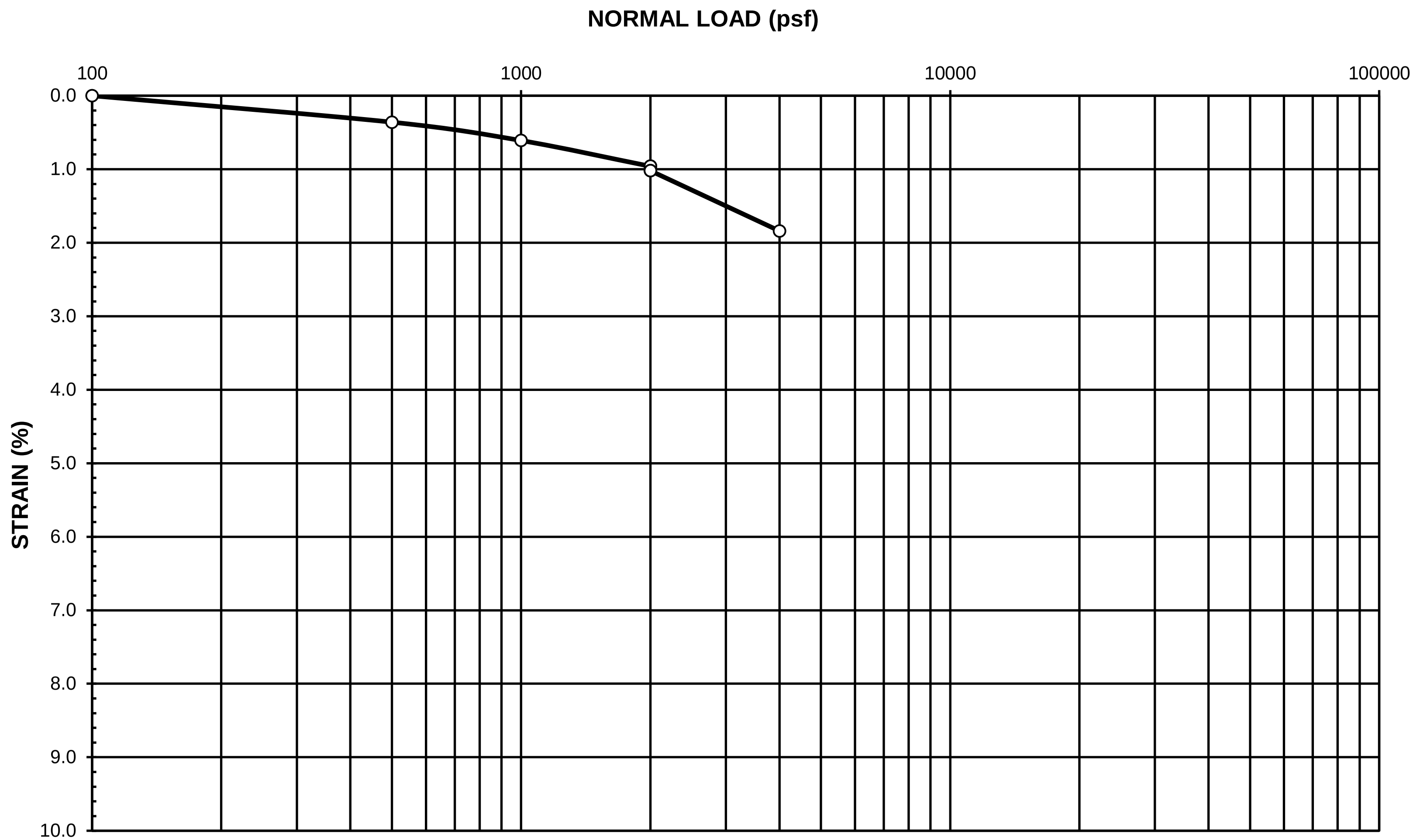
Expansion		
Initial Reading (in)	Final Reading (in)	Expansion (in)
0.0000	0.0124	0.0124

