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February 2, 2026 BSK Project No. G25001215

Ms. Tracie Rose
Quattrocchi Kwok Architects
636 Fifth Street
Santa Rose, California 95404

SUBJECT: **Supplemental Geotechnical Recommendations Letter**
Liberty Adult Community Education School
Gateway Portable Building and Shade Structures
Brentwood, California

Dear Ms. Rose:

We are pleased to submit our supplemental geotechnical recommendations letter for the planned portable building and shade structures at the Liberty Adult Community Education School at the Gateway location at 929 2nd Street in Brentwood, California. BSK presented geotechnical recommendations for the planned improvements at the campus in our report entitled *Geotechnical Investigation and Geologic and Seismic Hazards Assessment Report, Liberty Adult Education Center, Gateway and Tobin Sites, Brentwood, California,* dated August 4, 2023 (BSK Project No. G00000614). Since our recent report was published, a portable building and three shade structures were added to the Gateway location as shown on the attached Figure 1, Supplemental Site Plan.

PROJECT UNDERSTANDING

The Liberty Adult Education Center plans to construct one (1) portable classroom building and three new shade structures at the Gateway location as shown on Figure 1. We have assumed the portable building will be supported on shallow concrete foundations and the shade structures will be supported on drilled pier foundations. We also anticipate that grading will be minimal, less than 2 feet or so of cut or fill for the building, with up to 5 feet of cut for underground utilities.

If the actual project description differs significantly from that anticipated above, we should be notified so that we may review our proposed scope of services presented herein for applicability.

SUBSURFACE CONDITIONS

According to our previous exploration points located near the portable building and shade structures (boring B-1 and CPT-2), near surface soils consisted primarily of firm to hard sandy lean clay to an approximate depth of 32 feet below existing ground surface (bgs). Below 32 feet bgs, CPT-2 interpreted interbedded sand and clay layers to the maximum depth explored of 50 feet bgs. A more detailed description of the soils encountered can be found in the boring and CPT logs in the attached geotechnical report.

As shown on Figure 2, a supplemental geologic cross section was drawn to interpret subsurface conditions based on the boring and CPT. Discussion of the subsurface conditions encountered at the project site (Site) is presented in the "Subsurface Conditions" section of the geotechnical report.

CONCLUSIONS AND SUPPLEMENTAL RECOMMENDATIONS

The findings and conclusions regarding geologic and seismic hazards and the shallow foundation recommendations presented in our August 4, 2023 report apply to the foundations for the new portable building at the Gateway location. We have provided recommendations for drilled pier foundations for the shade structures in the following sections.

Drilled Pier Foundations

It is our understanding that the canopies for this project will be designed using the Superior Shade Pre-Check Design[1], 14 x 14 x 10 (designated as DSACU141410IN) cantilever umbrellas along with a 20 x 40 x 10 (designated as DSACU202012IN) Hip. The pier embedment depths will be per the highlighted column shown in Table 2 below.

TABLE 2 : Shade Foundation

✔	Shade Number	Spread Footing Depth	Spread Footing Size	Spread Footing Reinforcement	Pier Footing Depth	Pier Footing Diameter	Pier Footing Reinforcement
	DSACU101009IN	3.0'	5.5' x 5.5'	7 #5	7'	Ø3'	12 #6
	DSACU101010IN	3.0'	5.5' x 5.5'	7 #5	7'	Ø3'	12 #6
	DSACU101012IN	3.0'	6' x 6'	8 #5	7.5'	Ø3'	12 #6
	DSACU141409IN	3.0'	6.25' x 6.25'	8 #5	8'	Ø3'	12 #6
	DSACU141410IN	3.0'	6.25' x 6.25'	8 #5	8'	Ø3'	12 #6
	DSACU141412IN	3.0'	6.5' x 6.5'	9 #5	8'	Ø3'	12 #6
	DSACU202009IN	3.0'	7.25' x 7.25'	10 #5	9'	Ø3'	12 #6
	DSACU202010IN	3.0'	7.25' x 7.25'	10 #5	9'	Ø3'	12 #6
	DSACU202012IN	3.0'	7.75' x 7.75'	10 #5	9.5'	Ø3'	12 #6

Per the pre-check plan, the minimum recommended drilled pier depth is 8' bgs and 9.5' bgs for the DSACU141410IN and DSACU202012IN, respectively, if the geotechnical parameters are equal to or greater than those shown in Note 1.A below.

[1] DSA Pre-Check Design by Superior Shade on their plans entitled *Fabric Canopies DSA PC-CU, Sheet S4*, dated 8/4/2023, DSA App: 02-120866.

1.A. SOIL (NO SOIL REPORT PROVIDED): SOIL BEARING PRESSURE = 1500 PSF AT 24" BELOW THE LOWEST GRADE. LATERAL BEARING PRESSURE = 200 PSF/FT (CLASS 5), INCREASED PER CBC SECTION 1806A.3.4. A SITE-SPECIFIC GEOTECHNICAL REPORT IS REQUIRED AT THE TIME OF SITE APPLICATION WHEN USING LOAD-BEARING VALUES ABOVE THE STATED MAXIMUMS FOR CLASS 5 SOIL. ALLOWABLE PIER FRICTIONAL UPLIFT CAPACITY = 250 PSF. 1/3 INCREASE FOR SHORT TERM LOADS IS NOT ALLOWED.

Drilled piers may be designed for an allowable end bearing pressure of 1,500 psf (includes a factor of safety of 3) and an allowable average skin friction of 375 psf (includes a factor of safety of 2) to support vertical downward loads applied to the pier foundations. Allowable uplift capacity may be obtained by multiplying the allowable frictional compressive capacity by 2/3 and by adding the weight of the pier foundation. Drilled piers should be spaced a minimum of 3 pier diameters apart, otherwise reductions in axial capacities should be applied. For downward loads, **the allowable end bearing and skin friction can be combined, provided the recommendations below regarding pier cleanout are implemented. However, immediate settlement on the order of 5% of the diameter of the pier (about 2 inches for a 36-inch diameter pier) should be expected if end bearing is mobilized**.

It is recommended that an allowable lateral soil bearing pressure of 350 psf per foot of embedment be used for lateral capacity. The allowable lateral soil bearing pressure may increase with depth up to a maximum of 1,500 psf. This value includes a factor of safety of 1.5. Piers should be spaced a minimum of 6 pier diameters, center to center, in the direction of loading, otherwise reduction factors may need to be applied to the allowable lateral soil bearing pressure. Unless the drilled pier is surrounded by concrete paving, the upper one foot of the soil should be ignored when calculating skin friction and lateral capacity.

Additional Drilled Pier Considerations

Provided that the drilled piers are designed according to the recommendations presented above and constructed properly, the total and differential elastic settlement is anticipated to be about 2-inches and ½ inch, respectively if end bearing is used for axial capacity, and ½ inch and less than ½ inch, respectively, if only skin friction is used for axial capacity. As discussed in the geotechnical report, about ½-inch of liquefaction induced settlement is estimated to occur at the depth of the foundations, with about half of that as differential settlement. So total elastic and liquefaction-induced total and differential settlements are estimated to be about 2½-inch and 1-inch, respectively if end bearing is mobilized for axial capacity, and 1-inch and ¾-inch total and differential, respectively, if only skin friction is used for axial capacity. Differential settlement is defined in this report as the vertical difference in settlement between adjacent foundation supports or across a horizontal distance of 30 feet, whichever is less. A majority of the estimated elastic settlement is expected to occur during construction as the foundation is loaded.

It is our understanding that the drilled piers for the shade structures will be located in an asphalt or concrete paved courtyard. Therefore, we do not anticipate that inundation due to the Marsh Creek main dam will cause local erosion or scour of the soil surrounding the piers.

If a buried utility is located within 4D laterally from the center of the pier, where D is the pier diameter lateral movement of the pier in the direction of the utility may apply a horizontal pressure on the utility. A possible mitigation measure could consist of excavating the soil around and above the utility extending to a depth of 12 inches below the utility and 12 inches on either side of the widest part utility, for a lateral distance of at least 10 feet along the utility's alignment and centered on the nearby drilled pier. Then the excavation around and above the utility should be entirely encased with controlled low strength material (CLSM) to within a minimum of 12 inches above the utility over the 10-foot length. The remainder of the utility excavation should be backfilled with properly compacted engineered fill.

Drilled Pier Construction Considerations

If end bearing will be mobilized for the axial support of the pier, a cleanout bucket should be used to remove excess soil from the bottom of the pier excavation. We recommend that drilled pier steel reinforcement and concrete be placed within about 4 to 6 hours upon completion of each drilled hole. As a minimum, the holes should be poured the same day they are drilled. If the holes cannot be backfilled the same day they are drilled, the hole needs to be checked for caving, sloughing or squeezing prior to setting the rebar cage and checked again before pouring concrete. The steel reinforcement should be centered in the drilled hole. Concrete used for pier construction should be discharged vertically into the holes to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during construction.

