ADDENDUM 02 – MADISON ES – 2 STORY CR BLDG

Addendum No:	02	**Issue Date:**	01/08/2024
Project:	Madison Elementary School – 2 Story Classroom Building	**To Drawings + Specifications dated**	12/06/2024
School District:	**Madera Unified School District**		
Prepared By:	PBK Architects, Inc. 7790 N Palm Avenue Fresno, California 93711		
PBK Project No:	230278		
DSA App No:	02-122191		

LICENSED ARCHITECT
MICHAEL R. SCHOEN
No. C-35165
RENEWAL DATE
FEB 28, 2025
STATE OF CALIFORNIA

NOTICE TO PROPOSERS

A. The following changes, omissions, and/or additions to the Project Manual and/or Drawings shall apply to proposals made for and to the execution of the various parts of the work affected thereby, and all other conditions shall remain the same.

B. Careful note of the Addendum shall be taken by all parties of interest so that the proper allowances may be made in strict accordance with the Addendum, and that all trades shall be fully advised in the performance of the work which will be required of them.

C. Bidder shall acknowledge receipt of this Addendum in the space provided on the Bid Form. Failure to do so may subject Bidder to disqualification.

D. In case of conflict between Drawings, Project Manual, and this Addendum, this Addendum shall govern.

GENERAL ITEMS

2.1 Refer to **Bid No 121224-D Madison ES-: New Two-Story Classroom Building,** revise the following.

- Refer to **MADERA UNIFIED SCHOOL DISTRICT DOCUMENT 00020 NOTICE INVITING BIDS,** revise the bid date and time as follows:
    - Change "*Sealed Bids must be received by **January 22, 2025** at MUSD Purchasing Department, 1205 Madera Avenue, Madera CA 93637. (located on the 2nd floor) no later than **2:00:00 p.m.***"

2.2 Refer to **Geotechnical Engineering Investigation & Geologic Seismic Hazards Evaluation- Madison ES,** add the following.

- Add **Geotechnical Engineering Investigation & Geologic Seismic Hazards Evaluation- Madison ES** in its entirety with the attached (57 Pages).

END OF ADDENDUM 02

GEOTECHNICAL ENGINEERING INVESTIGATION AND GEOLOGIC SEISMIC HAZARDS EVALUATION
MADISON ELEMENTARY SCHOOL
109 STADIUM ROAD
MADERA, CALIFORNIA 95637

BSK PROJECT G00001343

PREPARED FOR:

MADERA UNIFIED SCHOOL DISTRICT
1902 HOWARD ROAD
MADERA, CALIFORNIA 93637

August 30, 2023

GEOTECHNICAL ENGINEERING INVESTIGATION AND GEOLOGIC SEISMIC HAZARDS EVALUATION
MADISON ELEMENTARY SCHOOL
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Madera Unified School District
1902 Howard Road
Madera, California 93637

BSK Project: G00001343

August 30, 2023

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1 INTRODUCTION

1.1 General

This report presents the results of the geotechnical engineering investigation and geologic/seismic hazards evaluation for the proposed new 2-story structure planned at 109 Stadium Road in Madera, California as shown on the Boring Location Map, Figure 2. This report provides geotechnical recommendations for the proposed new structures.

The geotechnical engineering investigation was conducted in general accordance with the scope of services outlined in BSK Proposal G00001343, dated July 7, 2023.

In the event that significant changes occur in the design or location of the proposed improvements, the conclusions and recommendations presented in the report will not be considered valid unless the changes are reviewed by BSK, and the conclusions and recommendations are modified or verified in writing as necessary.

1.2 Project Description

BSK understands that the project consists of the design and construction of three new structures and a parking lot on the east side of Madison Elementary School in Madera, California. We understand, based on the provided site plan and on email correspondence, that the structures are anticipated to be approximately 2,000 square feet, 3,700 square feet, and 3,800 square feet. Column, wall and floor slab loads were not provided but anticipated to be less than 2,000 psf with minimal grading. Previous geotechnical investigations were performed at this site for the proposed solar shade structures (BSK Project G21-354-11F, dated December 17, 2023).

If the actual project description differs significantly from that anticipated above, we should be notified so that we can review our scope of work for applicability.

1.3 Purpose and Scope of Services

The purpose of the geotechnical investigation is to assess soil conditions at the project site and provide geotechnical engineering recommendations and geologic/seismic hazards evaluations for use by the project designers during preparation of the project plans and specifications. The scope of the investigation included a field exploration, laboratory testing, engineering analysis, and geologic seismic hazards evaluations.

The investigation was performed in conformance with Chapter 18 "Soils and Foundations," Section 1803A of the 2022 California Building Code and Title 24, California Code of Regulations, for submission to Division of the State Architect.

2 FIELD INVESTIGATION AND LABORATORY TESTING

2.1 Field Investigation

The field exploration, conducted on July 26, 2023, consisted of a site reconnaissance and drilling three (3) exploratory test borings. The test borings were drilled to depths of approximately 21.5 to 51.5 feet below ground surface (bgs). The test borings were drilled with a truck-mounted drill rig, equipped with manually advanced 8-inch augers. The approximate boring locations are presented on Figure 2, Boring Location map. Details of the field exploration and the boring logs are provided in Appendix A.

2.2 Laboratory Testing

Laboratory testing of selected samples were performed to evaluate certain physical and engineering characteristics and properties. The testing program included in-situ moisture and dry density, gradation, direct shear, collapse, expansion, and corrosion potential.

The in-situ moisture and dry density test results are presented on the boring logs in Appendix A. Descriptions of the laboratory test methods and test results are provided in Appendix B.

3 SITE CONDITIONS

3.1 Site Description

At the time of the field investigation, the project site was east of the existing classrooms and occupied by grass, trees, a baseball fence, and improvements. The site was relatively flat. The general site coordinates are approximately 36.951537° North Latitude and 120.063269° West Longitude. The project site was bounded to the north by grass with Olive Avenue beyond, to the east by Santa Cruz Street with residences beyond, and to the west and south by campus facilities including parking areas and structures.

3.2 Subsurface Description

The near surface soils encountered within the test borings consisted of interbedded sandy silt, sandy clay and clayey sand, underlain by poorly graded sand to the maximum depth of exploration, 51.5 feet bgs. The boring logs in Appendix A provide a more detailed description of the soils encountered in each boring, including the applicable Unified Soil Classification System symbols.

Encountered soils were found to have a low expansion potential and a low collapse potential.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

Based upon the data collected during this investigation and from a geotechnical engineering standpoint, it is our opinion that there are no soil conditions that would preclude the construction of the proposed improvements. The planned improvements may be supported on shallow reinforced concrete spread and/or continuous footings, provided the recommendations provided in this report are followed.

4.2 Soil Corrosivity

Based on test results, on-site, near-surface soils have low soluble sulfate and chloride contents, an increasingly moderate minimum resistivity, and are alkaline. Thus, on-site soils are considered to have a low corrosion potential with respect to buried concrete and a moderate corrosive potential for unprotected metal in contact with soil.

We recommend that Type I/Type II cement be used in the formulation of concrete, and that buried reinforcing steel protection be provided with a minimum concrete cover required by the American Concrete Institute (ACI) Building Code for Structural Concrete, ACI 318, Chapter 7.7. Buried metal conduits must have protective coatings in accordance with the manufacturer's specifications. If detailed recommendations for corrosion protection are desired, a corrosion specialist must be consulted.

4.3 Site Preparation and Earthwork Construction

The following procedures must be implemented during site preparation for the proposed building addition. It should be noted that references to maximum dry density, optimum moisture content, and relative compaction are based on ASTM D1557 (or latest test revision) laboratory test procedures.

1. Prior to any site grading, all miscellaneous surface obstructions must be removed from the improvement area. Near surface soils containing vegetation, roots, organics, or other objectionable material, and debris must be stripped to a depth of at least 3-inches to expose a clean soil surface. Where trees and bushes are to be removed, the associated roots are expected to extend 3 feet or more below existing grade, as such, deeper excavation may be necessary for root removal. Roots larger than ½-inch in diameter must be removed. Surface strippings must not be incorporated into engineered fill unless the organic content is less than 3 percent by weight (ASTM D2974).
2. Existing utilities or irrigation pipes must be removed to a point at least 5-feet horizontally outside the proposed improvement area. Resultant cavities must be backfilled with engineered fill. Abandoned pipelines to remain that are less than 2 inches in diameter should be capped at the cutoff point, while pipelines greater than 2 inches in diameter must be filled with a 1-sack sand-cement slurry.

3. Soil disturbed as a result of demolition, undocumented shallow fill, debris, abandoned underground structures must be excavated to expose undisturbed native soil.

4. Following the required demolition, stripping, and/or removal of debris and underground structures, the exposed soil surface in areas to support fill or proposed improvements must be over-excavated a minimum of 12 inches below existing site grade or 12 inches below bottom of proposed foundations, which ever depth is greater. The exposed subgrade soil must be proof rolled under the observation of a BSK field representative to detect soft or pliant areas. Soft or pliant areas must be over-excavated to firm native soil. The exposed surface must be scarified at minimum of 8 inches and uniformly moisture conditioned at near optimum moisture and compacted to 90 percent relative compaction.

   Earthwork must extend at least 5-feet laterally beyond the outside edge of proposed improvements and areas to receive fill.

5. Excavated soils, free of deleterious substances (organic matter, demolition debris, etc.) and with less than 3 percent organic content by weight, may be returned to the excavations as engineered fill. Engineered fill must be placed in uniform layers not exceeding 8-inches in loose thickness, moisture-conditioned to within 2 percent of optimum moisture content and compacted to at least 90 percent of the maximum dry density. The upper 12 inches of engineered fill placed as backfill under pavement sections must be compacted to at least 95 percent of the maximum dry density. Acceptance of engineered fill placement must be based on moisture content at time of compaction and relative compaction.

6. Imported fill materials must be free of deleterious substances and have less than 3 percent organic content by weight. The project specifications must require the contractor to contact BSK for review of the proposed import fill materials for conformance with these recommendations at least two weeks prior to importing to the site, whether from on-site or off-site borrow areas. Imported fill soils must be non-hazardous and be derived from a single, consistent soil type source conforming to the following criteria:

Maximum Particle Size:	3-inches
Percent Passing #4 Sieve:	65 – 100
Percent Passing #200 Sieve:	20 – 45
Plasticity Index:	less than 12
Expansion Index:	< 20
Low Corrosion Potential:	
Soluble Sulfates:	< 1,500 mg/kg
Soluble Chlorides:	< 300 mg/kg
Soil Resistivity:	> 3,000 ohm-cm

The Department of Toxic Substance Control (DTSC) has detailed guidelines for the testing of import soils to school sites. These guidelines take into account the past and present land usage at a borrow pit, the acreage of the borrow pit and the volume of import soil to establish the amount of chemical testing of import fill recommended. BSK must be contacted for review and analytical testing of proposed import fill materials for conformance with these recommendations at least 15 days prior to transporting fill to the site.

Grading operations must be scheduled as to avoid working during periods of inclement weather. Should these operations be performed during or shortly following periods of inclement weather, unstable soil conditions may result in the soils exhibiting a "pumping" condition. This condition is caused by excess moisture, in combination with compaction, resulting in saturation and near zero air voids in the soils. If this condition occurs, the affected soils must be over-excavated to the depth at which stable soils are encountered and replaced with suitable soils compacted as engineered fill. Alternatively, the Contractor may proceed with grading operations after utilizing a method to stabilize the soil subgrade, which must be subject to review by BSK prior to implementation.

4.4 Shallow, Mat, and Pole-Type Foundations

Provided the recommendations contained in this report are implemented during design and construction, it is our opinion that the proposed structures can be supported on shallow, mat, or pole-type foundations. A structural engineer must evaluate reinforcement and embedment depth based on the requirements for the structural loadings.

4.4.1 Shallow Foundations

The proposed at-grade structures may be supported on reinforced concrete spread footings bearing on engineered fill. The allowable bearing pressure applies to the dead load plus live load (DL + LL) condition and includes a factor of safety of 3. Footing design must follow the criteria listed below:

Table 1: Allowable Bearing Pressure

Footing Embedment(1) (inches)	Minimum Footing Width (inches)		Allowable Bearing Capacity(2) (psf)	
	Continuous Footing	Isolated Spread Footing	Continuous Footing	Isolated Spread Footing
12	18	24	3,000	3,000

Note (1) – Measure with respect to the lowest adjacent subgrade surface.
(2) – The bearing pressure can be increased one-third for transient loading such as wind or seismic.

The estimated total and differential settlement for the recommended spread footings is shown below:

Table 2: Anticipated Post-Construction Settlement			
Footing Type	Post-Construction Settlement (inches)	Differential Settlement (inches)	Angular Distortion
Continuous	1.0	--	0.005
Isolated	1.0	0.5	--

Isolated footing differential settlement is based on adjacent similarly loaded footings spaced at 30-feet. The settlement values given above are applicable to the maximum loading conditions. For loads, other than the design maximum loads, the settlements can be decreased proportionally.

4.4.2 Mat Foundations

Miscellaneous structures may be supported on a thickened mat/slab foundation. The foundation may be designed for a maximum allowable bearing pressure of 2,000 psf (DL + LL). The bearing pressure shall be permitted to be increased by 1/3 where used with the alternative basic load combinations of CBC Section 1605A.3.2 that include wind or earthquake loads. Estimated total settlement for mat/slabs is approximately 1.0 inch. Differential settlement across mat/slab foundations is anticipated to be on the order of about half of the total settlement over the length of the mat foundation. The weight of the concrete should be included in evaluating the contact pressure at the base of mat/slab foundations. The weight of embedded concrete can be reduced by the unit weight of soil times the depth of embedded concrete.