Expansion Index, EI	
$EI_{measured}$	EI_{50}
12.4	11.6

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	230556	EXPANSION INDEX	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:		PROPOSED MULTIPURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL CHOWCHILLA, CALIFORNIA	
INPUT BY:	AA		
CHECKED BY	SA		
DATE:	10/31/2023		
REVISED:	-		



Boring	Depth (ft)	Sample Description
B-2	3.0	Clayey SAND (SC)

	Sample Diameter (in)	Sample Height (in)	Moisture Content (%)	Dry Density (pcf)
Initial	2.42	1.0000	6.3	115.1
Final	2.42	0.9816	21.1	117.3

PROJECT NO.:	230556
LAB TECH:	WJ
INPUT BY:	AA
CHECKED BY:	SA
DATE:	10/31/2023
REVISED:	-

COLLAPSE POTENTIAL

PROPOSED MULTIPURPOSE BUILDING
FAIRMEAD ELEMENTARY SCHOOL
CHOWCHILLA, CALIFORNIA

TECHNICON
ENGINEERING SERVICES, INC.

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

MINIMUM RESISTIVITY

Water Added (ml)	0	150	250	350					
Resistance (ohm)	80,000	3,300	3,100	3,400					
Resistivity (ohm-cm)*	85,200	3,515	3,302	3,621					

Box Constant=1.065

Minimum Resistivity (ohm-cm)	3,302
pH	6.66

Years to perforation*	20

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

CHEMICAL ANALYSIS

	Soluble Sulfate SO_4-S	Soluble Chloride Cl
	7.5 mg/kg	1.8 mg/kg
	5.3 mg/kg	1.8 mg/kg
	5.8 mg/kg	1.8 mg/kg
Average	6.2 mg/kg	1.8 mg/kg

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

PROJECT NO.: 230556	CORROSIVITY TESTS	TECHNICON ENGINEERING SERVICES, INC.
LAB TECH:	PROPOSED MULTIPURPOSE BUILDING FAIRMEAD ELEMENTARY SCHOOL CHOWCHILLA, CALIFORNIA	
INPUT BY: AA		
CHECKED BY: SA		
DATE: 10/31/2023		
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.

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

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (4.2.0)