As discussed in the 2023 geotechnical report, free groundwater could be as shallow as about 17 feet bgs at the Site. However, the actual depth at which groundwater may be encountered in excavations may vary and it is possible that seepage water could be encountered within the anticipated depth of excavations for the project. If water more than 6 inches deep is present during concrete placement, either the water needs to be pumped out or the concrete needs to be placed into the hole using tremie methods. If tremie methods are used, the end of the tremie pipe must remain below the surface of the in-place concrete at all times. Unit prices for dewatering and/or tremie placement methods should be obtained during the bidding process.

Concrete for drilled piers should be designed and placed in general conformance with the recommendations provided in ACI 336.3R-14, Design and Construction of Drilled Piers. The recommendations provided within ACI 336.3R-14 should be followed, especially when concrete placement is necessary below groundwater level, in caving or sloughing soils, or in sand, which may necessitate casing or the slurry displacement method for concrete placement. These methods require concrete placement at higher slumps than "dry" conditions and concrete mix specifications, including the addition of concrete admixtures and consideration of consolidation methods, should be provided by the design team. If temporary casing is used, it should consist of smooth walled steel. **Corrugated metal pipe (CMP) should not be used as temporary casing because it has a tendency to create voids or disturbed zones during removal. BSK should be notified if permanent casing is planned.**

Asphalt Concrete Pavements

Pavements for this project may include asphalt-paved parking and driveways. We have developed our pavement designs assuming the pavement subgrade soil will be similar to the near surface soils described in the boring logs. If site grading exposes soil other than that assumed, or import fill is used to construct pavement subgrades, we should perform additional tests to confirm or revise the recommended pavement sections for actual field conditions.

Asphalt pavement sections for this project have been calculated using Caltrans Flexible Pavement Design Method. Based on near-surface conditions, we have used an assumed design R-value of 5 in our analyses and we have developed the pavement sections presented in the table below. These sections take into consideration the moderate expansion potential of the near surface soils. Various alternative pavement sections for various Traffic Indices (TIs) are presented. Each TI represents a different level of use. The owner or designer should determine which level of use best reflects the project and select appropriate pavement sections.

ASPHALT CONCRETE PAVEMENT DESIGN Design R-Value = 5		
Traffic index	**AC**	**AB**
4.0	2.5	9.0
4.5	2.5	10.5
5.0	2.5	13.0
5.5	3.0	14.0
6.0	3.0	15.5
6.5	3.5	17.0

Note: Thicknesses shown are in inches.
AC = Type B Asphalt Concrete
AB = Class 2 Aggregate Base (Minimum R-Value = 78)

Concrete Pavements

If used, Portland Cement Concrete (PCC) pavement should have a minimum thickness of 6 inches supported over 6 inches of Caltrans Class 2 aggregate base over subgrade prepared per our 2023 geotechnical report. The aggregate base and subgrade for PCC pavements should be properly moisture conditioned and compacted. Construction joints should be located no more than 12 feet apart in both directions. Concrete compressive strength should be tested in lieu of third point loading for rupture strength. A minimum 28-day compressive strength of 3,000 pounds per square inch (psi) should be specified for the concrete mix design. The PCC pavement should be continuously reinforced using No. 4 bars (or larger) spaced no more than 18 inches on center in both directions. Steel reinforcement should be located near the mid-thickness of the concrete slab. Final design of the PCC pavement is the responsibility of the civil or structural engineer for the project.

Pavement Drainage

Paved areas should be sloped and drainage gradients maintained to carry all surface water to appropriate collection points. Surface water ponding should not be allowed anywhere on the Site during or after construction. We recommend that the pavement section be isolated from non-developed areas and areas of intrusion of irrigation water from landscaped areas, unless these areas are located at least 10 feet laterally from the pavement. Concrete curbs should extend a minimum of 2 inches below the aggregate base and into the subgrade to provide a barrier against drying of the subgrade soils, or reduction of lateral migration of landscape water, into the pavement section. Alternatively, a moisture barrier at least 80 mils thick that extends at least 6 inches below the aggregate base into the subgrade soils could be installed immediately behind concrete curbs.

In addition, we recommend that all pavements conform to the following criteria:

- All trench backfills within pavements, including utility and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade, in accordance with the compaction recommendations in our 2023 geotechnical report.
- Where utility trenches extend into a paved area from an undeveloped area, a 2-foot-thick “plug” or “cutoff” consisting of CLSM should be installed at the edges of the pavement for the entire depth of the trench to reduce the chance for the trench to act as a conduit for water under the pavement.
- An adequate drainage system should be provided to prevent surface water or subsurface seepage from saturating the subgrade soil.
- The asphalt concrete, aggregate base, and aggregate subbase materials should conform to Caltrans Specifications, latest edition.
- Placement and compaction of pavements should be performed in accordance to appropriate Caltrans procedures.

CLOSURE

All the recommendations in our 2023 geotechnical and geologic and seismic hazards assessment report are still valid for the new portable classroom building and three new shade structures . The limitations presented in our 2023 geotechnical report are applicable to this supplemental letter. We appreciate the opportunity of providing our services to you on this project and trust this letter meets your needs at this time. If you have any questions concerning the information presented, please contact us at (925) 315-3151.

Respectfully Submitted,

BSK Associates, Inc.

Danaige Tower, EIT
Associate I

Carrie L. Rodriguez, PE, GE #3016
Principal

REGISTERED PROFESSIONAL ENGINEER
CARRIE L. RODRIGUEZ
No. 3016
Exp. 09/30/27
GEOTECHNICAL
STATE OF CALIFORNIA

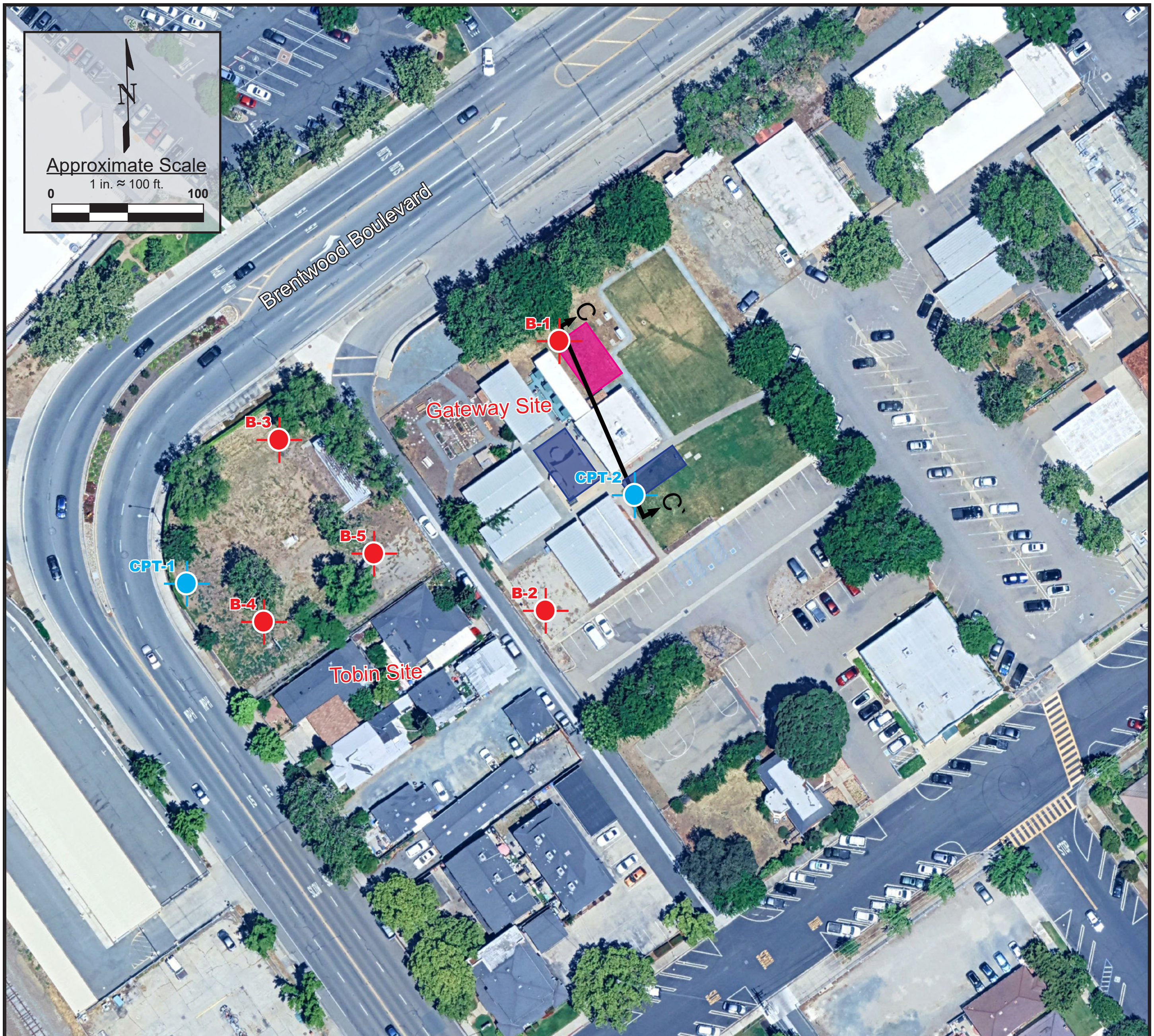
ENCLOSURES:

Figure 1 – Supplemental Site Plan

Figure 2 – Supplemental Geologic Cross Section

Appendix A – Geotechnical Investigation and Geologic and Seismic Hazards Assessment Report (BSK, 2023)

FIGURES



References: 1. http://earth.google.com, 2025
Notes: Locations are approximate

Legend

- B-1 Approximate Boring Location (BSK, 2023)
- CPT-1 Approximate CPT Location (BSK, 2023)
- Planned Shade Structures
- Planned Portable Building
- C C` Cross Section Line (See Figure 2)

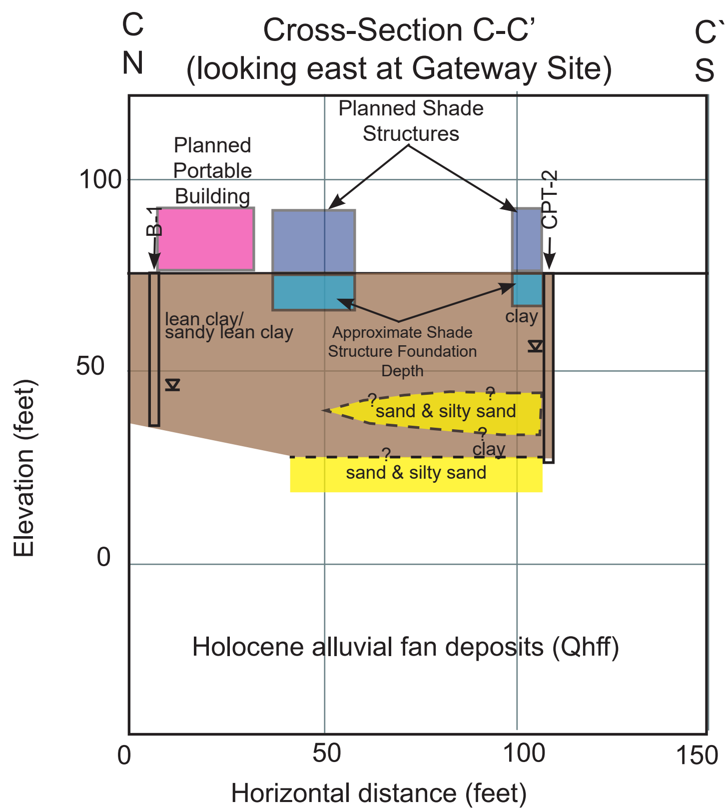
BSK ASSOCIATES

PROJECT NO.	G25001215
DRAWN:	1/30/26
DRAWN BY:	D. Tower
CHECKED BY:	C. Rodriguez
FILE NAME:	Figures.indd

SUPPLEMENTAL SITE PLAN

Liberty Adult Community Education School Gateway Additional Portable Building and Shade Structures Brentwood, California

FIGURE 1



LEGEND

Soil boring or CPT Location

– –?– Unit contact

Groundwater level

BSK ASSOCIATES

PROJECT NO.	G25001215
DRAWN:	1/30/26
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SUPPLEMENTAL

GEOLOGIC CROSS SECTION

Liberty Adult Community Education School Gateway Additional Portable Building and Shade Structures Brentwood, California

Figure

2

APPENDIX A

GEOTECHNICAL INVESTIGATION REPORT AND GEOLOGIC AND SEISMIC HAZARDS ASSESSMENT REPORT (BSK, 2023)

GEOTECHNICAL INVESTIGATION AND GEOLOGIC AND SEISMIC HAZARDS ASSESSMENT REPORT

LIBERTY ADULT EDUCATION CENTER
GATEWAY AND TOBIN SITES
BRENTWOOD, CALIFORNIA

BSK PROJECT NO. G00000614

PREPARED FOR:

LIBERTY UNION HIGH SCHOOL DISTRICT
20 OAK STREET
BRENTWOOD, CALIFORNIA 94513

AUGUST 4, 2023

399 Lindbergh Avenue
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August 4, 2023 BSK Project No. G00000614

Liberty Union High School District
20 Oak Street
Brentwood, CA 94513

ATTENTION: Mr. Paul Melloni

SUBJECT: **Geotechnical Investigation and Geologic and Seismic Hazards Assessment Report**
Liberty Adult Community Education School - Gateway and Tobin Sites
Brentwood, California

Dear Mr. Melloni:

We are pleased to submit our geotechnical investigation and geologic and seismic hazards assessment report for the planned portable building project at the Liberty Adult Community Education School, Gateway and Tobin sites (Site) in Brentwood, California. The enclosed report provides a description of the investigation performed and presents geotechnical site characterization, geologic and seismic hazards assessments, and design recommendations. This report was prepared in accordance with the 2022 California Building Code (CBC), California Code of Regulations, Title 24, Chapters 16A and 18A requirements for a Geotechnical/Engineering Geologic Report.

In summary, it is our opinion that the Site does not pose significant geotechnical concerns that would preclude the planned improvements provided the recommendations presented in our report are incorporated into the design and construction of the project. The main geotechnical concerns for the project Site are the presence of moderately expansive surface clays and near surface soils subject to moderate collapse potential. The buildings can be supported on spread footings, deepened to mitigate the moderately expansive soils or designed to withstand differential movement due to expansive soil and potentially collapsible soils at the Site.

The conclusions and recommendations presented in the enclosed report are based on limited subsurface investigation and laboratory testing programs. Consequently, variations between anticipated and actual subsurface conditions may be found in localized areas during construction. If significant variation in the subsurface conditions is encountered during construction, BSK should review the recommendations presented herein and provide supplemental recommendations, if necessary.

Additionally, design plans should be reviewed by our office prior to their issuance for conformance with the general intent of our recommendations presented in the enclosed report.

We appreciate the opportunity of providing our services to you on this project and trust this report meets your needs at this time. If you have any questions concerning the information presented, please contact us at (925) 315-3151.

Respectfully Submitted,
BSK Associates, Inc.

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Figures and Appendices

FIGURES

1. INTRODUCTION

This report presents the results of our geotechnical investigation and geologic and seismic hazards assessment for the construction of six (6) new portable buildings at the Tobin site and evaluation of seven (7) existing buildings at the Gateway site at the Liberty Adult Community Education School in Brentwood, California. This report was prepared in accordance with the 2022 California Building Code (CBC), California Code of Regulations, Title 24, Chapters 16A and 18A requirements for a Geotechnical/Engineering Geologic Report, and the California Geological Survey (CGS) Note 48 (2022).

1.1 Project Description

The proposed project consists of constructing six (6) new portable buildings at the Tobin Site and evaluation of seven (7) existing portable buildings at the Gateway Site. The general location of the project site (Site) is shown on the attached Site Plan, Figure 2. No significant site grading is planned. Other pertinent improvements may include asphalt-paved parking areas and courtyards, landscaping, new concrete flatwork, and underground utility lines.

A grading plan has not yet been developed, but we assume cuts and fills of less than 2 feet deep would be necessary for building pad preparation. Excavations up to 5 feet deep are anticipated for the installation of utility trenches.

If the actual project differs significantly from that described above, we should be contacted to review and/or revise our conclusions and recommendations presented in this report.

1.2 Approach and Scope of Services

The purpose of this investigation and geologic and seismic hazards assessment was to evaluate the subsurface conditions at the Site in order to provide geotechnical input for the existing portable buildings and design and construction of the planned portable buildings and the associated earthwork for this project as well as provide the Liberty Union High School District (District) with an evaluation of potential geologic or seismic hazards that may be present at the Site or due to regional influences. The scope of services consisted of:

- Review of published geologic literature of the area;
- Evaluation of the data collected/reviewed and preparation of geologic cross section(s);
- Evaluation of potential geologic hazards affecting the Site;
- Selection of Site Class and code-based seismic design parameters; and
- Engineering analysis and preparation of this report.

Our scope of services is described in our proposal for this project entitled “Proposal for Geotechnical Investigation and Geologic and Seismic Hazards Assessment, Liberty Adult Community Education School – Gateway and Tobin Sites, Livermore, California” dated March 21, 2023, BSK Proposal No. G00000614.

This investigation specifically excludes the assessment of site environmental characteristics in the air, water, or soil, particularly those involving hazardous substances, and the evaluation of a high-pressure pipeline risk.

2. SITE INVESTIGATION

2.1 Field Investigation

Our field investigation was performed on May 16th and June 12th, 2023 to evaluate the subsurface conditions at the Site. The field investigation consisted of advancing two (2) Cone Penetration Tests (CPTs), labeled CPT-1 and CPT-2, to depths of approximately 50 and 100 feet below the existing ground surface (BGS) and drilling five (5) borings, labeled B-1 through B-5, to a maximum depth of approximately 40 feet BGS. The approximate locations of these exploration points are shown on Figure 2.

Prior to subsurface exploration, Underground Service Alert (USA North 811) was contacted to provide utility clearance and each exploration location was cleared for detectable underground utilities by GeoTech Utility Locating of Moraga, California. Drilling permits were obtained from Contra Costa County. Upon completion of the field investigation, the borings and CPTs were backfilled with grout. Excess cuttings generated during drilling were disposed of at the Site in an unimproved area.