Mat foundations must be a minimum of 4-inches thick and must be supported on a compacted subgrade prepared in accordance with the "Site Preparation and Earthwork Construction" section of this report. In order to regulate cracking of the slabs, construction joints and/or saw-cut control joints must be provided in each direction at a maximum spacing of 10 feet on centers along with steel reinforcement as recommended by the project's Structural Engineer. Control joints must have a minimum depth of one-quarter of the slab thickness. It is recommended that steel reinforcement used in concrete slabs-on-grade consist of steel rebar. Structural concrete slabs-on-grade may be designed using an unadjusted long-term Modulus of Subgrade Reaction (Ks) of 60 pounds per cubic inch (pci) constructed on a properly compacted subgrade or engineered fill. This value is based on the correlations to soil strength using one foot by one-foot plate-load tests and should therefore be scaled (adjusted) to the actual slab width. The adjusted Ks value can be obtained by multiplying the value provided above by $[(B+B_1)/(2B)]^2$, where B is the slab width in feet and B_1 is 1 foot (width of a one foot by one foot plate-load test apparatus).

4.4.3 Pole Type Foundations

Structures such as stadium lighting, signs, etc. may be supported on pole type foundations. This type of foundation must be designed in accordance with Section 1807A.3 of the 2022 CBC. However, it is recommended that an allowable lateral soil bearing pressure of 210 psf per foot of embedment be used to develop parameters S_1 and S_3 rather than one of the values given in Table 1806A.2. This value includes a factor of safety of 2 and may be increased as indicated by 1806A.3 and the footnotes to Table 1806A.2. Unless the area surrounding the pole foundation is paved or covered with concrete flatwork, the upper 24 inches of soil should be ignored when calculating the minimum depth of embedment.

The following table provides expressions for the allowable and ultimate axial capacity using friction to resist axial loads. The skin friction within the upper two feet of embedded length must be ignored in unpaved areas. The total settlement of pier foundations designed in accordance with these recommendations should not exceed one-half inch.

Table 3: Friction Resistance for Vertical Loads	
Allowable (lbs)	**Ultimate (lbs)**
50 DL^2	125 DL^2

Note (1) – D is pile diameter (feet), and L is the total embedment length feet).

Prior to placing concrete, loose or disturbed soils must be removed from the bottom of the drilled pier excavations using a flat bottom clean-out bucket or other pre-approved method. A representative of BSK must observe the drilling and clean-out associated with the construction of pier foundations in order to assess whether the actual bearing conditions are compatible with the conditions anticipated during the preparation of this report. Relatively cohesionless soils were observed within the borings. To aid in the excavation for pole footings, consideration should be given to utilizing casing or mud drilling techniques to prevent/minimize potential caving. A representative of BSK must observe the drilling and clean-out associated with the construction of pier foundations in order to assess whether the actual bearing conditions are compatible with the conditions anticipated during the preparation of this report.

Pier deflection may govern the design lateral resistance. If provided with pier geometry, lateral load, and loading eccentricity, the estimated pier head deflection can be provided.

4.5 Lateral Earth Pressures and Frictional Resistance

Lateral loads applied against foundations may be resisted by a combination of passive resistance against the vertical faces of the foundations and friction between the foundation bottom and the supporting subgrade. An unfactored coefficient of friction of 0.73 may be used between soil subgrade and cast-in-place foundation bottom. The unfactored passive pressure is presented in Table 4. The coefficient of friction and passive earth pressure values given above represent ultimate soil strength values. BSK recommends that a safety factor consistent with the design conditions be included in their usage. For resistance against lateral sliding that is countered solely by the passive earth pressure against footings or friction along the bottom of footings, a minimum safety factor of 1.5 is recommended. For stability against lateral sliding that is resisted by combined passive pressure and frictional resistance, a minimum safety factor of 2.0 is recommended. For lateral resistance against seismic loading conditions, a minimum safety factor of 1.2 is recommended. We based these lateral resistance values on the assumption that the concrete for the foundations is either placed directly against undisturbed soils or that the voids created from the use of forms are backfilled with engineered fill or other approved materials, such as lean concrete. Passive resistance in the upper foot of soil cover below finished grades should be neglected unless the ground surface is confined by concrete slabs, pavements, or other such positive protection.

The following earth pressure parameters may be used for designing earth retaining structures and foundations.

Table 4: Lateral Earth Pressures

Lateral Pressure Conditions	Equivalent Fluid Pressure
Active Pressure	35 psf/ft
At-Rest Pressure	55 psf/ft
Passive Pressure	425 psf/ft
Dynamic Increment	7.2H psf

Notes: 1. H is wall height in feet

Parameters are shown in the above table for drained conditions of select engineered fill or prepared native soil. In addition, the drained condition assumes that positive drainage will be provided away from the structure improvements and that water does not accumulate around the structure and cause a build-up of hydrostatic pressure.

4.6 Concrete Slabs-on-Grade

Non-structural concrete slab-on-grade must be a minimum of 4-inches thick and must be supported on a compacted subgrade prepared in accordance with the "Site Preparation and Earthwork Construction" section of this report. Existing onsite surface soils are considered to have a very low expansion potential. For design purposes, in order to regulate cracking of the slabs, construction joints and/or saw-cut control joints must be provided in each direction at a maximum spacing of 10 feet on centers along with steel reinforcement as recommended by the project's Structural Engineer. Control joints must have a minimum depth of one-quarter of the slab thickness. It is recommended that steel reinforcement used in concrete slabs-on-grade consist of steel rebar. Structural concrete slabs-on-grade may be designed using an unadjusted long-term Modulus of Subgrade Reaction (Ks) of 150 pounds per cubic inch (pci) constructed on a properly compacted subgrade or engineered fill. This value is based on the correlations to soil strength using one foot by one foot plate-load tests and should therefore be scaled (adjusted) to the actual slab width. Field and laboratory tests were not performed to establish the Ks value provided herein. For sand soils, such as those found at this site, the adjusted Ks value can be obtained by multiplying the value provided above by $[(B+B_1)/(2B)]^2$, where B is the slab width in feet and B_1 is 1 foot (width of a one foot by one foot plate-load test apparatus).

Interior concrete slabs must be successively underlain by: 1-½ inches of washed concrete sand; a durable vapor barrier; and a smooth, compacted subgrade surface. The vapor barrier must meet the requirements of ASTM: E1745 Class A and have a water vapor transmission rate (WVTR) of less than or equal to 0.012 Perms as tested by ASTM: E96. Examples of acceptable vapor barrier products include: Stego Wrap (15-mil) Vapor Barrier by STEGO INDUSTRIES LLC; W.R. Meadows Premoulded Membrane with Plasmatic Core; and Zero-Perm by Alumiseal. Because of the importance of the vapor barrier, joints must be carefully spliced and taped.

If migration of subgrade moisture through the slab is not a concern, then the vapor barrier and overlying sand may be omitted. The slab subgrade must be kept in a moist condition until the vapor barrier or concrete slab is placed. BSK's representative must be called to the site to review soil and moisture conditions immediately prior to placing the vapor barrier or concrete slab.

As indicated in the PCA Engineering Bulletin 119, Concrete Floors and Moisture, and applicable ACI Committee reports (see ACI 360R-06, Design of Slabs-on-Ground, dated October 2006 and ACI 302.1R-04, Guide for Concrete Floor and Slab Construction, dated June 2004), the sand layer between the vapor barrier and concrete floor slab may be omitted. The advantage of this option is that it can reduce the amount of moisture that can be transmitted through the slab (especially if the sand layer becomes moist or wet prior to placing the concrete); however, the risk of slab "curling" is much greater. The "curling" may result from a sharp contrast in moisture-drying conditions between the exposed slab surface and the surface in contact with the membrane. As recommended in the referenced ACI Committee reports, measures must be taken to reduce the risk of "curling" such as reducing the joint spacing, using a low shrinkage mix design, and reinforcing the concrete slab. In order to regulate cracking of the slab, we recommend that full depth construction joints and control joints be provided in each direction with slab thickness and steel reinforcing recommended by the structural engineer.

Excessive landscape water or leaking utility lines could create elevated moisture conditions under concrete slabs, which could result in adverse moisture or mildew conditions in floor slabs or walls. Accordingly, care must be taken to avoid excess irrigation around the structures, as well as to periodically monitor for leaking utility lines. Likewise, positive surface drainage must be provided around the perimeter of the structures as discussed in the "Surface Drainage Control" section 4.11.

The adverse effects of moisture vapor transmission on flooring materials can be substantially reduced by the use of a low porosity concrete. This can be achieved by specifying a low water-cement ratio (0.45 or less by weight) a minimum compressive strength of 4,000 psi at 28 days, and a minimum of 7 days wet-curing.

4.7 Conventional Pavement Section Recommendations

R-value testing was completed on two samples based on the predominate soil types encountered at borings B-2 from 0 to 5 feet bgs. BSK recommends a design R-value of 49.

BSK calculated the conventional pavement section thicknesses using a design subgrade R-Value of 49 for traffic Indexes of 5 through 9. BSK has presented a summary of its pavement section thickness recommendations in Table 2, *Conventional Pavement Section Recommendations.*

TABLE 2: Conventional Pavement Section Recommendations (R-Value = 49, 20-yr design life)

Traffic Index	Conventional Section	
	HMA (inches)	AB (inches)
5.0	3	4
5.5	3	4
6.0	3.5	4
6.5	3.5	4
7.0	4	5
7.5	4.5	5
8.0	5	5.5
8.5	5	6.5
9.0	5.5	6.5

Notes:
HMA: Hot Mix Asphalt
AB: Caltrans Class 2 Aggregate Base (Minimum R-Value = 78)

Hot mix asphalt and Class 2 aggregate base should conform to and be placed in accordance with the latest revision of Caltrans Standard Specifications. It is recommended subgrade be scarified to a depth of 12 inches, moisture conditioned and compacted to at least 95 percent maximum density, based on ASTM D1557 prior to placing new aggregate base/subbase section.

4.8 Excavation Stability

The slopes surrounding or along temporary excavations should be no steeper than 2H:1V for excavations that are less than 5-feet deep and exhibit no indication of instability. If clean sand layers are encountered, slopes should be laid back. Temporary excavations for the project construction must be left open for as short a time as possible and must be protected from water runoff. In addition, equipment and/or soil stockpiles must be maintained at least 5 feet or a distance equal to the depth of excavation, whichever distance is greater, away from the top of the excavations. Slope height, slope inclination, and excavation depths (including utility trench excavations) must in no case exceed those specified in local, state, or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations 29 CFR Part 1926, or successor regulations). These excavation recommendations are based on soil characteristics derived from the test boring. Variations in soil conditions will likely be encountered during excavation. At the time of construction, BSK must be afforded the opportunity to observe and document sloping and shoring conditions, and the opportunity to provide review of actual field conditions to account for condition variations not otherwise anticipated in the preparation of these recommendations.

4.9 Utility Trench Excavation and Backfill

Pipes and conduits must be bedded and shaded in accordance with the requirements of the pipe manufacturer. Where no specific requirements exist, we recommend a minimum of 6-inches of sand bedding material for pipe installations greater than 12-inches in diameter. For pipe diameters smaller than 12-inches, the bedding thickness may be reduced to 4-inches. The bedding material and envelope (up to 6-inches above the pipe) must consist of sand (Sand Equivalent greater than 30), be placed in loose lifts not exceeding 8-inches in thickness, compacted to at least 90 percent of the maximum dry density, and moisture conditioned to within 2 percent of optimum moisture content. Water jetting to attain compaction must not be allowed.

Adequate excavation width must be provided to permit uniform compaction on both sides of utility lines installed within the trench. The trench backfill material may consist of engineered fill. Trench backfill outside the building footprint must be placed in loose lifts not to exceed 8-inches in loose thickness, compacted to at least 90 percent of the maximum dry density, and moisture conditioned to within 2 percent of optimum moisture content. The upper 12-inches of trench backfill below pavement sections must be compacted to at least 95 percent of the maximum dry density. Conduits extending through or below footings must be “sleeved” as determined by the Project Structural Engineer. Utility trench backfill beneath the building areas must be backfilled in accordance with Section 4.3 (Site Preparation and Earthwork Construction).

4.10 Surface Drainage Control

Final grading around site improvements must provide for positive and enduring drainage away from the building foundations. Ponding of water must not be allowed on or near the building or paved surfaces. Saturation of the soils immediately adjacent to or below the building area must not be allowed. Irrigation water must be applied in amounts not exceeding those required to offset evaporation, sustain plant life, and maintain a relatively uniform moisture profile around and below, site improvements.

5 PLANS AND SPECIFICATIONS REVIEW

BSK recommends that it be retained to review the draft plans and specifications for the project, with regard to foundations and earthwork, prior to their being finalized and issued for construction bidding.

6 CONSTRUCTION TESTING AND OBSERVATIONS

Geotechnical testing and observation during construction is a vital extension of this geotechnical investigation. BSK recommends that it be retained for those services. Field review during site preparation and grading allows for evaluation of the exposed soil conditions and confirmation or revision of the assumptions and extrapolations made in formulating the design parameters and recommendations. BSK's observations must be supplemented with periodic compaction tests to establish substantial conformance with these recommendations. BSK must also be called to the site to observe foundation excavations, prior to placement of reinforcing steel or concrete, in order to assess whether the actual bearing conditions are compatible with the conditions anticipated during the preparation of this report. BSK must also be called to the site to observe placement of foundation and slab concrete.

If a firm other than BSK is retained for these services during construction, that firm must notify the owner, project designers, governmental building officials, and BSK that the firm has assumed the responsibility for all phases (i.e., both design and construction) of the project within the purview of the geotechnical engineer. Notification must indicate that the firm has reviewed this report and any subsequent addenda, and that it either agrees with BSK's conclusions and recommendations, or that it will provide independent recommendations.