Latitude

Decimal degrees

37.0819

Longitude

Decimal degrees, negative values for western longitudes

-120.1941

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

Time Horizon

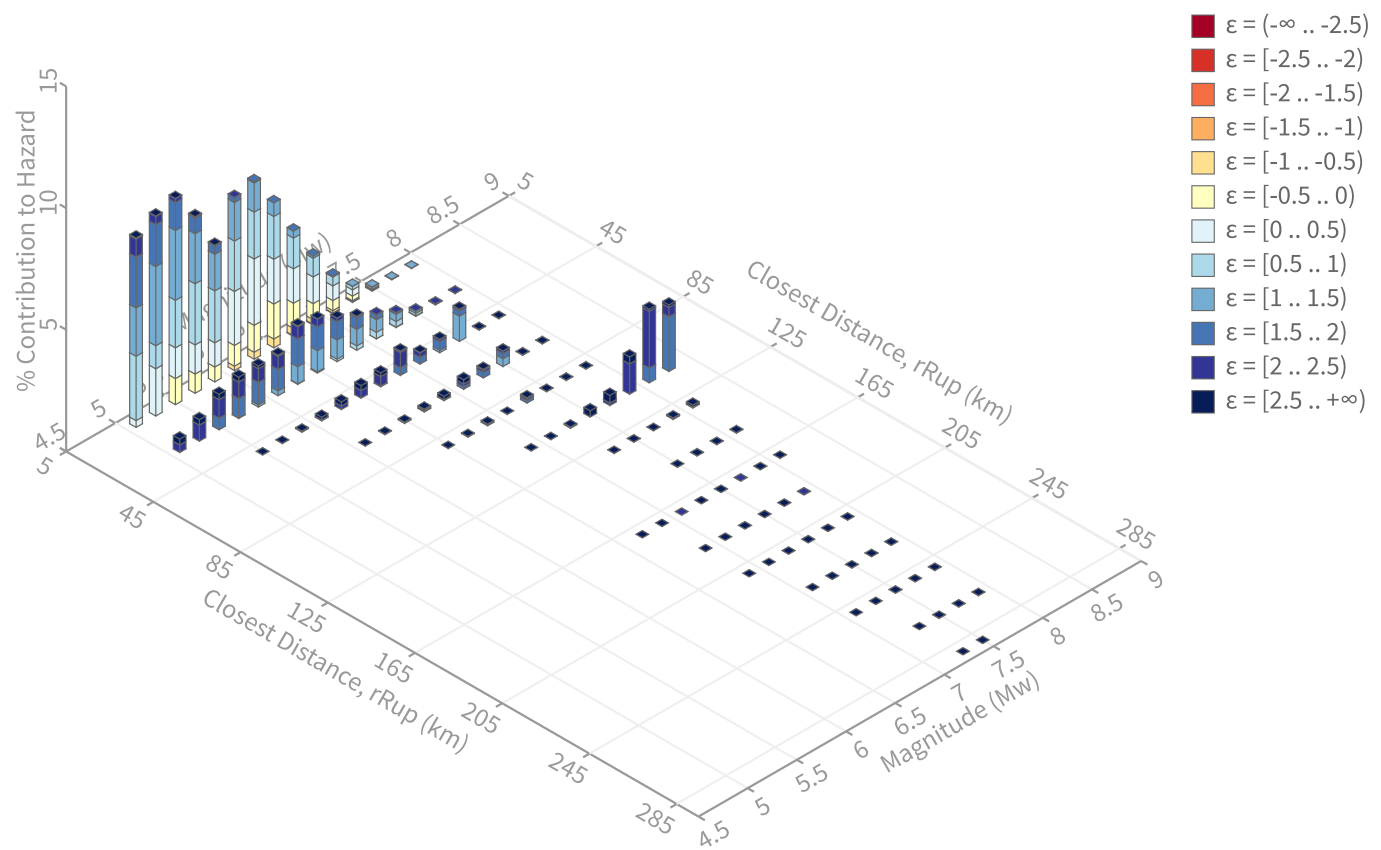
Return period in years

2475

^ Deaggregation

Component

Total


% Contribution to Hazard
Closest Distance, rRup (km)
Magnitude (Mw)
ε = (-∞ .. -2.5)
ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.34841478 g

Recovered targets

Return period: 2756.5109 yrs
Exceedance rate: 0.00036277745 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.14 %

Mean (over all sources)

m: 6.23
r: 24.59 km
ε_0: 1.1 σ

Mode (largest m-r bin)

m: 5.5
r: 10.57 km
ε_0: 0.89 σ
Contribution: 8.47 %

Mode (largest m-r-ε_0 bin)