The locations of our exploration points were estimated by our field representative based on rough measurements from existing features at the Site. The elevations shown on the boring logs were estimated based on Google Earth Pro. As such the elevations and locations of the exploration points should be considered approximate to the degree implied by the methods used.

2.1.1 Auger Borings

The borings were drilled, using a truck-mounted drill rig to a maximum depth of approximately 40 feet BGS. Exploration GeoServices of San Jose, California was subcontracted for the drilling and sampling. The borings were logged by a staff geologist of BSK Associates (BSK) in general accordance with the ASTM *Standard of Practice for Description and Identification of Soils (Visual-Manual Procedure)* (D2488).

Relatively undisturbed samples of the subsurface materials were obtained using a split spoon sampler fitted with stainless steel liners. The general diameter measurements of the sampler are about 3-inches for the outside diameter and about 2½-inches for the inside diameter (I.D.) fitted with stainless steel liners. The sampler was driven by the force of a 140-pound, semi-automatic trip hammer, dropping 30-inches. The successive blow counts were recorded for 6-inch penetration intervals until the sampler advanced 18-inches, or until practical refusal. The blow counts for each interval are reported on the final boring logs. After the samplers were withdrawn from the boreholes, the soil samples were removed from the samplers, sealed to reduce moisture loss, labeled, and returned to our laboratory. Prior to sealing the samples, strength characteristics of the cohesive soil samples recovered were evaluated using a handheld pocket penetrometer. The results of these tests are shown adjacent to the samples on the boring logs.

Soil classifications made in the field, based on visual/manual assessment of the auger cuttings and samples, were re-evaluated in the laboratory after further examination and testing. Where laboratory tests were performed, the results appear in the final boring logs, except for the corrosion test results, which are found in Appendix B. Final soil classification was confirmed through the judgement of a responsible Geotechnical Engineer supplemented with the laboratory testing at various intervals in general accordance with the ASTM *Standard Practice for Classification of Soils for Engineering Purposes* (D2487). A summary of the Unified Soil Classification System (USCS), adapted by ASTM D2487 and D2488, is presented in Appendix A, Figure A-1. The Soil Description Key and Log Key are presented on Figures A-2 and A-3. Sample classifications and blow counts recorded during sampling, and other related information are presented on the soil boring logs, within Appendix A. Discussion of the subsurface conditions encountered at the Site is presented in the "Subsurface Conditions" section of this report.

2.1.2 Cone Penetration Tests

The cone penetration test probes were advanced to depths of approximately 50 and 100 feet BGS. Middle Earth Geo Testing of Hayward, California was subcontracted to provide CPT services. The CPTs were performed using an integrated electronic cone system in accordance with ASTM D3441. The cone has a tip area of 10 square centimeters, a friction sleeve area of 150 square centimeters, and a ratio of end area friction sleeve to tip end area equal to 0.80. The cone resistance and sleeve friction were measured and recorded during the tests at approximately 5-centimeter (about 2 inches) depth intervals. In addition, shear wave velocity measurements were taken every approximately 5 feet at CPT-1.

The cone system was pushed using a 40,000-pound, all-wheel drive, CPT rig, having a down pressure capacity of approximately 20 tons. The information gathered from the CPTs was used for identifying potentially liquefiable and soft soils and for foundation design. The correlated CPT data collected from our CPT (cone resistance, friction ratio, pore pressure, and soil behavior type) versus penetration depth below the existing ground surface, generated using the CPT liquefaction assessment computer software CLiq[1], is presented in Appendix D along with shear wave velocity measurements.

The stratigraphic interpretation of the CPT data was performed based on relationships between cone resistance (also known as tip resistance) and sleeve friction versus penetration depth. The friction ratio, which is sleeve friction divided by cone resistance, is a calculated parameter which is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate large excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone resistance and generate small excess pore water pressures. The interpretation of soil properties from the cone data has been carried out using correlations developed by Robertson et al, 1990[2], and Lunne, Robertson & Powell, 1997[3]. It should be noted that it is not always possible to clearly identify a soil type based on cone resistance and sleeve friction. In these situations, experience and judgment and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type.

---

[1] CLiq v2.0 by Geologismiki

[2] Robertson P.K., 1990. Soil classification using the cone penetration test. Canadian Geotechnical Journal, 27(1): 151-158

[3] Lunne, T., Robertson, P.K., and Powell, J.J.M 1997. Cone penetration testing in geotechnical practice, E & FN Spon Routledge, 352 p, ISBN 0-7514-0393-8.

2.2 Laboratory Testing

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program included dry density and moisture content, Atterberg limits, percent passing #200 sieve, unconsolidated-undrained triaxial compression (TXUU), and collapse potential. Most of the laboratory test results are presented on the individual boring logs. The results of the Atterberg limits, TXUU, and collapse potential are also presented graphically in Appendix B.

Analytical testing was performed on samples of near-surface soils in borings B-3 and B-5 to assist in evaluating the corrosion potential of the on-site soils. The corrosivity testing was performed by CERCO Analytical of Concord, California, a State-Certified laboratory, using ASTM methods as described in the CERCO Analytical report. The corrosion results are presented in Appendix B and are discussed further in the "Corrosivity Results" section of this report.

3. GEOLOGIC SETTING AND SUBSURFACE CONDITIONS

3.1 Site Location and Description

The project sites, Gateway and Tobin are two neighboring parcels separated by Diablo Way and are located at 929 2nd Street in Brentwood, California (Site). A Vicinity Map showing the location of the Site is presented on Figure 1.

The campus is surrounded primarily by commercial properties and is bordered by Brentwood Boulevard to the northwest and southwest, 2nd Street to the northeast, and Pine Street to the southeast. The proposed portable buildings for the Tobin project site and the existing portable buildings for the Gateway project site are located as shown on the Site Plan, Figure 2. An existing classroom building currently occupies the proposed portable building locations at the Tobin project site. Based on our review of historical aerial photographs, the existing portable buildings for the Gateway project site were constructed between 2005 and 2009.

As shown on the Area Topographic Map, Figure 3, the Site and surroundings are generally level with minor topographic relief with an estimated ground surface elevation of approximately 75 feet according to Google Earth Pro.

The approximate coordinates near the center of the Site are:
Latitude: 37.935263˚N Longitude: -121.698019˚W

3.2 Site Geology

The City of Brentwood is located in the California Delta region of the Great Valley geomorphic province of California near the eastern boundary of the Coast Ranges geomorphic province. The Great Valley is a 400-mile long, low-relief, alluvial plain which runs north-south through California. The valley contains alluvial sediments which have been deposited almost continuously for the past 160 million years. Liberty Adult Community Education School is located in the upland region of the southwest portion of the San Joaquin River Delta complex. To the west, the area transitions to the Coast Ranges province that is characterized by northwest trending ridges and valleys that are typically highly folded with numerous faults (CGS, 2002).

According to Knudsen et al. (2000) and the CGS (2018), who have differentiated Quaternary deposits between Holocene and Pleistocene age, the northeast half of the Site is underlain by Holocene alluvial fan deposits, fine facies (map symbol Qhff) and the southwest half of the Site is underlain by Holocene alluvial fan deposits (map symbol Qhf). Locally, the alluvial fan deposits originated from the eroded surrounding hills and deposited into the inter-ridge valley and delta-alluvial plains. Holocene alluvial fan deposits, fine facies, are characterized as loose to medium dense and dominated by clay and silt with interbedded sand and gravel. A portion of Knudsen et al. (2000) geologic map is presented in Figure C-1, Quaternary Geologic Map.

According to Dibblee and Minch (2006), the Site is underlain by Holocene age alluvial clay of valley areas (map symbol Qc) as shown on Figure C-2, Geologic Map. These deposits generally include alluvial gravel, sand, silt, and clay.

Nearby active faults and potentially active faults, including approximate distances and direction from the Site are presented in Table 1 below:

TABLE 1- NEARBY ACTIVE AND POTENTIALLY ACTIVE FAULTS	
FAULT	**APPROXIMATE DISTANCE (miles), direction**
Davis	3½, southwest
Midland	4, east
Greenville	10, southwest
Rio Vista	11, northwest
Concord	16, west
Las Positas	18, southwest
Calaveras	18½, southwest
Green Valley Fault Zone	24, northwest

3.3 Subsurface Conditions

The soil encountered in our borings consisted predominantly of firm to hard lean clay with varying amounts of sand with one layer of medium dense clayey sand at about 12 feet BGS in boring B-4. The soil encountered in the upper 35 feet BGS in our CPTs, consisted primarily of clay and silty clay layers. Interbedded sand, silty sand and sandy silt layers were encountered at depths ranging from about 35 feet to approximately 82 feet BGS in our CPTs. Below 82 feet BGS, the soil consisted of clay and silty clay layers to the maximum CPT depth explored of 100 feet BGS. Figure C-4, Geologic Cross Sections, shows our interpretation of the subsurface conditions based on soil borings, CPT data and published reports in the area.