7 LIMITATIONS

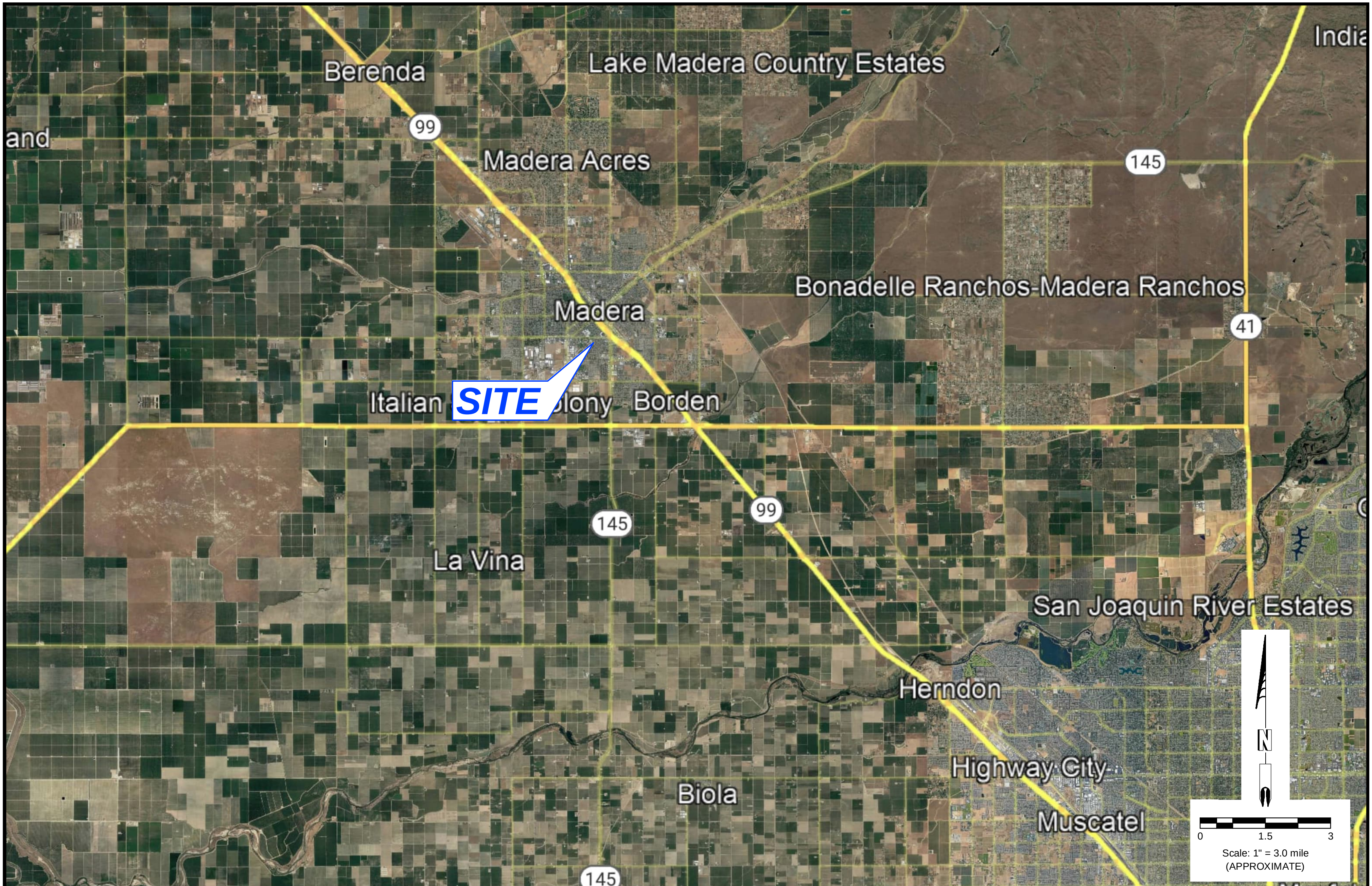
The analyses and recommendations submitted in this report are based upon the data obtained from the current investigation at locations shown on Figure 2 and data presented in the referenced reports. The report does not reflect variations which may occur between or beyond the borings. The nature and extent of such variations may not become evident until additional exploration and testing is performed or construction is initiated. If variations then appear, a re-evaluation of the recommendations of this report will be necessary after performing on-site observations during the excavation period and noting the characteristics of the variations.

The validity of the recommendations contained in this report is also dependent upon an adequate testing and observation program during the construction phase. BSK assumes no responsibility for construction compliance with the design concepts or recommendations unless it has been retained to perform the testing and observation services during construction as described above.

The findings of this report are valid as of the present. However, changes in the conditions of the site can occur with the passage of time, whether caused by natural processes or the work of man, on this property or adjacent property. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation, governmental policy or the broadening of knowledge.

The report has been prepared in accordance with generally accepted geotechnical engineering practices which existed in Madera County at the time the report was written. No other warranties either express or implied are made as to the professional advice provided under the terms of BSK's agreement with Client and included in this report.

FIGURES



C:\Users\tgonzalez\BSK Associates\BSK Documents - GEO\G00001343-Madison ES 2-Story Building\Graphics\G00001343.dwg User:tgonzalez Plotted:Aug 02, 2023 - 11:48am Last Save:Jul 20, 2023 - 10:34am

REFERENCE IMAGE: Google Earth

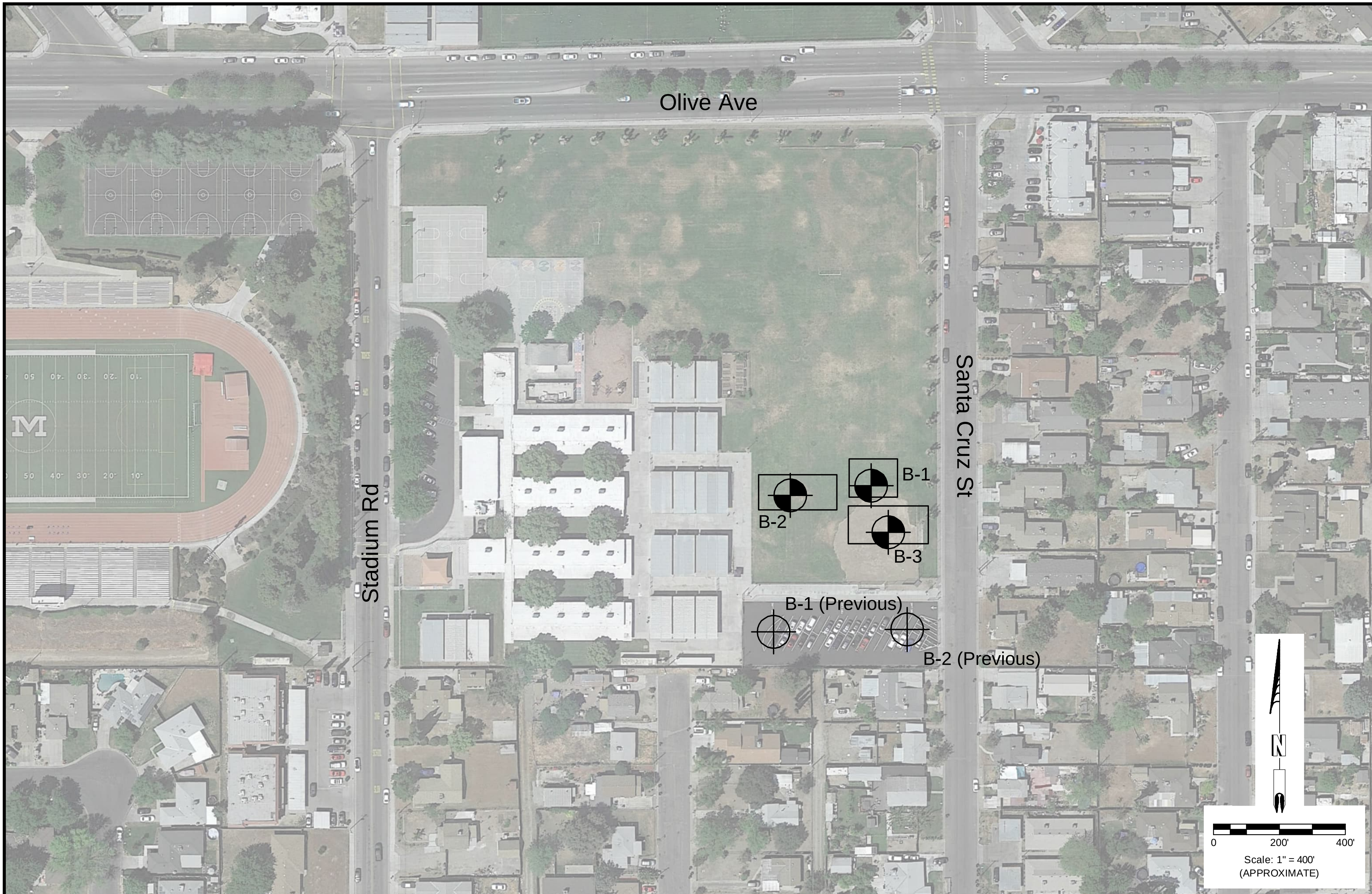
BSK ASSOCIATES
691 N. Laverne Avenue
Fresno, California 93727
Tel. (559) 497-2880

SITE VICINITY MAP

Madison ES 2-Story Building
Madera, California

FIGURE 1

JOB NO.	G00-001-343
DATE	August 2, 2023
DR. BY	TG
CH. BY	NP
SCALE AS SHOWN	
SHEET NO.	1
OF	1 SHEETS



LEGEND:

B-3 APPROXIMATE BORING LOCATIONS

B-2 (Previous) APPROXIMATE PREVIOUS BORING LOCATIONS (G21-354-11F)

REFERENCE IMAGE: Google Earth

BSK ASSOCIATES
691 N. Laverne Avenue
Fresno, California 93727
Tel. (559) 497-2880

BORING LOCATION MAP

Madison ES 2-Story Building
Madera, California

FIGURE 2

JOB NO. G00-001-343
DATE August 2, 2023
DR. BY TG
CH. BY NP
SCALE AS SHOWN
SHEET NO. 1
OF 1 SHEETS

APPENDIX A
FIELD EXPLORATION

APPENDIX A
Field Exploration

The field exploration, conducted on July 26, 2023, consisted of a site reconnaissance and drilling three (3) exploratory test borings. The test borings were drilled to depths of approximately 21.5 to 51.5 feet below ground surface (bgs). The test borings were drilled with a truck-mounted drill rig, equipped with manually advanced 8-inch augers. The approximate boring locations are presented on Figure 2, Boring Location map.

The soil materials encountered in the test boring were visually classified in the field and a log was recorded during the excavation and sampling operations. Visual classification of the materials encountered in the test boring was made in general accordance with the Unified Soil Classification System (ASTM D2487). A soil classification chart is presented herein. Boring logs are presented herein and should be consulted for more details concerning subsurface conditions. Stratification lines were approximated by the field staff on the basis of observations made at the time of excavation while the actual boundaries between different soil types may be gradual and soil conditions may vary at other locations.

For the hollow stem auger drilling, subsurface samples were obtained at the successive depths shown on the boring logs by driving samplers which consisted of a 2.5-inch inside diameter (I.D.) California Sampler and a 1.4-inch I.D. Standard Penetration Test (SPT) Sampler. The samplers were driven 18-inches using a 140-pound hammer dropped from a height of 30-inches by means of either an automatic hammer or a down-hole “safety hammer”. The number of blows required to drive the last 12-inches was recorded as the blow count (blows/foot) on the boring logs. The relatively undisturbed soil core samples were capped at both ends to preserve the samples at their natural moisture content. Soil samples were also obtained using the SPT Sampler (marked X in logs) lined with metal tubes or unlined in which case the samples were placed and sealed in polyethylene bags. At the completion of the field exploration, the test borings were backfilled with the excavated soil cuttings.

It should be noted that the use of terms such as “loose,” “medium dense,” “dense” or “very dense” to describe the consistency of a soil is based on sampler blow count and is not necessarily reflective of the in-place density or unit weight of the soils being sampled. The relationship between sampler blow count and consistency is provided in the following Tables A-1 and A-2 for coarse-grained (sandy and gravelly) soils and fine grained (silty and clayey) soils, respectively.

Table A-1: Density of Coarse-Grained Soil versus Sampler Blow Count		
Consistency	**SPT Blow Count Blows / Foot)**	**2.5” I.D. Cal. Sampler (Blows / Foot)**
Very Loose	<4	<6
Loose	4 – 10	6 – 15
Medium Dense	10 – 30	15 – 45
Dense	30 – 50	45 – 80
Very Dense	>50	>80

Table A-2: Consistency of Fine-Grained Soil versus Sampler Blow Count		
Consistency	**SPT Blow Count (Blows / Foot)**	**2.5” I.D. Cal. Sampler (Blows / Foot)**
Very Soft	<2	<3
Soft	2 – 4	3 – 6
Medium Stiff	4 – 8	6 – 12
Stiff	8 – 15	12 – 24
Very Stiff	15 – 30	24 – 45
Hard	>30	>45

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 15% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 15% FINES	SM	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

■	Modified California	RV	R-Value
	Standard Penetration Test (SPT)	SA	Sieve Analysis
☒	Split Spoon	SW	Swell Test
	Pushed Shelby Tube	TC	Cyclic Triaxial
	Auger Cuttings	TX	Unconsolidated Undrained Triaxial
	Grab Sample	TV	Torvane Shear
	Sample Attempt with No Recovery	UC	Unconfined Compression
CA	Chemical Analysis	(1.2)	(Shear Strength, ksf)
CN	Consolidation	WA	Wash Analysis
CP	Compaction	(20)	(with % Passing No. 200 Sieve)
DS	Direct Shear	▽	Water Level at Time of Drilling
PM	Permeability	▼	Water Level after Drilling(with date measured)
PP	Pocket Penetrometer		

SOIL CLASSIFICATION CHART AND LOG KEY

BSK ASSOCIATES

LGD A NNNN02 TEST.GPJ 5/10/18

BSK ASSOCIATES
BSK Associates
691 N Laverne Ave. Suite 101
Fresno, CA 93727
Telephone: (559) 497-2880

Project: Madison ES 2-Story Building
Location: Madera, CA
Project No.: G00-001-343
Logged By: D. Ferrer
Checked By: N. Popenoe

Boring: **B-1**

Depth (Feet)	Samples	Bulk Samples	Penetration Blows / Foot	In-Situ Dry Density (pcf)	In-Situ Moisture Content (%)	% Passing No. 200 Sieve	Graphic Log	USCS	MATERIAL DESCRIPTION	REMARKS
0		Bulk (0–5)						SC	**Clayey SAND** - brown, moist, fine to coarse grained sand, trace silt	
4.5								SM	**Silty SAND** - brown, moist, fine to coarse grained sand, trace clay, loose	
5–6.5	Sample		8	99	19.0					
7.5								SC	**Clayey SAND** - brown, moist, fine to coarse grained sand, loose	
10–11.5	Sample		11	120	13.7					
13.5								CL	**Sandy CLAY** - brown, moist, fine to coarse grained sand, very stiff	
15–16.5	SPT		30							
20–21.5	Sample		58	116.4	11.2					
25–26.5	SPT		17							

Drilling Contractor: Baja Exploration
Drilling Method: Hollow Stem Auger
Drilling Equipment: CME 75
Date Started: 7/26/23
Date Completed: 7/26/23

Surface Elevation:
Sample Method: 2.5" Modified Cal & 1.5" I.D. SPT Split Spoon
Groundwater Depth: Not Encountered
Completion Depth: 51.5 Feet
Borehole Diameter: 8"

* See key sheet for symbols and abbreviations used above.