m: 5.3
r: 10.47 km
ε_0: 1.22 σ
Contribution: 3.25 %

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							43.74
PointSourceFinite: -120.194, 37.104		5.62	5.65	0.19	120.194°W	37.104°N	0.00	4.00
PointSourceFinite: -120.194, 37.104		5.62	5.65	0.19	120.194°W	37.104°N	0.00	3.96
PointSourceFinite: -120.194, 37.167		9.99	5.83	0.72	120.194°W	37.167°N	0.00	3.81
PointSourceFinite: -120.194, 37.167		9.99	5.83	0.72	120.194°W	37.167°N	0.00	3.79
PointSourceFinite: -120.194, 37.203		12.93	5.95	0.97	120.194°W	37.203°N	0.00	2.01
PointSourceFinite: -120.194, 37.203		12.93	5.95	0.97	120.194°W	37.203°N	0.00	1.99
PointSourceFinite: -120.194, 37.158		9.29	5.80	0.65	120.194°W	37.158°N	0.00	1.86
PointSourceFinite: -120.194, 37.158		9.29	5.80	0.65	120.194°W	37.158°N	0.00	1.81
PointSourceFinite: -120.194, 37.194		12.18	5.92	0.91	120.194°W	37.194°N	0.00	1.22
PointSourceFinite: -120.194, 37.194		12.18	5.92	0.91	120.194°W	37.194°N	0.00	1.21
UC33brAvg_FM32 (opt)	Grid							43.68
PointSourceFinite: -120.194, 37.104		5.62	5.65	0.19	120.194°W	37.104°N	0.00	4.00
PointSourceFinite: -120.194, 37.104		5.62	5.65	0.19	120.194°W	37.104°N	0.00	3.96
PointSourceFinite: -120.194, 37.167		9.99	5.83	0.72	120.194°W	37.167°N	0.00	3.80
PointSourceFinite: -120.194, 37.167		9.99	5.83	0.72	120.194°W	37.167°N	0.00	3.79
PointSourceFinite: -120.194, 37.203		12.93	5.95	0.97	120.194°W	37.203°N	0.00	2.01
PointSourceFinite: -120.194, 37.203		12.93	5.95	0.97	120.194°W	37.203°N	0.00	1.99
PointSourceFinite: -120.194, 37.158		9.29	5.80	0.65	120.194°W	37.158°N	0.00	1.86
PointSourceFinite: -120.194, 37.158		9.29	5.80	0.65	120.194°W	37.158°N	0.00	1.81
PointSourceFinite: -120.194, 37.194		12.18	5.92	0.91	120.194°W	37.194°N	0.00	1.22
PointSourceFinite: -120.194, 37.194		12.18	5.92	0.91	120.194°W	37.194°N	0.00	1.21
UC33brAvg_FM32	System							6.31
San Andreas (Creeping Section) [12]		102.92	8.14	2.08	121.083°W	36.497°N	230.86	2.93
Great Valley 09 (Laguna Seca) [12]		58.94	7.24	1.74	120.754°W	36.806°N	238.48	1.30
UC33brAvg_FM31	System							6.26
San Andreas (Creeping Section) [12]		102.92	8.15	2.08	121.083°W	36.497°N	230.86	2.91
Great Valley 09 (Laguna Seca) [12]		58.94	7.24	1.75	120.754°W	36.806°N	238.48	1.35

SITE SPECIFIC GROUND MOTION ANALYSIS

APPENDIX D

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

Technicon Engineering Services, Inc.

Project:	**Proposed Multipurpose Building**
Job #:	**230556**
Date:	**11/1/2023**
Checked by:	**S. Alvarez**

TECHNICON
ENGINEERING SERVICES, INC.

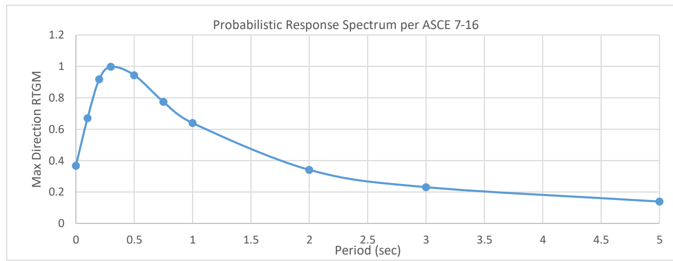
INPUT
OUTPUT
ANALYSIS

S_S	**0.571**
S_1	**0.23**
S_{DS}	**0.511**
PGA_M	**0.332**
F_a	**1.343**

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

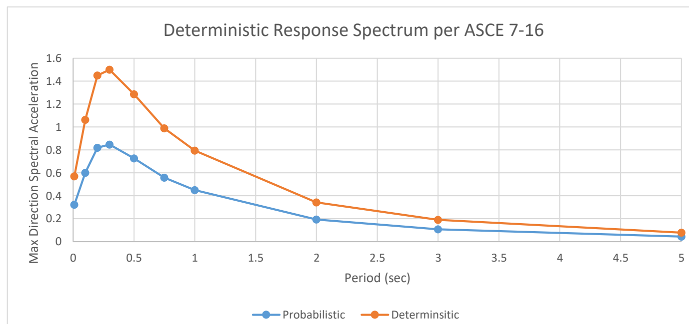
1. Use Unified Hazard Tool "raw data" from Hazard Curve & Risk-Targeted Ground Motion Calculator to get "UHGM & RTGM" values
   a. Plot time vs. adjusted RTGM
2. Input M_w and R_{rup} into NGAW2 Excel worksheet. M_w & R_{rup} can be found with deagg sheet (unified hazard tool) "Mean (over all sources)".
   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	0.348	0.334	1.1	0.3674
0.1	0.637	0.61	1.1	0.671
0.2	0.867	0.835	1.1	0.9185
0.3	0.923	0.887	1.125	0.997875
0.5	0.836	0.804	1.175	0.9447
0.75	0.655	0.626	1.2375	0.774675
1	0.514	0.492	1.3	0.6396
2	0.268	0.253	1.35	0.34155
3	0.176	0.165	1.4	0.231
5	0.101	0.093	1.5	0.1395