Free groundwater was observed at depths between about 29 and 31 feet BGS in our borings, and 21 and 25 feet BGS in our CPTs (via tape drop). According to the CGS, historic high groundwater is about 17 feet BGS at the Site. It should be noted that groundwater levels can fluctuate several feet depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this or adjacent properties.

The above is a general description of soil and groundwater conditions encountered at the Site. For a more detailed description of the soils encountered, refer to the boring logs in Appendix A and CPT logs in Appendix D. It should be noted that subsurface conditions can deviate from those conditions encountered at the boring and CPT locations. If significant variation in the subsurface conditions is encountered during construction, it may be necessary for BSK to review the recommendations presented herein and recommend adjustments as necessary.

4. GEOLOGIC AND SEISMIC HAZARDS

The types of geologic and seismic hazards assessed in this section and the "Site Seismicity" section of this report include surface ground fault rupture, liquefaction, lateral spreading, seismically induced settlement, slope failure, flood hazards, inundation hazards, soil corrosion, soil expansion, soil collapse, and tsunami.

4.1 Fault Rupture Hazard Zones in California

The purpose of the Alquist-Priolo Earthquake Fault Zoning Act, as summarized in CGS Special Publication 42 (2018), is to "address the hazard of surface fault rupture through the regulation of development in areas near Holocene-active faults and prevent the construction of structures for human occupancy across traces of active faults." As indicated by Special Publication 42, "the State Geologist (Chief of the California Geological Survey) is required to delineate 'Earthquake Fault Zones (EFZ)' along known Holocene-active faults in California. The EFZs are distributed as 'Earthquake Fault Zone maps.' The zones are regulatory in nature and are one class of 'Earthquake Zones of Required Investigation', which includes other geologic hazards such as liquefaction and earthquake-induced landslides. 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 the sites are not threatened by surface displacement from future faulting."

The City of Brentwood and the campus are located within the Brentwood 7½-minute quadrangle in Contra Costa County. According to the California Geological Survey (CGS), **the Site is not located in an Earthquake Fault Zone**. As shown on the Alquist-Priolo Earthquake Fault Zone map, Figure C-5, the closest Fault-Rupture Hazard Zone is associated with the Greenville fault zone located approximately 10 miles southwest of the project site (CDMG, 1982).

4.2 State of California Seismic Hazard Zones (Liquefaction and Landslides)

Zones of Required Investigation, referred to as "Seismic Hazard Zones" 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 not located in an Earthquake-Induced Landslide Hazard Zone. However, **the Site is located in a state-delineated Liquefaction Hazard Zone**, as shown on the State Seismic Hazard Zones, Figure C-6 (CGS, 2018).

The results of our liquefaction evaluation are presented in the section entitled "Liquefaction" below.

4.3 Slope Stability and Potential for Slope Failure

The Site is essentially flat, with little to no topographic relief. Therefore, the potential for landslides and slope failures (seismically induced or otherwise) to occur at the Site is considered negligible.

4.4 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.

4.4.1 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 the FEMA Flood Zones map, Figure C-7, according to FEMA Flood Hazard Map Layer GIS data, 06001C-NFHL, dated 05/07/2018, the Site lies in Zone X outside of the 100-year and 500-year floodplains.

4.4.2 Inundation Hazards – Dams

According to the Department of Water Resources, Division of Safety of Dams (DSOD), **most of the Site is located in Marsh Creek main dam inundation zone** (see Figure C-8, Dam Inundation Zones). Marsh Creek dam is located approximately 3.5 miles southwest of the Site.

4.5 Corrosion

Please refer to the section entitled “Corrosivity Results” below for discussion of the corrosivity potential of the Site soils.

4.6 Expansive Soils

The near-surface soils encountered within the borings at the Site consist of lean clay and sandy lean clay **which exhibits a moderate to high expansion potential** when exposed to cycles of moisture fluctuation.

4.7 Soil Collapse Potential

Some soil samples obtained within the upper approximately 11 feet BGS at the Site were observed to be slightly porous to porous. This can be indicative of the soil having a collapse potential upon saturation. Saturation could occur due to many reasons such as a flooded adjacent landscaping area or a leaking underground utility. Therefore, we performed collapse potential testing on two samples displaying the most visible porosity. These samples were obtained from depths of about 6 and 11 feet BGS in boring B-4. According to our test results, **the shallower sample has no collapse potential, while the deeper sample**

has a severe collapse potential according to ASTM D5333. Other samples that exhibited any observable porosity had significantly higher dry density (17 to 22% higher) and moisture content (200% higher). In addition, the void ratio for the severe collapse potential sample was about 30 to 70% higher than the other five samples that had void ratio test results within this depth range. Therefore, the severe collapse potential appears to be limited to the soil layer at a depth of 11 feet BGS in boring B-44 based on the dry density, moisture content, and void ratio.

Using the interpolation method presented in ASTM D5333 to estimate collapse potential magnitude at different applied stresses, we calculated an approximate collapse potential of 9% at 10 feet BGS. (New footings are anticipated to apply negligible additional pressure at this depth). This would result in approximately 3 inches of vertical settlement. However, we anticipate that the settlement from a collapse event will attenuate by about half before reaching the elevation of the bottoms of the footings, therefore, **we estimate total settlement due to soil collapse potential of about 1½ inches at the depth of the bottoms of the footings and differential settlement between adjacent foundation supports or over a horizontal distance of 30 feet, whichever is less, of about ¾ inch**.

4.8 Tsunami Hazard

According to the Tsunami Inundation Map for Emergency Planning (Cal-EMA, 2009), the Site is not located in a California State Tsunami Hazard Zone.

5. SITE SEISMICITY

5.1 Seismic Source Deaggregation

Figures C-9 and C-10, Regional Fault Map and Local Fault Map, respectively, present the major faults that may impact the project area 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 using the USGS Unified Hazard Tool. The deaggregation determination, at the maximum considered earthquake hazard level, results in distance, magnitude, and epsilon (ground-motion uncertainty) for each source that contributes to the hazard.

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 7.28 earthquake from the Calaveras fault. The modal magnitude (Mw) of 6.9 with a distance of 11.58 km would be appropriate for probabilistic input parameter. However, a 2014 earthquake scenario catalog, which includes a set of deterministic ruptures created by the Building Seismic Safety Council 2014 (BSSC2014), shows a possible multi-segment rupture event of the Calaveras fault with a 7.4 Mw earthquake. For liquefaction and seismic settlement, the more conservative earthquake magnitude of 7.4 will be used.

5.2 Historical Seismicity

The Site and its vicinity are located in an area characterized by high seismic activity. A number of large earthquakes have occurred within the Site region during historic time (since 1800). The Historical Earthquakes map, Figure C-11, presents earthquake magnitudes of significant earthquakes based on the National Seismic Hazard Model (NSHM) Earthquake Catalogs. This earthquake catalog is for the Western United States and provides a listing for all known $M \geq 2.5$ earthquakes.

Table 2 below provides the location, earthquake magnitude, site to earthquake distances, dates and the resulting site peak horizontal acceleration for the period 1800 to 2021. The table below shows that the Site has experienced mean plus one sigma peak horizontal acceleration up to 0.32g from the magnitude 8.3 earthquake that occurred on the San Andreas Fault in 1906. In general, the Site has been subjected to moderate intensity ground motion and primarily from moderate to large earthquakes in the region.

TABLE 2
HISTORIC EARTHQUAKES WITHIN 100 MILES OF THE SITE
GROUND MOTION GREATER THAN 0.14G

File Code	Latitude (North)	Longitude (West)	Date	Depth (km)	Earthquake Magnitude	Site Acceleration (g)	Distance mi (km)
DMG	37.700	122.500	4/18/1906	0	8.3	0.32	46.7(75.1)
BRK	37.830	121.810	1/24/1980	0	5.8	0.29	9.5(15.3)
DMG	38.000	121.900	05/19/1889	0	6.0	0.28	11.9(19.1)

TABLE 2 HISTORIC EARTHQUAKES WITHIN 100 MILES OF THE SITE GROUND MOTION GREATER THAN 0.14G							
File Code	Latitude (North)	Longitude (West)	Date	Depth (km)	Earthquake Magnitude	Site Acceleration (g)	Distance mi (km)
DMG	37.700	122.100	10/21/1868	0	6.8	0.23	27.3(43.9)
DMG	37.900	121.700	9/20/1940	0	4.0	0.22	2.4(3.9)
DMG	37.900	121.700	9/19/1940	0	4.0	0.22	2.4(3.9)
DMG	37.800	122.200	06/10/1836	0	6.8	0.22	28.9(46.5)
DMG	38.010	121.820	9/10/1965	16	4.9	0.20	8.4(13.5)
BRK	37.760	121.730	1/27/1980	0	5.4	0.20	12.2(19.7)
BRK	37.810	121.790	1/24/1980	0	5.1	0.19	10.0(16.1)
BRK	37.840	121.790	1/24/1980	0	4.8	0.19	8.3(13.3)
BRK	37.860	121.810	1/25/1980	0	4.6	0.17	8.0(12.9)
DMG	37.600	122.400	06/01/1838	0	7.0	0.17	44.8(72.0)
DMG	37.800	122.000	07/04/1861	0	5.6	0.16	18.9(30.5)
BRK	37.840	121.790	1/25/1980	0	4.4	0.15	8.3(13.3)
DMG	38.400	122.000	04/19/1892	0	6.4	0.15	36.0(58.0)

In March 2015, scientists and engineers released a new earthquake forecast for the State of California, which was compiled by the USGS, the Southern California Earthquake Center, and the CGS with support from the California Earthquake Authority (Field et al., 2014). It updates the earthquake forecast made for the greater San Francisco Bay Area by the 2007 Working Group for California Earthquake Probabilities. According to this recent study, there is a 72 percent probability that one or more magnitude M6.7 or greater earthquakes will occur in the San Francisco Bay Area between 2014 and 2044. As has been demonstrated recently by the 1989 (M6.9) Loma Prieta, the 1994 (M6.7) Northridge, and the 1995 (M6.9) Kobe earthquakes, earthquakes of this magnitude range can cause severe ground shaking and significant damage to modern urban environments.