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Fresno, CA 93727
Telephone: (559) 497-2880

Project: Madison ES 2-Story Building
Location: Madera, CA
Project No.: G00-001-343
Logged By: D. Ferrer
Checked By: N. Popenoe

Boring: **B-1**

Depth (Feet)	Samples	Bulk Samples	Penetration Blows / Foot	In-Situ Dry Density (pcf)	In-Situ Moisture Content (%)	% Passing No. 200 Sieve	Graphic Log	USCS	MATERIAL DESCRIPTION	REMARKS
31			50/ 5.5"						**Sandy CLAY** - brown, moist, fine to coarse grained sand, very stiff *(continued)* ... decreasing sand content	
36			39							
41			45						... increasing sand content	
45								SP	**Poorly Graded SAND** - yellowish brown, moist, fine to coarse grained sand, medium dense	
46			25							
51			32	101	5					
52									Boring terminated at approximately 51.5 feet bgs. No groundwater encountered. Boring backfilled with soil cuttings.	

Drilling Contractor: Baja Exploration
Drilling Method: Hollow Stem Auger
Drilling Equipment: CME 75
Date Started: 7/26/23
Date Completed: 7/26/23

Surface Elevation:
Sample Method: 2.5" Modified Cal & 1.5" I.D. SPT Split Spoon
Groundwater Depth: Not Encountered
Completion Depth: 51.5 Feet
Borehole Diameter: 8"

* See key sheet for symbols and abbreviations used above.

BSK Associates
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Fresno, CA 93727
Telephone: (559) 497-2880

Project: Madison ES 2-Story Building
Location: Madera, CA
Project No.: G00-001-343
Logged By: D. Ferrer
Checked By: N. Popenoe

Boring: **B-2**

Depth (Feet)	Samples	Bulk Samples	Penetration Blows / Foot	In-Situ Dry Density (pcf)	In-Situ Moisture Content (%)	% Passing No. 200 Sieve	USCS	MATERIAL DESCRIPTION	REMARKS
0							ML	**Sandy SILT** - brown, moist, fine to coarse grained sand, medium stiff	
2–3.5	■	0–5	11	98	12				
5–6.5	■		11	87	15				
7.5							CL	**Sandy CLAY** - brown, moist, fine to coarse grained sand, stiff	
10–11.5	■		22	124	10				
14.5							SC	**Clayey SAND** - brown, moist, fine to coarse grained sand, stiff	
15–16.5	⧗		17						
20–21.5	■		35					... decreasing sand content	
21.5								Boring terminated at approximately 21.5 feet bgs. No groundwater encountered. Boring backfilled with soil cuttings.	

Drilling Contractor: Baja Exploration
Drilling Method: Hollow Stem Auger
Drilling Equipment: CME 75
Date Started: 7/26/23
Date Completed: 7/26/23

Surface Elevation:
Sample Method: 2.5" Modified Cal & 1.5" I.D. SPT Split Spoon
Groundwater Depth: Not Encountered
Completion Depth: 21.5 Feet
Borehole Diameter: 8"

* See key sheet for symbols and abbreviations used above.

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BSK Associates
691 N Laverne Ave. Suite 101
Fresno, CA 93727
Telephone: (559) 497-2880

Project: Madison ES 2-Story Building
Location: Madera, CA
Project No.: G00-001-343
Logged By: D. Ferrer
Checked By: N. Popenoe

Boring: **B-3**

Depth (Feet)	Samples	Bulk Samples	Penetration Blows / Foot	In-Situ Dry Density (pcf)	In-Situ Moisture Content (%)	% Passing No. 200 Sieve	Graphic Log	USCS	MATERIAL DESCRIPTION	REMARKS
0–7.5								ML	**Sandy SILT** - brown, moist, fine to coarse grained sand, stiff	
2–3.5	■		13	110	8					
5–6.5	■		28						... trace clay	
7.5–13.5								SC	**Clayey SAND** - brown, moist, fine to coarse grained sand, stiff	
10–11.5	■		23							
13.5–21.5								CL	**Sandy CLAY** - brown, moist, fine to coarse grained sand, hard	
15–16.5	■		50/ 4"							
20–21.5	■		30							
21.5									Boring terminated at approximately 21.5 feet bgs. No groundwater encountered. Boring backfilled with soil cuttings.	

Drilling Contractor: Baja Exploration
Drilling Method: Hollow Stem Auger
Drilling Equipment: CME 75
Date Started: 7/26/23
Date Completed: 7/26/23

Surface Elevation:
Sample Method: 2.5" Modified Cal & 1.5" I.D. SPT Split Spoon
Groundwater Depth: Not Encountered
Completion Depth: 21.5 Feet
Borehole Diameter: 8"

* See key sheet for symbols and abbreviations used above.

APPENDIX B
LABORATORY TESTING

APPENDIX B
Laboratory Testing

The results of laboratory testing performed in conjunction with this project are contained in this Appendix. The following laboratory tests were performed on soil samples in general conformance with applicable standards.

In-Situ Moisture and Density

The field moisture content and in-place dry density determinations were performed on a relatively undisturbed samples obtained from the test borings. The field moisture content, as a percentage of dry weight of the soils, was determined by weighing the samples before and after oven drying in accordance with ASTM D2216 test procedures. Dry densities, in pounds per cubic foot, were also determined for undisturbed core samples in accordance with ASTM D2937 test procedures. Test results are presented on the boring logs in Appendix A.

Direct Shear Test

One (1) direct shear test was performed on selected soil specimens. The three-point shear tests were performed in general accordance with ASTM D3080, Direct Shear Test for Soil under Consolidated Drained Conditions. The test specimens, each 2.42 inches in diameter and 1 inch in height, were subjected to shear along a plane at mid-height after allowing for pore pressure dissipation. The results of the tests are presented on Figure B-1.

Collapse Potential Test

One (1) collapse potential test weas performed on a relatively undisturbed soil sample to evaluate compressibility and collapse potential characteristics. The test was performed in general accordance with ASTM D 2435. The sample was initially loaded under as-received moisture content to a selected stress level, saturated, and then incrementally loaded up to a maximum load of 4 ksf. The test results are presented on Figure B-2.

Expansion Index Test

One expansion index test was performed in general accordance with ASTM D-4829. The specimen was moisturized and compacted to a dry density and moisture content corresponding to a degree of saturation between 48 to 52 percent, was subjected to a 1-PSI normal load and then saturated. The vertical movement of the specimen was monitored during the process. The test results are presented on Figure B-3.

R-Value Test

The Resistance-Value of one (1) sample of the surficial soil was tested in accordance with California Department of Transportation's Test Method CT 301. The results of the R-Value test are presented on Figure B-4.

Soil Corrosivity

The results of chemical analyses performed on a bulk soil sample using CT 643 (for minimum resistivity and PH) and CT 417 and 422 (for soluble sulfate and chlorides, respectively).

SUMMARY OF CHEMICAL TEST RESULTS

Sample Location	pH	Sulfate (mg/kg)	Chloride (mg/kg)	Minimum Resistivity (ohms-cm)
B-1 @ 0 – 5'	6.36	Not Detected	25	7,730

BSK ASSOCIATES

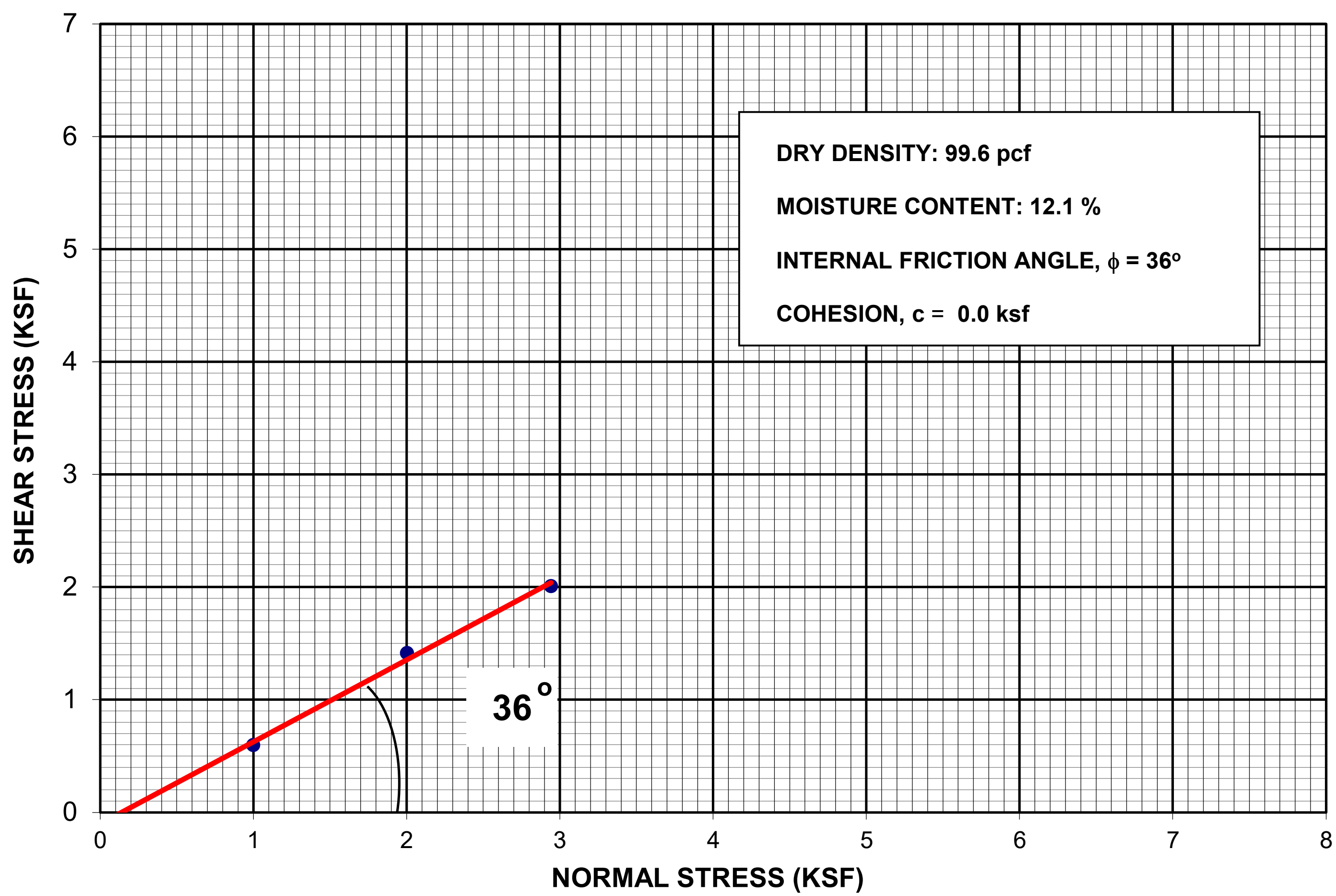
FIGURE B-1
691 N Laverne, Ste 101
Fresno, CA 93727
Ph: (559) 497-2880

Direct Shear Test

ASTM D-3080

Project Name:	Madison ES	**Sampled By:**	D. Ferrer	**Sample Date:**	7/26/23
		Tested By:	D. Messfin	**Test Date:**	7/28/23
Project Number:	G00001343	**Lab Tracking ID:**	N/A	**Report Date:**	8/14/23
Sample Location:	B-2 @ 3'	**Sample Description:**	Sandy Silt (ML) - brown, moist, fine to coarse grained sand		