Scaling Factor: **1.769836353**

	*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	0.292939955	1.1	0.32223395	0.570301359
0.1	0.546031167	1.1	0.600634284	1.063024391
0.2	0.7445393	1.1	0.81899323	1.449483992
0.3	0.753365321	1.125	0.847535987	1.5
0.5	0.618580721	1.175	0.726832347	1.286374311
0.75	0.451589078	1.2375	0.558841485	0.989057975
1	0.345675294	1.3	0.449377882	0.795325313
2	0.14308488	1.35	0.193164588	0.34186971
3	0.076800641	1.4	0.107520897	0.190294392
5	0.029570309	1.5	0.044355463	0.078501911



- 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 @ $S_S \geq 1.5$ → $F_{a=}$ 1.343

F_a1.5 → $F_{a=}$ 2.0145

- 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: $S_{M1} = S_1 * F_V$ → 0.8625

equ. 11.4-4: $S_{D1} = \left(\frac{2}{3}\right) S_{M1}$ → 0.575

T_0 → 0.225

T_S → 1.125

S_S	0.571
S_1	0.23
S_{DS} * from seismic design map	0.511
S_{D1} * from section 11.4.6	0.575
T_0	0.225
T_S	1.125

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

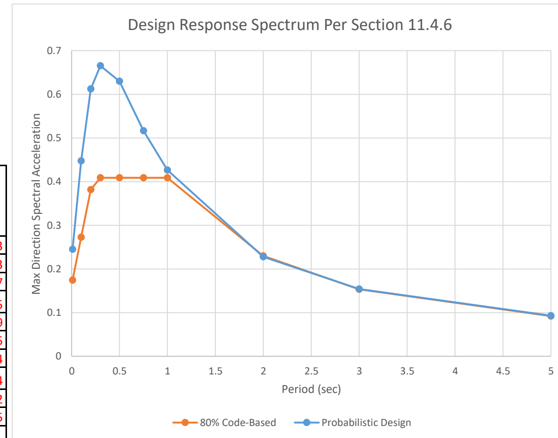
Technicon Engineering Services, Inc.

Project:	**Proposed Multipurpose Building**
Job #:	**230556**
Date:	**11/1/2023**
Checked by:	**S. Alvarez**

TECHNICON
ENGINEERING SERVICES, INC.

INPUT
OUTPUT
ANALYSIS

S_S	0.571
S_1	0.23
S_{DS}	0.511
PGA_M	0.332
F_a	1.343



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	0.218023704	T less than T_o	0.174418963	0.244933333	0.002449333
0.1	0.340637043		0.272509635	0.447333333	0.044733333
0.2	0.476874087		0.38149927	0.612333333	0.122466667
0.3	0.511	T_0 < T < T_S; T = S_{DS}	0.4088	0.66525	0.199575
0.5	0.511		0.4088	0.6298	0.3149
0.75	0.511		0.4088	0.51645	0.3873375
1	0.511		0.4088	0.4264	0.4264
2	0.2875	T > T_S; S_a = S_{D1}/T	0.23	0.2277	0.4554
3	0.191666667		0.153333333	0.154	0.462
5	0.115		0.092	0.093	0.465