5.3 Earthquake Ground Motion, 2022 California Building Standards Code

5.3.1 Site Class

Based on Section 1613A.2.2 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 average shear wave velocity (V_{S30}), CPT-1 measured 822 feet/second (250 meters/second), as per Table 20.3-1 of ASCE 7-16, the Site is Class D (Stiff Soil; 600 ft/ s ≤ Vs ≤ 1,200 ft/s).

5.3.2 2022 Seismic Design Criteria

The 2022 California Building Code (CBC) utilizes ground motion based on the Risk-Targeted Maximum Considered Earthquake (MCE). The Risk-Targeted MCE 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 an adjustment for targeted risk. Ground motion parameters in the 2022 CBC are based on ASCE 7-16, Chapter 11, Supplement 3.

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 3 below presents the spectral acceleration parameters produced for Site Class D by the OSHPD Ground Motion Parameter Application and Chapter 16 of the 2022 CBC based on ASCE 7-16.

TABLE 3
SPECTRAL ACCELERATION PARAMETERS
RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE

Criteria	Value		Reference
MCE Mapped Spectral Acceleration (g)	S_S = 1.377	S_1 = 0.484	USGS Mapped Value
Site Coefficients (Site Class D)	F_a = 1.000	F_v = null (1.816)[a]	ASCE Table 11.4
Site Adjusted MCE Spectral Acceleration (g)	S_{MS} = 1.377	S_{M1} = null (0.879)[a]	ASCE Equations 11.4.1-2
Site Adjusted MCE Spectral Acceleration (g)	NA	*S_{M1} = 1.318*[b]	*ASCE Equations 11.4.1-2*
Design Spectral Acceleration (g)	S_{DS} = 0.918	S_{D1} = null (0.586)[a]	ASCE Equations 11.4.3-4
Design Spectral Acceleration (g)	NA	*S_{D1} = (0.879)*[b]	*ASCE Equations 11.4.3-4*
Site Short Period - T_S (Seconds)	Ts =0.638		Ts = S_{D1}/ S_{DS}
Site Short Period - T_S (Seconds)	*Ts =0.957*[b]		*Ts = S_{D1}/ S_{DS}*
Site Long-Period - T_L (Seconds)	T_L = 8		USGS Mapped Value

[a] 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.

[b] Values include 50% increase per ASCE 7-16 Supplement 3 for Site Class D with S_1 greater than or equal to 0.2.

5.3.3 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 21.2) 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 4 based on Site Class D.

TABLE 4
GEOMETRIC MEAN PEAK GROUND ACCELERATION
MAXIMUM CONSIDERED EARTHQUAKE

Criteria	Value	Reference
Mapped PGA (g)	PGA = 0.567	USGS Mapped Value
Site Coefficients (Site Class D)	F_{PGA} = 1.100	ASCE Table 11.8-1
Geometric Mean PGA (g)	PGA_M = 0.624	ASCE Equation 11.8-1

5.4 Seismically Induced Ground Failure

5.4.1 Liquefaction

Settlement of the ground surface with consequential differential movement of structures is a major cause of seismic damage for buildings founded on alluvial deposits. Vibration settlement of relatively dry and loose granular deposits beneath structures can by readily induced by the horizonal components of ground shaking associated with even moderate intensity earthquakes. Silver and Seed (1971) have demonstrated that settlement of dry sands due to cyclic loading is a function of 1) the relative density of the soil; 2) the magnitude of the cyclic shear stress; and 3) the number of strain cycles. As indicated above, seismically induced ground settlement can also occur due to the liquefaction of relatively loose, saturated granular deposits. In order for liquefaction to triggering to occur due to ground shaking, it is generally accepted that four conditions will exist:

- The subsurface soils are in a relatively loose state,
- The soils are saturated,
- The soils have low plasticity, and
- Ground shaking is of sufficient intensity to act as a triggering mechanism.

In addition, after the soil liquefies, dissipation of the excess pore pressures can produce volume changes within the liquefied soil layer, which can result in ground surface settlement.

The Site is located within a State mapped liquefaction zone; therefore, we performed liquefaction analyses for our CPTs using the methods proposed by Boulanger and Idriss (2014)[4] using the program CLiq v.3.3.2.9 by Geologismiki. For our analyses, we used a peak ground acceleration of 0.63g associated with an earthquake magnitude of M7.49 and a design groundwater of 17 feet BGS. Our analysis estimates a maximum of about 0.4 inches of liquefaction-induced settlement could occur at the Site during a design level seismic event (see Appendix D for liquefaction results). See Table 5 below for additional information on liquefaction magnitudes and depths for CPT-1 and CPT-2.

TABLE 5
ESTIMATED LIQUEFACTION-INDUCED SETTLEMENT

CPT-1 (inch, depth range)	CPT-2 (inch, depth range)
<0.1 inch (11 feet)	0.35 inch (33 to 36 feet)
0.15 inch (35 to 48 feet)	0.05 inch (48 to 50 feet)

Based on the estimated settlement magnitudes and depth at which settlement is anticipated to occur, we anticipate the settlement magnitude to attenuate by about half at the depth of the footings. Therefore,

[4] Boulanger, R. W., and Idriss, I. M. (2014), CPT and SPT based liquefaction triggering procedures. Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.

we anticipate total settlement due to liquefaction to be about ¼ inch and differential settlement to be less than ¼ inch between adjacent foundation supports or across a horizontal distance of 30 feet, whichever is less.

5.4.2 Lateral Spreading

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 Site is relatively flat and there are no free faces in the vicinity of the Site; **therefore, we conclude that the potential for lateral spreading to occur at the Site is negligible.**

5.4.3 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. These types of soils were not encountered within the maximum depth of the borings and CPTs of approximately 100 feet BGS. **Therefore, we conclude that the potential for seismic settlement to occur at the Site is low.**

6. CONCLUSIONS AND RECOMMENDATIONS

Presented below are recommendations for foundations, seismic considerations, exterior flatwork, earthwork, construction considerations, and site drainage for this project.

6.1 Shallow Foundations

Based on our investigation, the loads for the proposed portable buildings can be supported by continuous perimeter footings and isolated interior footings bearing on native undisturbed soil or engineered fill provided that the bottom of the footing excavations have been checked by a BSK representative. The recommended allowable soil bearing pressures in pounds per square foot (psf) are presented below.

TABLE 6
FOOTING DESIGN CRITERIA

Static Allowable Bearing Capacity[1]	1,800 psf
Seismic/Wind Allowable Bearing Capacity[1]	2,700 psf
Passive Resistance (Equivalent Fluid Pressure)[2,3]	350 pcf
Allowable Lateral Sliding Resistance Adhesion[3]	500 psf
Minimum Embedment Depth[4]	18 inches
Minimum Width	12 inches (continuous) 18 inches (isolated)

Notes:

1. Includes a factor of safety of at least 3 for static loading and at least 2 for transient loading (i.e., seismic or wind conditions).
2. Neglect upper 1 foot if surface is not confined by concrete slab or pavement. **For foundations located on or proximate to sloping ground such as bioretention areas, the passive resistance should be neglected in the upper portion of the foundation until there is a horizontal distance of at least 7 feet between the slope face and the nearest edge of the foundation.**
3. The sliding resistance and passive resistance may be used concurrently, and the passive resistance can be increased by one-third for wind and/or seismic loading. Values include a factor of safety of at least 1½. The sliding resistance adhesion should be multiplied by the foundation area to obtain horizontal sliding resistance.
4. Below lowest adjacent grade considered to be bottom of slab on the interior or finished grade on the exterior.

It should be noted that because of the moderately to highly expansive nature of the near-surface soils, the actual bearing capacities should not be less than 1,300 psf to reduce potential movement of the foundations as a result of expansion potential of the surface soils. Footings with bearing pressures below 1,300 psf may need to be extended 6 inches deeper, to compensate for the lower pressure.

If footings are supported at the ground surface, the surface should be covered by a minimum of 4-inches of Caltrans Class 2 aggregate base and up to about ½-inch of shrink and swell of the ground surface should be anticipated.

6.1.1 Shallow Foundation Settlement

Shallow foundation settlements will consist of elastic settlement, which will occur relatively soon after application of the load, collapse-induced settlement, and liquefaction-induced settlement, The approximate magnitudes of settlement at the foundation depth are included in the table below along with cumulative total and differential settlements.