SHEAR STRENGTH DIAGRAM

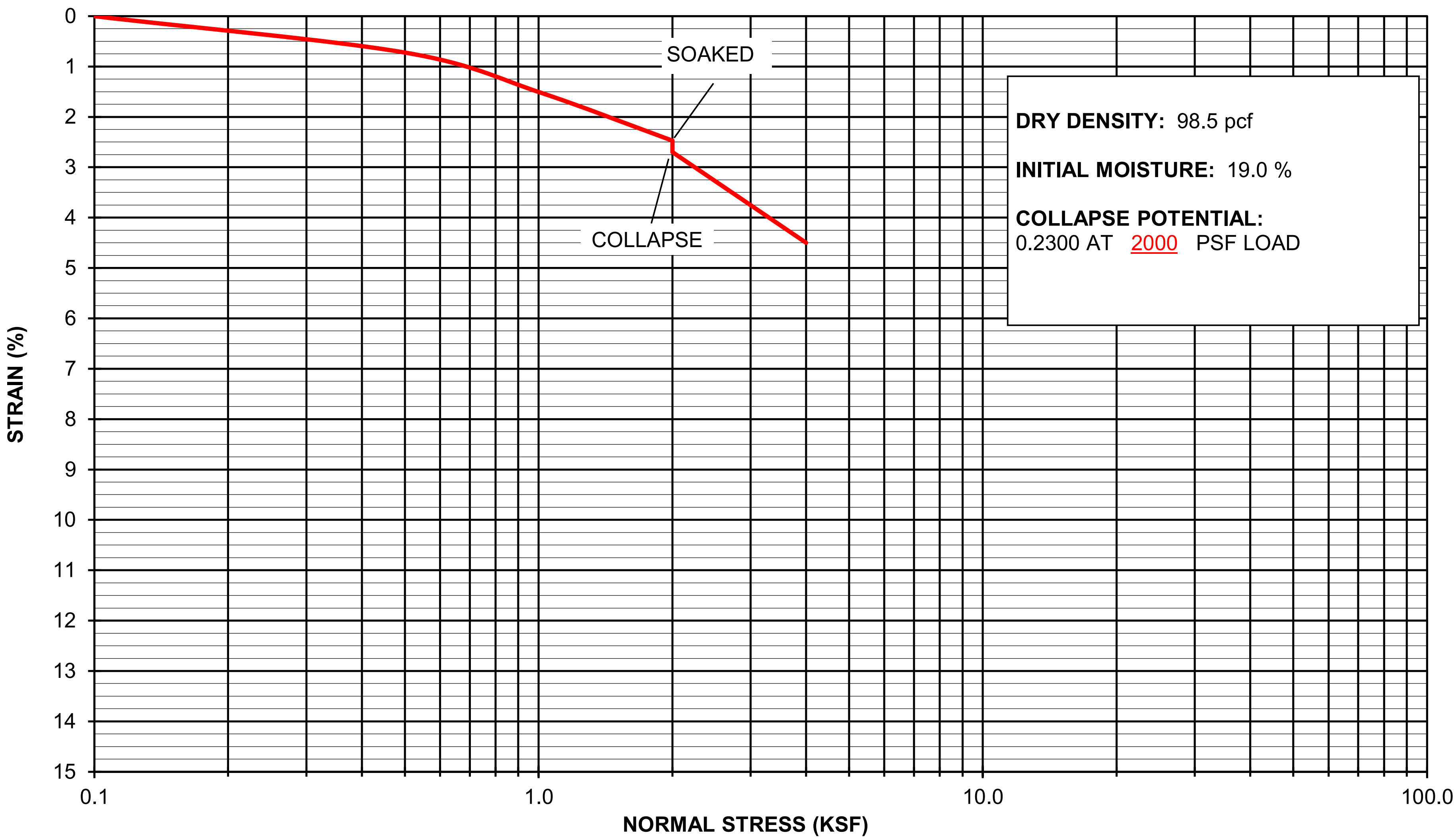


BSK ASSOCIATES

COLLAPSE POTENTIAL
ASTM D-5333

FIGURE B-2
691 N. Laverne, Suite 101
Fresno, CA 93727
Ph: (559) 497-2880

Project Name:	Madison ES	**Sampled By:**	D.F	**Sample Date:**	7/26/23
		Tested By:	D.Messfin	**Test Date:**	8/1/23
Project Number:	G00001343	**Lab Tracking ID:**	N/A	**Report Date:**	8/14/23
Sample Location:	B-1 @ 6'	**Sample Description:**	Sandy Silt (ML) - brown, moist, fine to coarse grained sand, trace clay		



DRY DENSITY: 98.5 pcf

INITIAL MOISTURE: 19.0 %

COLLAPSE POTENTIAL:
0.2300 AT 2000 PSF LOAD

ASSOCIATES

Expansion Index of Soils

ASTM D 4829 / UBC Standard 18-2

FIGURE B-3

691 N. Laverne, Suite 101
Fresno, CA 93727
Ph: (559) 497-2868

Project Name: **Madison ES Solar** **Report Date:** 1/3/23
Project Number: G00001343 **Sample Date:** 12/19/22
Lab Tracking ID: N/A **Test Date:** 12/22/22
Sample Location: B-1 @ 0' - 5'
Sample Source Bulk
Sampled By: D.F **Tested By:** D.Messfin **Reviewed By:** N. Popenoe

TEST DATA

INITIAL SET-UP DATA			
Sample + Tare Weight (g)	760.0		
Tare Weight (g)	368.5	**FINAL TAKE-DOWN DATA**	
		Moisture Content Data	
Wet Weight + Tare	117.6	Wet Weight + Tare	602.2
Dry Weight + Tare	106	Dry Weight + Tare	533.5
Tare Weight (g)	0	Tare Weight (g)	180.1
Moisture Content (%)	11.0%	Moisture Content (%)	19.4%
Initial Volume (ft^3)	0.007272	Final Volume (ft^3)	0.007314
Remolded Wet Density (pcf)	118.7	Final Wet Density (pcf)	127.0
Remolded Dry Density (pcf)	107.0	Final Dry Density (pcf)	106.4
Degree of Saturation	**51**	**Degree of Saturation**	**90**

EXPANSION READINGS

Initial Gauge Reading (in)	0.2844
Final Gauge Reading (in)	0.2901
Expansion (in)	0.0057

Uncorrected Expansion Index	**6**
Corrected Expansion Index, EI	**6**

Classification of Expansive Soil

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

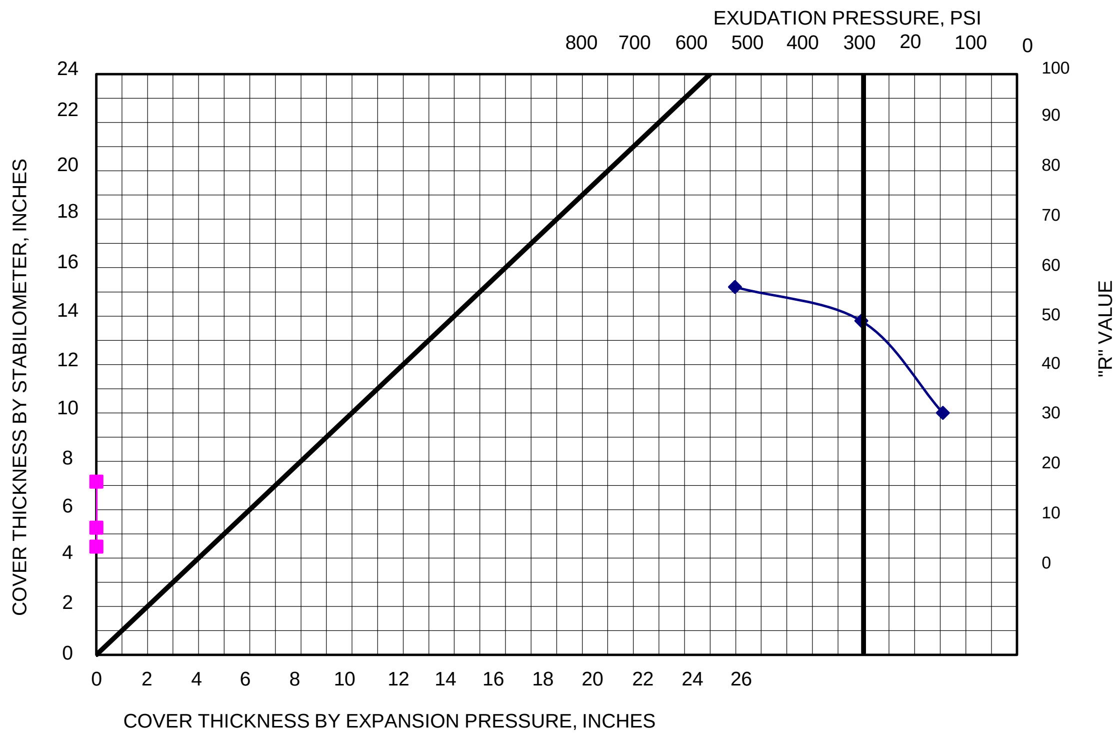
BSK ASSOCIATES

Standard Test Methods for Resistance R-Value and Expansion Pressure of Compacted Soil

ASTM D-2844

FIGURE B-4
700 22nd St.
Bakersfield, CA 93301
Ph: (661) 327-0670

Project Name:	Madison ES 2-Storey Building	**Sample Date:**	7/26/23
Project Number:	G00001343	**Test Date:**	8/21/23
Lab Tracking ID:	B23-096	**Report Date:**	8/23/23
Sample Location:	B-2 @ 0.0-5.0 feet bgs	**Tested By:**	ILT Remotigue



Sample Description: ML: SANDY SILT; Reddish Brown; Fine to Medium. Moist

SPECIMEN	A	B	C
EXUDATION PRESSURE, LOAD (lb)	6923.6	3823.6	1823.6
EXUDATION PRESSURE, PSI	551	304	145
EXPANSION, * 0.0001 IN	0.0023	0.0019	0.0017
EXPANSION PRESSURE, PSF	0	0	0
STABILOMETER PH AT 2000 LBS	54	68	98
DISPLACEMENT	3.81	3.56	3.67
RESISTANCE VALUE "R"	56	49	30
"R" VALUE CORRECTED FOR HEIGHT	56	49	30
% MOISTURE AT TEST	8.3	9.3	10.3
DRY DENSITY AT TEST, PCF	109.7	106.5	106.3
"R" VALUE AT 300 PSI EXUDATION PRESSURE	**49**		
"R" VALUE BY EXPANSION PRESSURE TI = 4.0, GF=1.50	**N/A**		

Reviewed By: *Ian Leo T. Remotigue*

APPENDIX C

APPENDIX C

GEOLOGIC AND SEISMIC HAZARDS EVALUATION REPORT

MADISON ELEMENTARY SCHOOL – PROPOSED 2-STORY STRUCTURE

109 STADIUM ROAD

MADERA, CALIFORNIA

Table of Contents

TABLES

FIGURES

LIQUEFACTION ANALYSIS

C1. INTRODUCTION

This report presents the geologic and seismic hazards assessment prepared in accordance with the 2022 California Building Code (CBC), CCR Title 24, Chapters 16A and 18A requirements for a Geotechnical/Engineering Geologic Report. This report focuses solely on the planned new structures. This report is not intended to assess the geologic and seismic hazards for the rest of the school campus. The assessment was performed in conformance with California Geological Survey (CGS) Note 48 (2022).

C1.1 Purpose and Scope of Services

The purpose of the geologic and seismic hazards assessment is to provide the Client with an evaluation of potential geologic or seismic hazards which may be present at the site or due to regional influences. BSK's scope of services for this assessment included the following:

1. Review of published geologic literature, and current and past investigations at the Site;
2. Evaluation of the data collected and preparation of geologic cross sections;
3. Evaluation of potential geologic hazards affecting the site; and
4. Determination of Site Class and seismic design parameters.

The observations and conclusions presented in this report specifically exclude the assessment of environmental characteristics, particularly those involving hazardous substances, and a high-pressure pipeline risk evaluation.

C1.2 Site Location

Madison Elementary School is located at 109 Stadium Road in Madera, Madera County, California (Site). The approximate coordinates near the center of the proposed new 2-story building, modular buildings and parking lot are:

Latitude: 36.951554ºN
Longitude: -120.063324ºW

The Site is primarily surrounded by residential properties with Madera High School to the north and west.

C1.3 Site Topography

As shown on Figure C-1, the Site and surrounding area topography is relatively flat with a ground surface elevation of approximately 265 feet, USGS datum.

C1.4 Groundwater Conditions

The Site is within the Madera sub-basin of the San Joaquin Basin Hydrologic Study Area. This includes approximately the southern two-thirds of the Great Valley. Within the Study Area, 39 groundwater basins and areas of potential storage have been identified. The boundaries of these areas are based largely on hydrologic as well as political considerations.

At the time of the field exploration in July 2023, groundwater was not encountered in our borings completed to a maximum depth of approximately 51.5-feet below the ground surface (BGS). According to California Department of Water Resources' Sustainable Groundwater Management Act (SGMA) database, monitoring well number 11S18E30D001M located approximately 0.43 miles east of the Site, measured groundwater at a depth of approximately 59.10-feet BGS in 1960. The water level hydrograph from well 11S18E30D001M is presented on Figure C-2.

Please note that the groundwater level may fluctuate both seasonally and from year to year due to variations in rainfall, temperature, pumping from wells and possibly as the result of other factors that were not evident at the time of our investigation.

C2.0 GEOLOGIC SETTING

The Site is located in the Great Valley geomorphic province near the zone of transition from the alluvial valley to the foothills of the Sierra Nevada Mountain Range. The area lies within the structural region identified by Bartow (1991) as the San Joaquin Valley portion of the southern Sierran block. This region forms a broad syncline with deposits of marine and overlying continental sediments, Jurassic to Holocene in age. The thickness of the sediments increases to the west and reaches a thickness of as much as 20,000 feet on the west side of the San Joaquin Valley syncline. Approximately 25 miles northeast of the Site, the slightly inclined alluvial fan geomorphology transitions into the foothills of Sierra Nevada, generally consisting of pre-Cretaceous metamorphic rocks and Mesozoic granitic rocks.

As shown on Figure C-3, the site is situated on recent alluvial fan deposits (Jennings and Strand, 1958). These sediments are derived from the Sierra Nevada Mountain Range to the east and deposited from streams emerging from highlands surrounding the Great Valley.

Nearby active faults include the Great Valley Fault and San Andreas Fault located approximately 40 miles and 67 miles southwest of the site, respectively.

C2.1 Subsurface Conditions

Subsurface conditions are described in the main body of the report prepared by BSK Associates (BSK) and to which this geologic and seismic hazards report is appended. The Site was the subject of a current field investigation of three (3) soil borings completed to depths ranging from 21.5 feet to 51.5 feet BGS (see

Figure C-4, Site Map). We encountered approximately 8 feet of sandy silt in our borings. This layer is underlain by clayey sand to a depth of about 14-feet BGS. This layer rests on clayey sand and sandy clay layers to an approximate depth of 45-feet BGS. Below 45-feet BGS, we encountered poorly graded sand to the bottom depth of 51.5 feet BGS.

As shown on Figure C-5, a simplified geologic cross-section was constructed to interpret subsurface conditions based the current soil borings.

C3.0 GEOLOGIC/SEISMIC HAZARDS

The types of geologic and seismic hazards assessed include surface ground fault rupture, liquefaction, seismically induced settlement, slope failure, flood hazards and inundation hazards.