- Section 21.4 Design Acceleration Parameters

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

$S_{a\ max}$ → 0.66525

$S_{DS} = 90\% * S_{a\ max}$ → 0.599

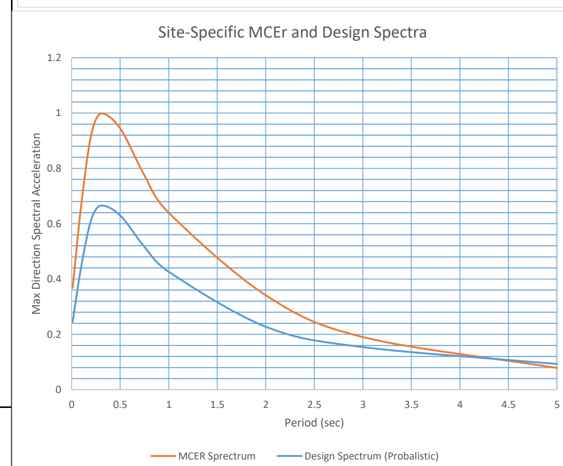
$S_{MS} = 1.5 * S_{DS}$ → 0.898

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 → 0.465

S_{D1} → 0.465

$S_{M1} = 1.5 * S_{D1}$ S_{M1} → 0.698



- Section 21.5.1 - Probabilistic MCE_G Peak Ground Acceleration

Probablisitc PGA from UHGM @ T=0 sec

$PGA_{prob.}$ → 0.348

- Section 21.5.2 - Determinsitic MCE_G Peak Ground Acceleration

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

PGA → 0.293

Table **11.8-1**: Site Class D @ PGA = 0.5 → F_{PGA} = 1.355
$0.5F_{PGA}$ = 0.6775

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

Therefore; $PGA_{det.}$ → 0.6775

- Section 21.5.3 - Site Specific MCE_G Peak Ground Acceleration

$PGA_{prob.}$ → 0.348
→ *Take the lesser PGA_{SSe} 0.348
$PGA_{det.}$ → 0.6775

PGA CHECK

From Seismic Design Map: PGA_M → 0.332

80 % of PGA_M → 0.266

*Take the greater
Site-Specific PGA → 0.348

Final Seismic Design Values	
S_S	0.571
S_1	0.230
S_{MS}	0.898
S_{DS}	0.599
S_{D1}	0.465
S_{M1}	0.698
F_A	1.343
F_V	2.500
PGA_M	0.348

LIQUEFACTION ANALYSIS AND SEISMICALLY INDUCED SETTLEMENT CALCULATIONS

APPENDIX E

Proposed Multipurpose Building
DSA File
DSA App No.

Calc by AA Date 11/1/23
Checked by SA Date 11/1/23

Project No: TES 230556
Boring: B-1

Hammer Efficiencies - Technicon Drilling Rigs	
CME 45	80.0%
CME 55	82.4%
CME 75	87.8%
SIMCO	88.0%

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

Rod Length = **1.22** meters above grounds surface
Hammer Efficiency = **88%** Emean/E60 = Energy Ratio to correct to standard 60% Energy
Ring Sampler Corr. = **0.65**

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= 1.467 Sur.= 0 psf Measured Ground Water Depth = 100 feet Design Ground Water Depth = 49 feet acc. max = 0.348 g Earthq. Mw = 6.23