TABLE 7 SHALLOW FOUNDATION SETTLEMENT		
Settlement Type	**Total (inches)**	**Differential (inches)***
Collapse-Induced	1½	¾
Liquefaction-Induced	¼	< ¼
Immediate**	½	< ¼
Cumulative	2¼	1
Notes: *Over a horizontal distance of approximately 30 feet. **Anticipated to occur relatively soon after application of load.		

6.1.2 Additional Considerations for Shallow Foundations

Where foundations are located adjacent to below-grade structures (including existing footings) or near major underground utilities, the foundation should extend below a 1H:1V (horizontal to vertical) plane projected upward from the structure foundation or bottom of the underground utility to avoid surcharging the below grade structure and underground utility with foundation loads. Where this is not possible or feasible, we recommend that CLSM be used to backfill the portion of the utility trench that extends below the 1H:1V projection. Also, if a utility crosses perpendicular to a footing, if it is located within 2 x W of the bottom of the footing, where W = width of footing, the utility should be encased in CLSM or lean concrete. If a perpendicular utility is located below a depth of 2 x W below the footing, the utility does not need to be encased in CLSM or lean concrete, but the trench should be backfilled with impervious material a distance of 2 feet laterally on each side of the perimeter footing centerline as recommended in the "Excavation and Backfill" section of this report.

Concrete for foundations should be placed neat against firm native soil or engineered fill. **It is critical that foundation excavations not be allowed to dry before placing concrete.** If shrinkage cracks appear in the foundation excavations, the excavations should be thoroughly moistened to close all cracks prior to concrete placement. The foundation excavations should be monitored by a representative of BSK for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials.

Where utilities cross under the perimeter footings line and enter "interior" space, the trench backfill should consist of a vertical barrier of impervious type material as explained in the "Excavation and Backfill" section of this report. In addition, where utilities cross through footings, flexible waterproof caulking should be provided between the sleeve and the pipe. Utility plans should be reviewed by BSK prior to trenching for conformance to these requirements.

6.2 Slabs-on-Grade

Slabs-on-grade for this project may consist of exterior flatwork. The near-surface soils are moderately to highly expansive and will be subject to shrink/swell cycles with fluctuations in moisture content. To reduce these potentially adverse effects, we recommend that exterior flatwork be underlain by 6 inches of "non-expansive" engineered fill, placed on subgrade prepared as described in the "Earthwork" section of this report. The properties of this "non-expansive" fill should also meet the criteria listed in the "Earthwork" section of this report.

The "non-expansive" fill should extend a minimum horizontal distance of 3 feet beyond exterior flatwork, where achievable. It is important that placement of this material be done as soon as possible after compaction of the subgrade to prevent drying of the native subgrade soils and that slabs be constructed as soon as possible after "non-expansive" material soil is placed, as subgrades will dry out even through "non-expansive" fills. A representative of BSK should be present to observe the condition of the subgrade and observe and test the installation of the "non-expansive" engineered fill soil prior to slab construction.

6.2.1 Exterior Concrete Flatwork

Exterior concrete flatwork for this project will likely consist of walkway areas adjacent to the new portable buildings. As previously discussed, the near-surface soils exhibit a moderate to high expansion potential and can be subject to shrink/swell cycles with fluctuations in moisture content. Some of the adverse effects of swelling and shrinking can be reduced with proper moisture treatment. The intent is to reduce the fluctuations in moisture content by moisture conditioning the soils, sealing the moisture in, and controlling it. Near-surface soils should be moisture conditioned according to the recommendations in Appendix E. In addition, all exterior concrete slabs should be supported on a minimum of 6 inches of "non-expansive" imported fill. Even with the 6 inches of "non-expansive" fill, some movement of exterior slabs may occur. Where concrete flatwork is to be exposed to vehicle traffic, the 6 inches of "non-expansive" fill should consist of Class 2 Aggregate Base as specified in the current California Department of Transportation Standard Specifications.

Practices recommended by the Portland Cement Association (PCA) and the American Concrete Institute (ACI) for proper placement and curing of concrete, as well as for joint spacing and construction, should be followed during exterior concrete flatwork slab construction. New pedestrian concrete flatwork should have a minimum thickness of 4 inches and minimum reinforcing of #4 bars at 18 inches on center (both ways). The rebar should be discontinued at expansion joints. Slip Dowels should be used at expansion joints. Buildings, foundations, and other structures should be structurally isolated from any adjacent exterior concrete flatwork by a felt strip or similar material. Final design of exterior concrete flatwork is the responsibility of the civil or structural engineer for the project.

Exterior flatwork will be subjected to edge effects due to the drying out of subgrade soils. To protect against edge effects adjacent to unprotected areas, such as vacant or landscaped areas, lateral cutoffs, such as inverted curbs that extend at least 2 inches below the aggregate base or "non-expansive" fill into the subgrade soils, are recommended. Alternatively, a moisture barrier at least 80 mils thick extending at least 6 inches below the aggregate base or “non-expansive” fill into the subgrade soils could be installed at the edge of the flatwork. Because of the moderately to highly expansive soils, flatwork should have control joints on no greater than 8 feet on center.

Prior to construction of the flatwork, the “non-expansive” fill layer should be moisture conditioned to near optimum moisture content. If the “non-expansive” fill layer is not covered within 30 days after placement, the soils below this material will need to be checked for appropriate moisture of at least 2 percent over optimum. If the moisture is found to be below this level, the flatwork areas will need to be moisture conditioned until the proper moisture content is reached. Where flatwork is adjacent to curbs, reinforcing bars should be placed between the flatwork and the curbs. Expansion joint material should be used between flatwork and curbs, and flatwork and buildings.

Our recommendations above are intended to reduce the effects of expansive soil subgrade conditions in exterior concrete flatwork areas. However, some seasonal movement of exterior flatwork should be anticipated due to the inability to completely eliminate soil movement in expansive soil areas.

6.3 Demolition

6.3.1 Existing Improvements

As part of the demolition process, existing foundations and other improvements should be removed. Excavations from removal of foundations, underground utilities or other below ground obstructions should be cleaned of loose soil and deleterious material and backfilled with properly compacted fill. As discussed in the “Earthwork” section of this report, following stripping and removal of deleterious materials, areas of the Site to receive fill should be scarified to a minimum depth of 12 inches, moisture-conditioned, and recompacted as indicated in Appendix E. This process should be observed and tested by a BSK representative.

6.3.2 Existing Utilities

Active or inactive utilities within the construction area should be protected, relocated, or abandoned. Pipelines that are 2 inches in diameter or less may be left in place beneath improvements provided they are cut off and capped at the perimeter of the improvement. Pipelines larger than 2 inches in diameter within the planned improvements should be removed or filled with CLSM. Active utilities to be reused should be carefully located and protected during demolition and during construction.

6.4 Earthwork

Earthwork at the Site will generally consist of subgrade preparation for concrete slabs and building pads, and excavation, removal, and backfill for existing and new underground utility line trenches. We anticipate that the required grading will consist of cuts and fills up to 2 feet to create a building pad and grade the

Site to drain. Excavations for the removal of existing underground utilities and installation of new ones are expected to be up to 5 feet deep. BSK should review the final grading plans for conformance to our design recommendations prior to construction bidding. In addition, it is important that a representative of BSK observe and evaluate the competency of existing soils or new fill underlying structures and concrete flatwork. In general, soft/loose or unsuitable materials encountered should be overexcavated, removed, and replaced with compacted engineered fill material.

Site preparation and grading for this project should be performed in accordance with the site-specific recommendations provided below. A summary of compaction requirements for this project is presented in Appendix E. Additional earthwork recommendations are presented in related sections of this report.

6.4.1 Site Preparation and Grading

Prior to the start of grading and subgrade preparation operations, where appropriate, the Site should first be cleared and stripped (minimum of 3 inches deep) to remove all surface vegetation, organic laden topsoil and landscaping located within the Site. Stripped topsoil from landscaped areas may be stockpiled for later use in landscaping areas; however, this material should not be reused for engineered fill.

The root system of trees to be demolished should be removed such that there are no roots thicker than approximately 1-inch in diameter and the remaining organics are no greater than 3 percent by dry unit weight within the upper 5 feet of finished subgrade or the depth of the root ball, whichever is shallower and laterally a minimum of 5 feet beyond the limits of the improvements (defined as the outside perimeter of walls or footing outer limits, whichever results in the greatest building envelope) and 3 feet beyond the edge of flatwork, pavers, and pavement where achievable. A BSK representative should check the excavation bottoms before they are backfilled with engineered backfill[5]. Following stripping, removal of deleterious materials, and overexcavation (if required), the Site should be scarified to a minimum depth of 12 inches, moisture conditioned, and recompacted as indicated in Appendix E. Scarification and recompaction should extend laterally a minimum of 5 feet beyond the limits of structures and 3 feet beyond flatwork, where achievable.

All fills should be compacted in lifts of 8-inch maximum uncompacted thickness. A summary of compaction requirements for the project is presented in Appendix E. Laboratory maximum dry density and optimum moisture content relationships should be evaluated based on ASTM Test Designation D1557 (latest edition).