C3.1 Fault Rupture Hazard Zones in California

The purpose of the Alquist-Priolo Geologic Hazards Zones Act, as summarized in CDMG Special Publication 42 (SP 42), is to "prohibit the location of most structures for human occupancy across the traces of active faults and to mitigate thereby the hazard of fault-rupture." As indicated by SP 42, "the State Geologist is required to delineate "earthquake fault zones" (EFZs) along known active faults in California. Cities and counties affected by the zones must regulate certain development 'projects' within the zones. They must withhold development permits for sites within the zones until geologic investigations demonstrate that a site is not threatened by surface displacement from future faulting.

The Site is not located in an Alquist-Priolo Earthquake Fault Zone. The closest Fault-Rupture Hazard Zone is associated with the Ortigalita Fault and Nunez Fault, located approximately 48 miles west and 55 southwest of the Site, respectively.

C3.2 State of California Seismic Hazard Zones (Liquefaction and Landslides)

Zones of Required Investigation referred to as "Seismic Hazard Zones" (SHZ) in CCR Article 10, Section 3722, are areas shown on Seismic Hazard Zone Maps where site investigations are required to determine the need for mitigation of potential liquefaction and/or earthquake-induced landslide ground displacements.

The Site is within the Madera 7.5 Minute Quadrangle and there are no mapped areas that have Seismic Hazard Zones in the project area.

C3.3 Local General Plans Safety Element

The 2015 amended Madera County General Plan did not identify geologic hazard zones in Madera County.

C3.4 Slope Stability and Potential for Slope Failure

The project area is essentially flat and the potential hazard due to landslides from adjacent properties is low.

C3.5 Flood and Inundation Hazards

An evaluation of flooding at the site includes review of potential hazards from flooding during periods of heavy precipitation and flooding due to a catastrophic dam breach from up-gradient surface impoundments.

C3.5.1 Flood Hazards

Federal Emergency Management Agency (FEMA) flood hazard data was obtained to present information regarding the potential for flooding at the Site. As shown on Figure C-6 according to FEMA D-Firm GIS data NFHL 06039C, effective date 06/15/2017, the Site lies in Zone X, area of minimal flooding outside the 500-year and 100-year floodplains.

C3.5.2 Inundation Hazards - Dams

As shown on Figure C-6, according to the Department of Water Resources, Division of Safety of Dams (DSOD), the Site is located in the pathway of the Hidden Dam inundation area.

C3.6 Volcanic Hazards

According to USGS Bulletin 1847, dated 1989, the site is not located in an area which would be subject to hazards from volcanic eruptions (Miller, 1989).

C3.7 Corrosion

Please refer to the section titled "Soil Corrosivity" in the geotechnical report for discussion of the corrosivity of the site soils.

C3.8 Expansive Soils

As discussed in the geotechnical report, the near-surface soil encountered within the current borings at the Site consists of sandy silt which exhibits a very low expansion potential.

C3.9 Land Subsidence

Four types of subsidence are known to occur in the San Joaquin Valley (Galloway, 1999). In order of decreasing magnitude they are:

1. Subsidence caused by aquifer system compaction due to the lowering of ground-water levels by sustained groundwater overdraft;
2. Subsidence caused by the hydrocompaction of moisture-deficient deposits above the water table;
3. Subsidence related to fluid withdrawal from oil and gas fields; and

4. Subsidence related to crustal neotectonic movements.

The Site is located in an area known to be susceptible to subsidence due to groundwater withdrawal. According to surface elevation data obtained from the Sustainable Groundwater Management Act (SGMA) Data Viewer, a GPS station located approximately 2,300 feet southeast of the Site has experienced about 2.2 inches of vertical displacement (settlement) since 2005.

The Site is not located in an area known to be susceptible to subsidence due to petroleum extraction. The Site is not located in an area in which soils are known to be impacted by hydrocompaction.

C3.10 Tsunami Hazard

According to the Tsunami Inundation Map for Emergency Planning (Cal-EMA, 2009) and the ASCE Tsunami Hazard Tool (ASCE 2016) the Site is not located in a Tsunami Hazard zone.

C4. SEISMIC HAZARD ASSESSMENT

C4.1 Seismic Source Deaggregation

Figure C-7 presents a fault map showing the major faults that may impact the site in the future. Seismically induced ground motion at a site can be caused by earthquakes on any of the sources surrounding the Site. Deaggregation of the seismic hazard was performed by using the USGS Interactive Deaggregation website. The deaggregation determination, at the maximum considered earthquake (MCE) hazard level, results in distance, magnitude and epsilon (ground-motion uncertainty) for each source that contributes to the hazard. Each source has a corresponding epsilon, which is the probabilistic value relative to the mean value of ground motion for that source.

Deaggregation based on a probabilistic model developed by the USGS indicates that the extreme seismic source with the highest magnitude that contributes to the peak ground acceleration (PGA) is a magnitude 8.14 earthquake from the San Andreas Fault. For liquefaction and seismic settlement, the modal magnitude (Mw) of 5.5 with a distance of 11.19 km would be appropriate for probabilistic input parameter that is consistent with the design earthquake ground motion.

C4.2 Historical Seismicity

Table C-1 provides the location, earthquake magnitude, site to earthquake distances, dates and the resulting site peak horizontal acceleration for the period 1800 to 2021. Figure C-8 presents historical earthquake magnitudes and locations relative to the Site.

TABLE C-1 HISTORIC EARTHQUAKES WITHIN 100 MILES OF THE SITE GROUND MOTION GREATER THAN 0.05G							
File Code	**Latitude (North)**	**Longitude (West)**	**Date**	**Depth (km)**	**Earthquake Magnitude**	**Site Acceleration (g)**	**Distance mi (km)**
MGI	37.000	120.070	9/12/1928	0	4.6	0.23	3.4(5.4)
BRK	36.220	120.290	5/2/1983	0	6.7	0.11	52.1(83.8)
T-A	36.830	121.570	10/18/1800	0	7.0	0.09	83.6(134.6)
BRK	36.220	120.400	7/22/1983	0	6.0	0.07	53.9(86.7)
DMG	36.400	121.000	04/12/1885	0	6.2	0.07	64.4(103.6)
PAS	37.556	118.791	5/25/1980	6.4	6.5	0.07	81.4(131.0)
DMG	35.750	120.250	3/10/1922	0	6.5	0.07	83.6(134.5)
PAS	36.151	120.049	8/4/1985	6	5.8	0.06	55.3(89.0)
PAS	37.464	118.823	5/27/1980	2.4	6.3	0.06	76.8(123.6)
PAS	37.608	118.821	5/25/1980	3.7	6.4	0.06	81.9(131.8)
DMG	37.500	118.500	04/11/1872	0	6.6	0.06	93.9(151.1)
DMG	37.250	121.750	7/1/1911	0	6.6	0.06	95.1(153.1)
PAS	36.286	120.413	10/25/1982	6	5.6	0.06	49.9(80.3)
BRK	36.220	120.290	5/2/1983	0	5.6	0.06	52.1(83.8)
DMG	37.000	121.500	06/20/1897	0	6.2	0.06	79.3(127.6)
T-A	36.750	119.750	08/16/1864	0	4.3	0.06	22.2(35.7)
DMG	36.900	121.200	03/06/1882	0	5.7	0.06	62.8(101.1)
PAS	37.470	118.597	11/23/1984	6	6.2	0.06	88.2(141.9)

Table C-1 shows that the site has experienced mean plus one sigma peak horizontal acceleration up to 0.23g from an 4.6 magnitude earthquake in 1928. In general the Site has been subjected to relatively low to moderate intensity ground motion, primarily from large earthquakes on distance faults and closer low magnitude earthquakes.

C4.3 Earthquake Ground Motion, 2022 California Building Code

C4.3.1 Site Class

Based on Section 1613A.3.2 of the 2022 California Building Code (CBC), the Site shall be classified as Site Class A, B, C, D, E or F based on the Site soil properties and in accordance with Chapter 20 of ASCE 7-16. Based on the N values from our 2023 soil boring, as per Table 20.3-1 of ASCE 7-10, the Site is Class D (15 ≤ N ≤50).

C4.3.2 Seismic Design Criteria

The 2022 California Building Code (CBC) utilizes ground motion based on the Risk-Targeted Maximum Considered Earthquake (MCE_R) that is defined in the 2022 CBC as the most severe earthquake effects considered by this code, determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk. Ground motion parameters in the 2022 CBC are based on ASCE 7-16, Chapter 11.

The United States Geologic Survey (USGS) has prepared maps presenting the Risk-Targeted MCE spectral acceleration (5% damping) for periods of 0.2 seconds (S_S) and 1.0 seconds (S_1). The values of S_s and S_1 can be obtained from the OSHPD Seismic Design Maps Application available at:
https://seismicmaps.org/
Table 2 below presents the spectral acceleration parameters produced for Site Class D by OSHPD Ground Motion Parameter Application and Chapter 16 of the 2022 CBC based on ASCE 7-16.

TABLE C-2
SPECTRAL ACCELERATION PARAMETERS
RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE

Criteria	Value		Reference
MCE Mapped Spectral Acceleration (g)	S_S = 0.601	S_1 = 0.235	USGS Mapped Value
Site Coefficients (Site Class D)	F_a = 1.319	F_v = null (2.130)	ASCE Table 11.4
Site Adjusted MCE Spectral Acceleration (g)[1]	S_{MS} = 0.793	S_{M1} = null[1](0.501)[2]	ASCE Equations 11.4.1-2
Site Adjusted MCE Spectral Acceleration (g) [2]		*S_{M1} = 0.752*	*ASCE Equations 11.4.1-2*
Design Spectral Acceleration (g)	S_{DS} = 0.528	S_{D1} = null[1](0.334)[2]	ASCE Equations 11.4.3-4
Design Spectral Acceleration (g) [2]		*S_{D1} = 0.501*	*ASCE Equations 11.4.3-4*
Site Short Period - T_S (Seconds)	Ts =0.633		Ts = S_{D1}/ S_{DS}
Site Short Period - T_S (Seconds) [2]	*Ts = 0.949*		*Ts = S_{D1}/ S_{DS}*
Site Long-Period - T_L (Seconds)	T_L = 12		USGS Mapped Value

[1] Requires site-specific ground motion procedure or exception as per ASCE 7-16 Section 11.4.8. No increase per ASCE 7-16 Supplement 3 applied.
[2] Values include 50% increase per ASCE 7-16 Supplement 3 for Site Class D with S_1 greater than or equal to 0.2

C4.3.4 Geometric Mean Peak Ground Acceleration

As per Section 1803A.5.12 of the CBC, peak ground acceleration (PGA) utilized for dynamic lateral earth pressures and liquefaction, shall be based on a site specific study (ASCE 7-16, Section 21.5) or ASCE 7-16, Section 11.8.3. The USGS Ground Motion Parameter Application based on ASCE 7-16, Section 11.8.3 produced the values shown in Table 3 based on Site Class D.

TABLE C-3
GEOMETRIC MEAN PEAK GROUND ACCELERATION
MAXIMUM CONSIDERED EARTHQUAKE

Criteria	Value	Reference
Mapped Peak Ground Acceleration (g)	PGA = 0.259	USGS Mapped Value
Site Coefficients (Site Class D)	F_{PGA} = 1.341	ASCE Table 11.8-1
Geometric Mean PGA (g)	PGA_M = 0.348	ASCE Equations 11.8-1

C4.4 Seismically Induced Ground Failure

C4.4.1 Liquefaction and Lateral Spreading

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, and fine-grained sand deposits and some lean clays. If liquefaction occurs, foundations resting on or within the liquefiable layer may undergo settlements and/or a loss of bearing capacity.

Due to the absence of shallow groundwater, we conclude that the potential for liquefaction to occur at the Site is low.

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to temporary lateral migration of subsurface liquefied soils during a design seismic event. These phenomena typically occur adjacent to free faces such as slopes and creek channels. The potential for lateral spreading to impact the Site is also low due to the low liquefaction potential and flat topography.

C4.4.2 Dynamic Compaction/Seismic Settlement

Another type of seismically-induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. Loose to medium dense sand layers were encountered in our borings. Using the methodology by Tokimatsu and Seed (1987), we estimate that the seismic settlement of dry sand during a design-level earthquake will be negligible. Our seismic settlement analysis is appended to this report.