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?
4	4	SC	4	0.12	0.12	0.12	0.6	1.68	120	120	2	5	5.000	1.200	0.995	1.79	70.0	1.1	0.75	1.00	0.83	6.7	13.0	0.125	0.140	1.12	ABOVE
8	4	ML	4	0.37	0.37	0.37	1.8	1.43	128	128	1	22	4.706	1.154	0.986	1.79	30.0	1.1	0.80	1.20	1.07	49.1	61.4	0.124	LARGE	LARGE	ABOVE
13	4	ML	5	0.65	0.65	0.65	3.2	1.22	125	125	2	30	5.000	1.200	0.976	1.79	60.0	1.0	0.85	1.00	0.85	29.7	40.6	0.123	LARGE	LARGE	ABOVE
18	4	SP-SM	5	0.96	0.96	0.96	4.7	1.06	120	120	1	21	5.000	1.200	0.964	1.79	60.0	1.0	0.85	1.20	1.02	33.2	44.8	0.121	LARGE	LARGE	ABOVE
23	4	SP-SM	5	1.26	1.26	1.26	6.2	0.93	120	120	1	24	5.000	1.200	0.952	1.79	60.0	1.0	0.95	1.20	1.14	37.4	49.9	0.120	LARGE	LARGE	ABOVE
28	4	CL	5	1.56	1.56	1.56	7.8	0.83	125	125	2	44	0.419	1.015	0.941	1.79	8.5	1.0	0.95	1.00	0.95	33.2	34.1	0.119	LARGE	LARGE	ABOVE
33	4	ML	5	1.86	1.86	1.86	9.3	0.75	115	115	1	20	0.419	1.015	0.926	1.79	8.5	1.0	1.00	1.20	1.20	26.5	27.3	0.117	0.348	2.99	ABOVE
38	4	ML	5	2.15	2.15	2.15	10.8	0.69	115	115	2	33	5.000	1.200	0.885	1.79	70.0	1.0	1.00	1.00	1.00	21.7	31.1	0.112	LARGE	LARGE	ABOVE
43	4	ML	5	2.44	2.44	2.44	12.3	0.64	115	115	1	21	5.000	1.200	0.844	1.79	70.0	1.0	1.00	1.20	1.20	23.6	33.3	0.106	LARGE	LARGE	ABOVE
48	4	ML	5	2.75	2.75	2.75	13.9	0.59	130	130	2	69	5.000	1.200	0.804	1.79	70.0	1.0	1.00	1.00	1.00	38.8	51.5	0.101	LARGE	LARGE	ABOVE
49	4	ML	1	2.94	2.94	2.94	14.8	0.40	130	130	1	35	5.000	1.200	0.779	1.79	60.0	1.0	1.00	1.20	1.20	24.6	34.6	0.098	LARGE	LARGE	ABOVE
51.5	4	ML	2.5	3.05	3.05	3.02	15.3	0.40	130	135	1	35	5.000	1.200	0.765	1.79	60.0	1.0	1.00	1.20	1.20	24.6	34.6	0.098	LARGE	LARGE	NO

Proposed Multipurpose Building
DSA File
DSA App No.

Calc by AA Date 11/1/23
Checked by SA Date 11/1/23

Project No: TES 230556
Boring: B-1

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)**
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******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 = **88%** 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] $C_N = (P_a/s'_{vo})^{0.5}$ Youd and Idriss 2001 Formula (9)

Emean/E60= 1.467 Sur.= 0 psf Measured Ground Water Depth = 100 feet Design Ground Water Depth = 49.0 feet acc. max = 0.348 g Earthq. Mw = 6.23

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
4	4	SC	4	0.12	0.12	0.12	0.6	1.68	120	120	2	5	0.995	1.79	70.0	0.83	6.7	5.6	12.2	0.125	1.12	-	ABOVE
8	4	ML	4	0.37	0.37	0.37	1.8	1.43	128	128	1	22	0.986	1.79	30.0	1.07	49.1	2.4	51.6	0.124	LARGE	-	ABOVE
13	4	ML	5	0.65	0.65	0.65	3.2	1.22	125	125	2	30	0.976	1.79	60.0	0.85	29.7	4.8	34.5	0.123	LARGE		ABOVE
18	4	SP-SM	5	0.96	0.96	0.96	4.7	1.06	120	120	1	21	0.964	1.79	60.0	1.02	33.2	4.8	38.0	0.121	LARGE	-	ABOVE
23	4	SP-SM	5	1.26	1.26	1.26	6.2	0.93	120	120	1	24	0.952	1.79	60.0	1.14	37.4	4.8	42.2	0.120	LARGE	-	ABOVE
28	4	CL	5	1.56	1.56	1.56	7.8	0.83	125	125	2	44	0.941	1.79	8.5	0.95	33.2	0.7	33.9	0.119	LARGE	-	ABOVE
33	4	ML	5	1.86	1.86	1.86	9.3	0.75	115	115	1	20	0.926	1.79	8.5	1.20	26.5	0.7	27.3	0.117	2.99	-	ABOVE
38	4	ML	5	2.15	2.15	2.15	10.8	0.69	115	115	2	33	0.885	1.79	70.0	1.00	21.7	5.6	27.3	0.112	LARGE	-	ABOVE
33	4	ML	5	2.44	2.44	2.44	12.3	0.64	115	115	1	21	0.844	1.79	70.0	1.20	23.6	5.6	29.2	0.106	LARGE	-	ABOVE
48	4	ML	5	2.75	2.75	2.75	13.9	0.59	130	130	2	69	0.804	1.79	70.0	1.00	38.8	5.6	44.4	0.101	LARGE	-	ABOVE
49	4	ML	1	2.94	2.94	2.94	14.8	0.40	130	130	1	35	0.779	1.79	60.0	1.20	24.6	4.8	29.4	0.098	LARGE	-	ABOVE
51.5	4	ML	2.5	3.05	3.05	3.02	15.3	0.40	130	135	1	35	0.765	1.79	60.0	1.20	24.6	4.8	29.4	0.098	LARGE	-	NONE