C5. REFERENCES

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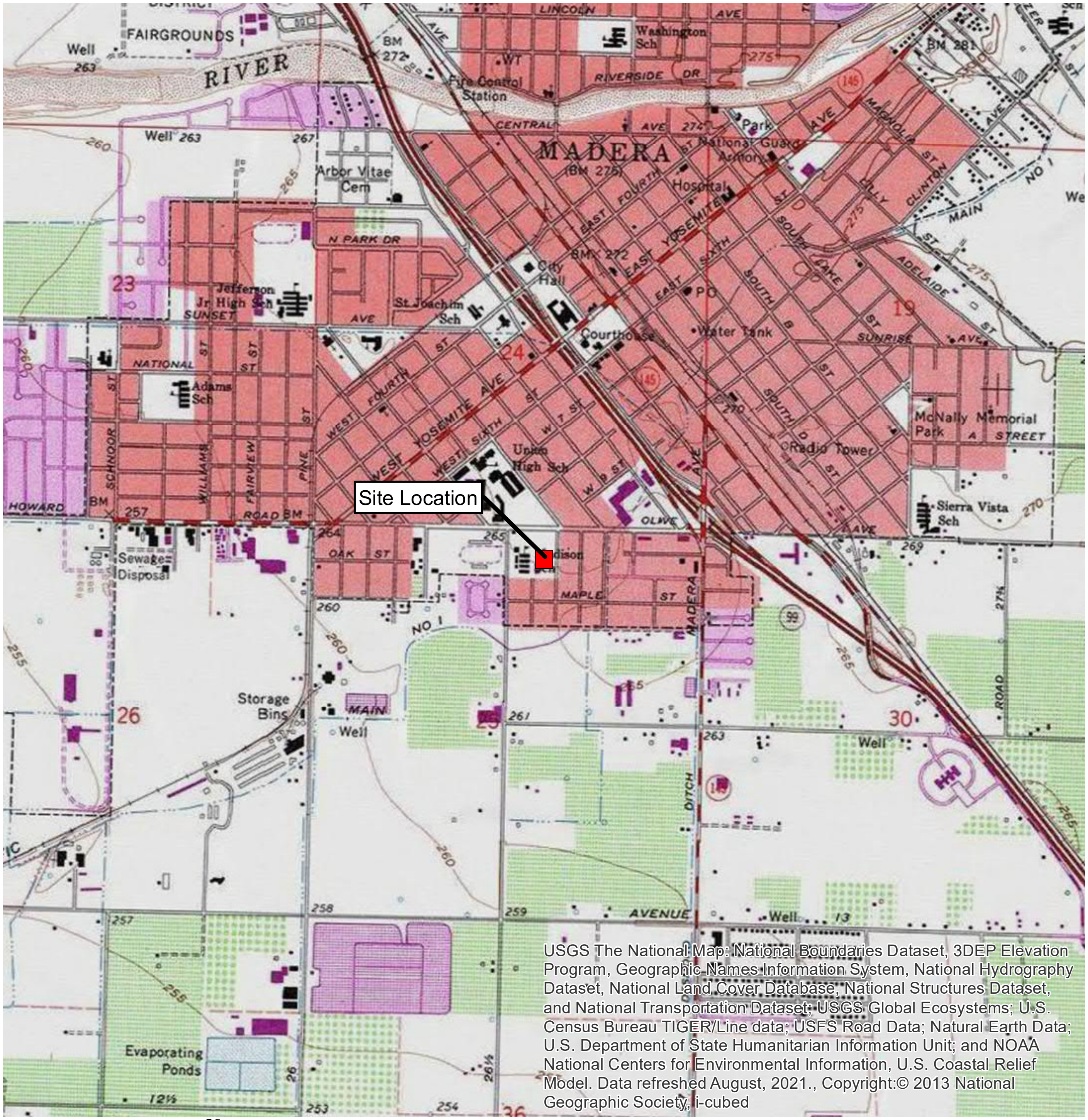

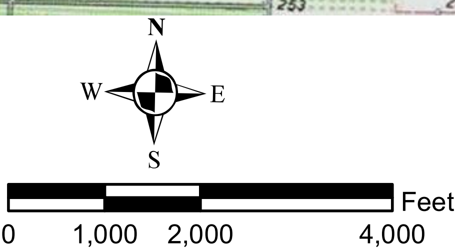

Geologic/Seismic Hazards Evaluation Proposed 2-Story Structure Madison Elementary School Madera, California	Figure C-1 Topographic Map BSK Project G0000-1343

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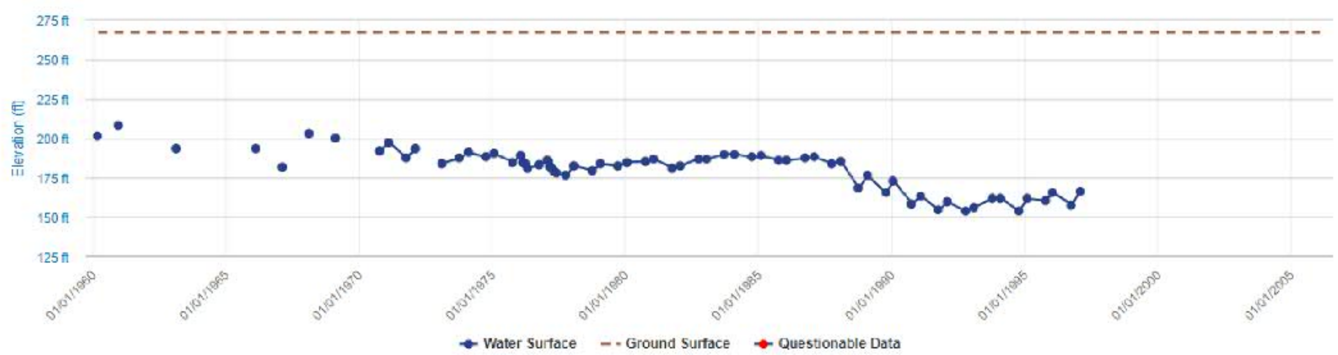
Groundwater Levels for Well 369524N1200557W001



Measurement Date (PST)	Reference Point Elevation	Ground Surface Elevation	Distance from RP to WS	Groundwater Elevation	Ground Surface to Water Surface	Measurement Issue	Submitting Organization	Collecting Organization	Water Level Measurement Comments
12/08/1960 00:00:00	269.320	267.320	61.1	208.22	59.1		Department of Water Resou...	Madera Irrigation District	
02/08/1968 00:00:00	269.320	267.320	66.2	203.12	64.2		Department of Water Resou...	Madera Irrigation District	
03/04/1960 00:00:00	269.320	267.320	68.1	201.22	66.1		Department of Water Resou...	Madera Irrigation District	
02/10/1969 00:00:00	269.320	267.320	69.4	199.92	67.4		Department of Water Resou...	Madera Irrigation District	
02/08/1971 00:00:00	269.320	267.320	72	197.32	70		Department of Water Resou...	Madera Irrigation District	
02/19/1963 00:00:00	269.320	267.320	75.6	193.72	73.6		Department of Water Resou...	Madera Irrigation District	
02/10/1966 00:00:00	269.320	267.320	75.8	193.52	73.8		Department of Water Resou...	Madera Irrigation District	

State Well Number: 11S18E30D001M
Latitude (NAD83): 36.9524
Longitude (NAD83): -120.0557
Groundwater Basin (code): Madera (5-022.06)
Reference Point Elevation (NAVD88 ft): 268.320
Ground Surface Elevation (NAVD88 ft): 267.320

Reference: https://wdl.water.ca.gov/waterdatalibrary/

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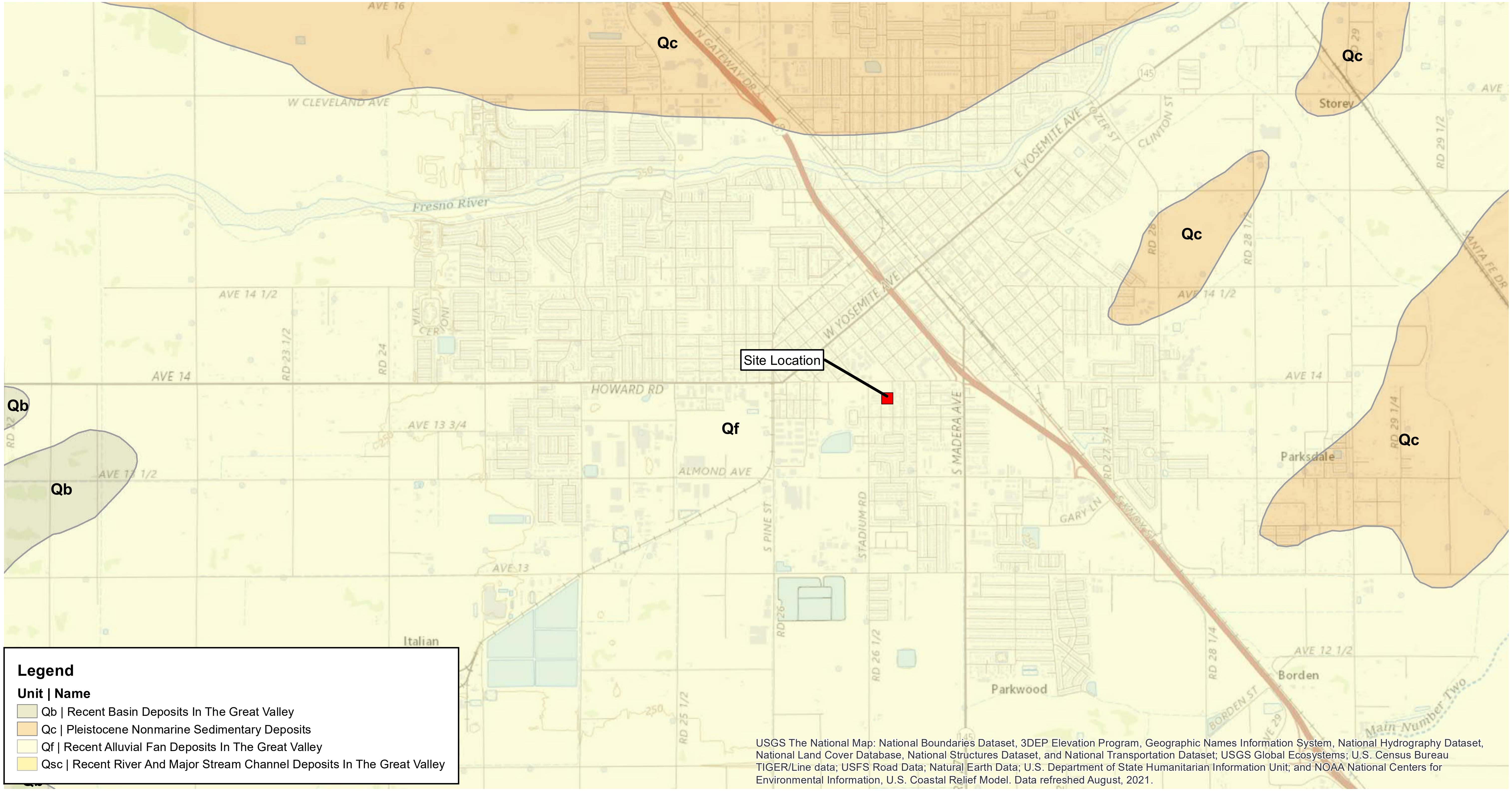
Geologic/Seismic Hazards Evaluation
Proposed 2-Story Structure
Madison Elementary School
Madera, California

Figure C-2
Area Hydrograph
BSK Project G0000-1343



References: Jennings, C.W. and Strand, R.G., (1958)
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Geologic Atlas of California GAM-20

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Geologic/Seismic Hazards Evaluation Proposed 2-Story Structure Madison Elementary School Madera, California	Figure C-3 Geologic Map BSK Project G0000-1343

Map Date: 8/10/2023

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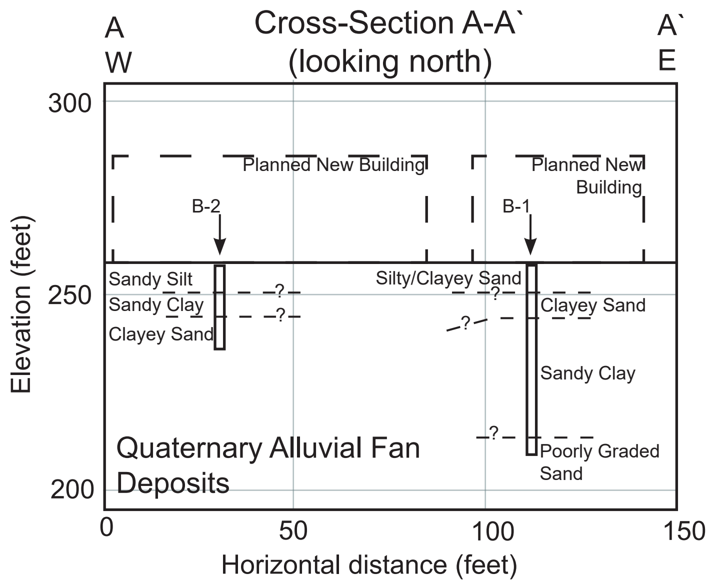
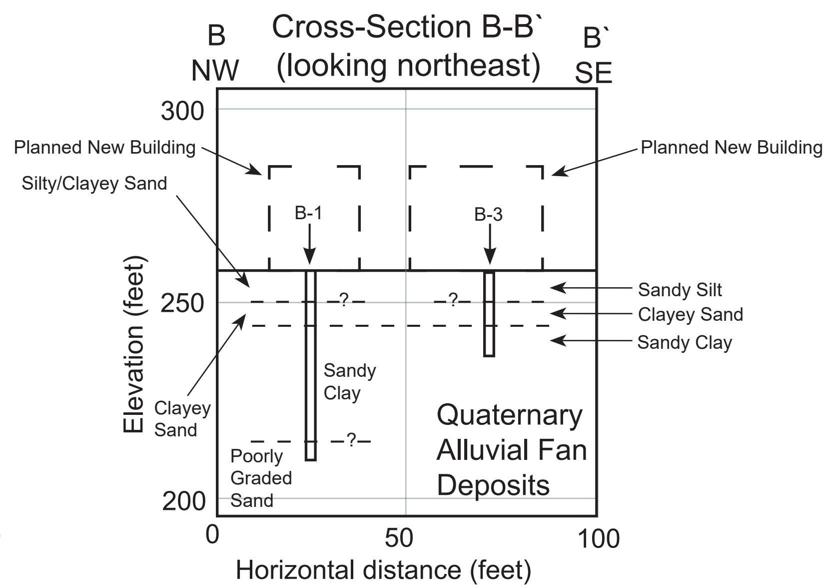
Geologic/Seismic Hazards Evaluation
Proposed 2-Story Structure
Madison Elementary School
Madera, California

Figure C-4
Site Map
BSK Project G0000-1343





LEGEND

Soil boring location

- - ?- Unit contact

Note: Locations are approximate

Scale 1 inch : 50 feet

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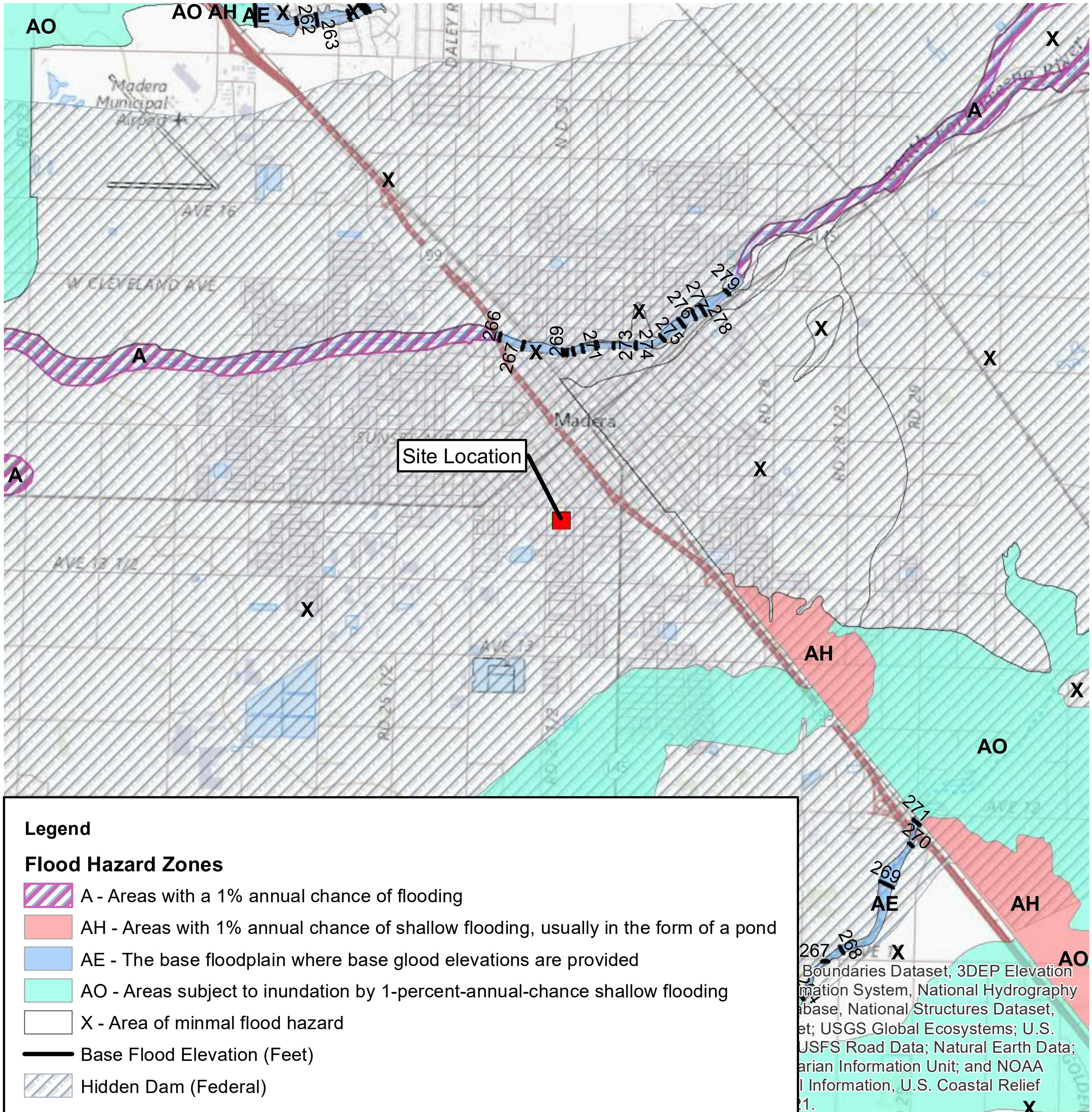
Geologic/Seismic Hazard Evaluation
Proposed 2-Story Structure
Madison Elementary School
Madera, California

Figure C-5
Geologic Cross Section A-A` and Section B-B`
BSK Project G0000-1343



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References: Federal Emergency Management Agency (FEMA), Flood Hazard Layer NFHL_06039C, Effect. Date 06/15/2017
https://fmds.water.ca.gov/webgis/?appid=dam_prototype_v2
State of California OES Dam Inundation DVD

Geologic/Seismic Hazards Evaluation Proposed 2-Story Structure Madison Elementary School Madera, California	Figure C-6 Flood Hazard Map BSK Project G0000-1343

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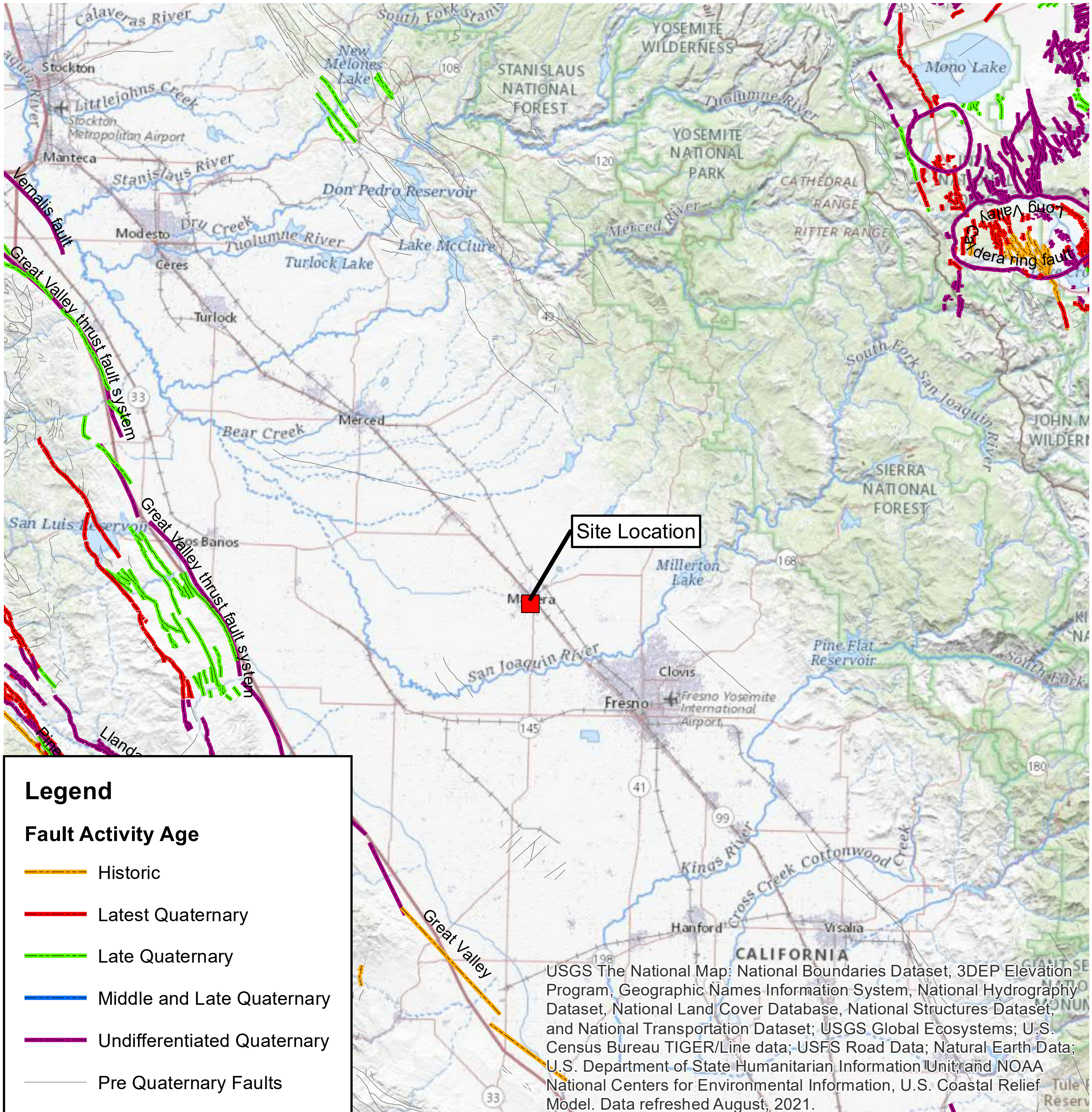
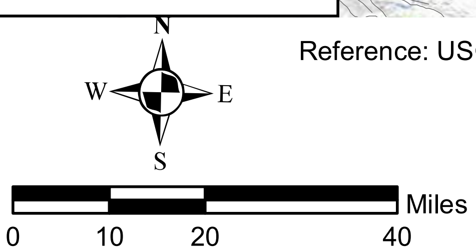


Reference: USGS Quaternary Fault Database ftp://hazards.cr.usgs.gov/maps/qfault/



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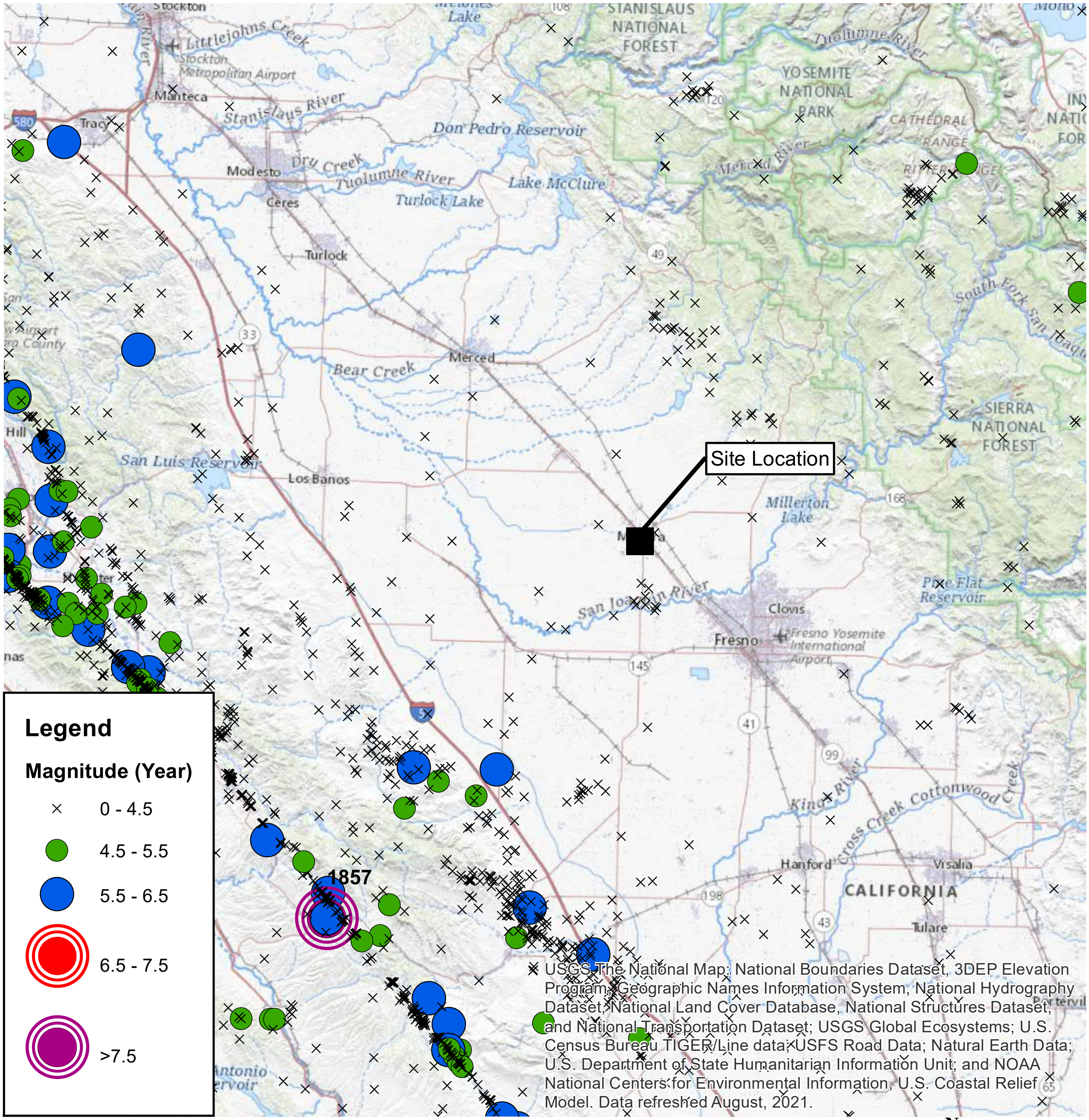
Geologic/Seismic Hazards Evaluation Proposed 2-Story Structure Madison Elementary School Madera, California	Figure C-7 Area Fault Map BSK Project G0000-1343



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0 10 20 40 Miles

Geologic/Seismic Hazards Evaluation Proposed 2-Story Structure Madison Elementary School Madera, California	Figure C-8 Historical Earthquakes BSK Project G0000-1343

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Seismic Settlement of Dry Sands
Tokimatsu & Seed (1987)

Project No. G00001343
Project Name Madison ES - Proposed 2-Story Structure
Analysis by D. Tower

M = 5.5 Moment Magnitude (Use Modal value)
PHA = 0.348 g (Peak horizontal acceleration)
γ = 120 pcf (unit weight of soil)
Ko = 0.5 (at-rest coefficient)
Hammer Energy (%) = 70.6

Results

Boring	Depth at top of sampler (ft)	Layer Thickness (ft)	Soil Classification	Anticipated Fines Content (%)	r_d	σ_0 (psf)	σ'_m (psf)	σ'_m (tsf)	N (blows/ft)	SAMPLER TYPE (1) SPT w/out liners (2) SPT w/ liners (3) MC (4) CAL	Hammer Correction C_E	Sampler Correction, C_S	Overbuden Correction, C_N	Fine Content Correction	N_1 (blows/ft)	Gmax (psf)	Effective Shear Strain, γ_{eff} (Geff/Gmax)	Effective Shear Strain, γ_{eff} (from Fig. 11)	Effective Shear Strain, γ_{eff} (%)	Volumetric Strain (from Figure 13) (%)	Seismic Settlement for M7.5 (in)	Seismic Settlement for M5.25 (in)	Seismic Settlement for M6 (in)	Seismic Settlement for M6.75 (in)	Seismic Settlement for M8.5 (in)
B-1	5	7	SM	37	0.989	600	400	0.2	8	4	1.18	0.65	1.70	3	13	949178	1.41E-04	2.6E-04	2.6E-02	5.00E-02	0.08	0.03	0.05	0.07	0.11
	10	7	SC	15	0.978	1200	800	0.4	11	4	1.18	0.65	1.29	1	12	1302063	2.04E-04	4.4E-04	4.4E-02	9.00E-02	0.15	0.06	0.09	0.13	0.19
																					0.00	0.00	0.00	0.00	0.00
																					0.00	0.00	0.00	0.00	0.00
																					0.00	0.00	0.00	0.00	0.00
																					0.24	0.09	0.14	0.20	0.29

Select= < 1/4 for M 5.5