Total Settlement 0.0
May be off by 0.1 inches due to rounding

Proposed Multipurpose Building
DSA File
DSA App No.

Calc by AA Date 11/1/23
Checked by SA Date 11/1/23

Project No: TES 230556
Boring: B-1

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 = 0.348 g Earthq. Mw = 6.23

Elev. Base of Layer (ft)	Boring Diameter (in)	Soil Type	Layer Thickness (ft)	Depth to Midpoint (m)	Total Unit Wt. (pcf)	Total Overburden Pressure s_{vo} (psf)	Sampler Type 1 = SPT 2=Ca.Mod	Field Blow Count N (SPT)	Stress Reduct. Coeff. 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)
4	4	SC	4	0.6	120	240.0	2	5	0.995	13.0	9.20E-05	0.08	1.80E-04	1.80E-02	3.30E-02	0.6898	0.0228	0.0455	0.0219
8	4	ML	4	1.8	128	736.0	1	22	0.986	61.4	9.51E-05	0.24	1.50E-04	1.50E-02	5.00E-03	0.6898	0.0034	0.0069	0.0033
13	4	ML	5	3.2	125	1304.5	2	30	0.976	40.6	1.44E-04	0.42	2.00E-04	2.00E-02	6.00E-03	0.6898	0.0041	0.0083	0.0050
18	4	SP-SM	5	4.7	120	1917.0	1	21	0.964	44.8	1.67E-04	0.62	2.10E-04	2.10E-02	5.00E-03	0.6898	0.0034	0.0069	0.0041
23	4	SP-SM	5	6.2	120	2517.0	1	24	0.952	49.9	1.82E-04	0.82	2.70E-04	2.70E-02	4.00E-03	0.6898	0.0028	0.0055	0.0033
28	4	CL	5	7.8	125	3129.5	2	44	0.941	34.1	2.28E-04	1.02	9.50E-04	9.50E-02	9.50E-02	0.6898	0.0655	0.1311	--
33	4	ML	5	9.3	115	3729.5	1	20	0.926	27.3	2.63E-04	1.21	4.50E-04	4.50E-02	3.00E-02	0.6898	0.0207	0.0414	0.0248
38	4	ML	5	10.8	115	4304.5	2	33	0.885	31.1	2.59E-04	1.40	4.40E-04	4.40E-02	5.00E-02	0.6898	0.0345	0.0690	0.0414
43	4	ML	5	12.3	115	4879.5	1	21	0.844	33.3	2.57E-04	1.59	4.30E-04	4.30E-02	2.00E-02	0.6898	0.0138	0.0276	0.0166
48	4	ML	5	13.9	130	5492.0	2	69	0.804	51.5	2.25E-04	1.78	3.20E-04	3.20E-02	4.00E-03	0.6898	0.0028	0.0055	0.0033
49	4	ML	1	14.8	130	5882.0	1	35	0.779	34.6	2.57E-04	1.91	3.90E-04	3.90E-02	1.50E-02	0.6898	0.0103	0.0207	0.0025
51.5	4	ML	2.5	15.3	130	6109.5	1	35	0.765	34.6	2.58E-04	1.99	3.80E-04	3.80E-02	1.40E-02	0.6898	0.0097	0.0193	0.0058
																		Total Settlement	0.13