Site preparation (including stripping, clearing, and grubbing) and fill placement should be observed by a BSK representative. It is important that, during the stripping and scarification process, our representative

[5] “Engineered fill” is defined in this report as suitable on-site soil and imported fill that is used to backfill excavations or raise site grade and is properly moisture conditioned and compacted per the requirements of this report. The requirements for the suitability of on-site soils and import fill are provided in the “Fill Material” section of this report.

be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during our field investigation.

6.4.2 Fill Material

Based on the soil encountered in our borings, except for organic laden soil, the on-site soil appears suitable for use as general engineered fill if it is free of deleterious matter. Maximum particle size for fill material should be limited to 3 inches, with at least 90 percent by weight passing the 1-inch sieve. Proper granular bedding and shading should be used beneath and around new utilities (if applicable). Where imported “non-expansive” fill is required, it is recommended that it be granular in nature, adhere to the above gradation recommendations and conform to the following minimum criteria:

TABLE 8
IMPORTED “NON-EXPANSIVE” FILL CRITERIA

Plasticity Index	12 or less
Liquid Limit	Less than 30%
% Passing #200 Sieve	8% – 40% (general fill) Less than 8% (bedding and shading)

Highly pervious materials such as pea gravel or clean sands are not recommended for use as general fill because they permit transmission of water to the underlying soils. Imported fill material should not be any more corrosive than the on-site soils and should not be classified as being more corrosive than "moderately corrosive." Prior to transporting proposed import materials to the Site, the contractor should make representative samples of the material available to BSK at least 10 working days in advance to allow us enough time to confirm the material meets the above requirements. All on-site or import fill material should be compacted to the recommendations provided for engineered fill in Appendix E.

Due to the moderately to highly expansive soil content within the surficial on-site soils, proper moisture conditioning is important. The moisture conditioning should be performed in accordance with Appendix E. Where ”non expansive” material or aggregate base in paved areas is used, it should immediately be placed over the prepared subgrade to avoid drying of the subgrade. The subgrade for exterior concrete flatwork should be conditioned to the required moisture content prior to their construction and may require additional conditioning if allowed to dry.

Controlled Low Strength Material (CLSM) typically consists of a mixture of cement, fly ash, coarse and fine aggregate, an air entrainment admixture, and water. CLSM should have a 28-day compressive strength of at least 50 pounds per square inch (psi) tested in conformance with ASTM D4832 and sampled in accordance with ASTM D5971. For future excavatability of the CLSM, its 28-day compressive strength should not exceed 1,000 psi. A minimum of one set of cylinders should be cast each day CLSM is placed. One flowability test should be conducted per ASTM D6103 each day CLSM is placed and should be at least 8 inches diameter prior to placement.

6.4.3 Excavation and Backfill

We anticipate that excavation for the foundations and utility trenches can be made with either a backhoe or trencher, or similar earthwork equipment. Where trenches or other excavations are extended deeper than 5 feet, the excavation may become unstable and should be evaluated to monitor stability prior to personnel entering the trenches. Shoring or sloping of any trench wall may be necessary to protect personnel and to provide stability. All trenches should conform to the current OSHA requirements for work safety. It is the contractor's responsibility to follow OSHA temporary excavation guidelines and grade the slopes with adequate layback or provide adequate shoring and underpinning of existing structures and improvements, as needed. Slope layback and/or shoring measures should be adjusted as necessary in the field to suit the actual conditions encountered, in order to protect personnel and equipment within excavations.

Care should be taken during construction to reduce the impact of trenching on adjacent structures and pavements (if applicable). Excavations should be located so that no structures, foundations, and slabs, existing or new, are located above a plane projected 2:1 (horizontal to vertical) upward from any point in an excavation, regardless of whether it is shored or unshored, unless the adjacent surcharge loads are accounted for in the shoring design.

At the time of this geotechnical investigation, free groundwater was observed in some of our borings at depths of approximately 29 and 31 feet BGS; however, high groundwater has been recorded at about 17 feet BGS. The actual depth at which groundwater may be encountered in trenches and excavations may vary. As a minimum, provisions should be made to ensure that conventional sump pumps used in typical trenching and excavation projects are available during construction in case groundwater is found to be higher than observed during our investigation, and/or if substantial runoff water accumulates within the excavations as a result of wet weather conditions or if groundwater is encountered in the excavations.

Backfill for trenches and other small excavations beneath slabs should be compacted as noted in Appendix E. Special care should be taken in the control of utility trench backfilling under structures and flatwork/slab areas. Poor compaction may cause excessive settlements resulting in damage to overlying structures and slabs.

Where utility trenches extend from the exterior into the interior limits of a building, CLSM should be used as backfill material for a distance of 2 feet laterally on each side of the perimeter footing centerline to reduce the potential for the trench to act as a conduit to exterior surface water. In addition, where utilities cross through exterior footings, flexible waterproof caulking should be provided between the sleeve and the pipe. Utility trenches located in landscaped areas should be capped with a minimum of 12 inches of compacted on-site clayey soils.

6.5 Site Drainage

Proper site drainage is important for the long-term performance of the planned structure. The Site should be graded so as to carry surface water away from the building foundations at a minimum of 2 percent in paved areas and 5 percent in landscaped areas to a minimum of 10 feet laterally from the buildings, as required by the 2022 CBC. In addition, all roof gutters should be connected directly into the storm drainage system or drain onto impervious surfaces provided that a safety hazard is not created. Water should not be allowed to pond anywhere on-site.

6.6 Corrosivity Results

Soil samples were collected during our subsurface investigation at depths of approximately 3 to 2 feet BGS at boring B-3 and B-5, respectively, and were submitted for corrosion testing. The samples were tested by CERCO Analytical, a State-certified laboratory in Concord, California, for pH, resistivity, chloride content, and sulfate content in accordance with Caltrans test methods. The test results are presented at the end of Appendix B. Also included is the evaluation by CERCO Analytical of the corrosion test results.

Based upon the resistivity measurement, the sample tested is classified as "corrosive" by CERCO Analytical. The sulfate concentrations measured ranged between 18 and 27 mg/kg (ppm) in the samples. These results are indicative of an exposure category S0 per Table 19.3.1.1 of ACI 318-19. For an S0 exposure class, Table 19.3.2.1 indicates that the minimum f'c of the concrete is 2,500 psi. CERCO Analytical recommends that all buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron be properly protected against corrosion depending upon the critical nature of the structure. They also recommend that all buried metallic pressure piping, such as ductile iron firewater pipelines, should be protected against corrosion. Because we are not corrosion specialists, we recommend that a corrosion specialist be consulted for advice on proper corrosion protection for improvements which will be in contact with the soils and other design details.

The above are general discussions. A more detailed investigation may include more or fewer concerns and should be directed by a corrosion expert. BSK does not practice corrosion engineering. Consideration should also be given to soils in contact with concrete that will be imported to the Site during construction, such as topsoil and landscaping materials, which typically contain fertilizers and other chemicals that can be highly corrosive to metals and concrete. Any imported soil or landscaping materials should not be any more corrosive than the on-site soils and should not be classified as being more corrosive than "moderately corrosive." Also, on-site cutting and filling may result in soils contacting concrete that were not anticipated at the time of this investigation.

6.7 Plan Review and Construction Observation

We recommend that BSK be retained by the Client to review the final foundation and grading plans and specifications before they go out to bid. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstandings prior to the start of construction.

Variations in soil types and conditions are possible and may be encountered during construction. To permit correlation between the soil data obtained during this investigation and the actual soil conditions encountered during construction, we recommend that BSK be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in our investigation and to provide supplemental recommendations if warranted by the exposed conditions. Earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by BSK during construction. BSK should be notified at least two weeks prior to the start of construction and prior to when observation and testing services are needed.

7. ADDITIONAL SERVICES AND LIMITATIONS

7.1 Additional Services

The review of plans and specifications, and field observation and testing during construction by BSK are an integral part of the conclusions and recommendations made in this report. If BSK is not retained for these services, the client will be assuming BSK's responsibility for any potential claims that may arise during or after construction due to the misinterpretation of the recommendations presented herein. The recommended tests, observations, and consultation by BSK during construction include, but are not limited to:

- review of plans and specifications;
- observations of site grading, including stripping and engineered fill construction;
- observation of foundation excavation; and
- in-place density testing of fills, backfills, and finished subgrades.

7.2 Limitations

The recommendations contained in this report are based on our field observations and subsurface explorations, laboratory tests, and our present knowledge of the proposed construction. It is possible that subsurface conditions could vary between or beyond the points explored. If subsurface conditions are encountered during construction that differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction, including the proposed loads or structural locations, changes from that described in this report, our recommendations should also be reviewed.

We prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty, either express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by BSK during the construction phase in order to evaluate compliance with our recommendations. Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the author of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference", as that latter term is used relative to contracts or other matters of law.

This report may be used only by the Client and only for the purposes stated within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report, or if conditions at the Site have changed. If this report is used beyond this period, BSK should be contacted to evaluate whether site conditions have changed since the report was issued.

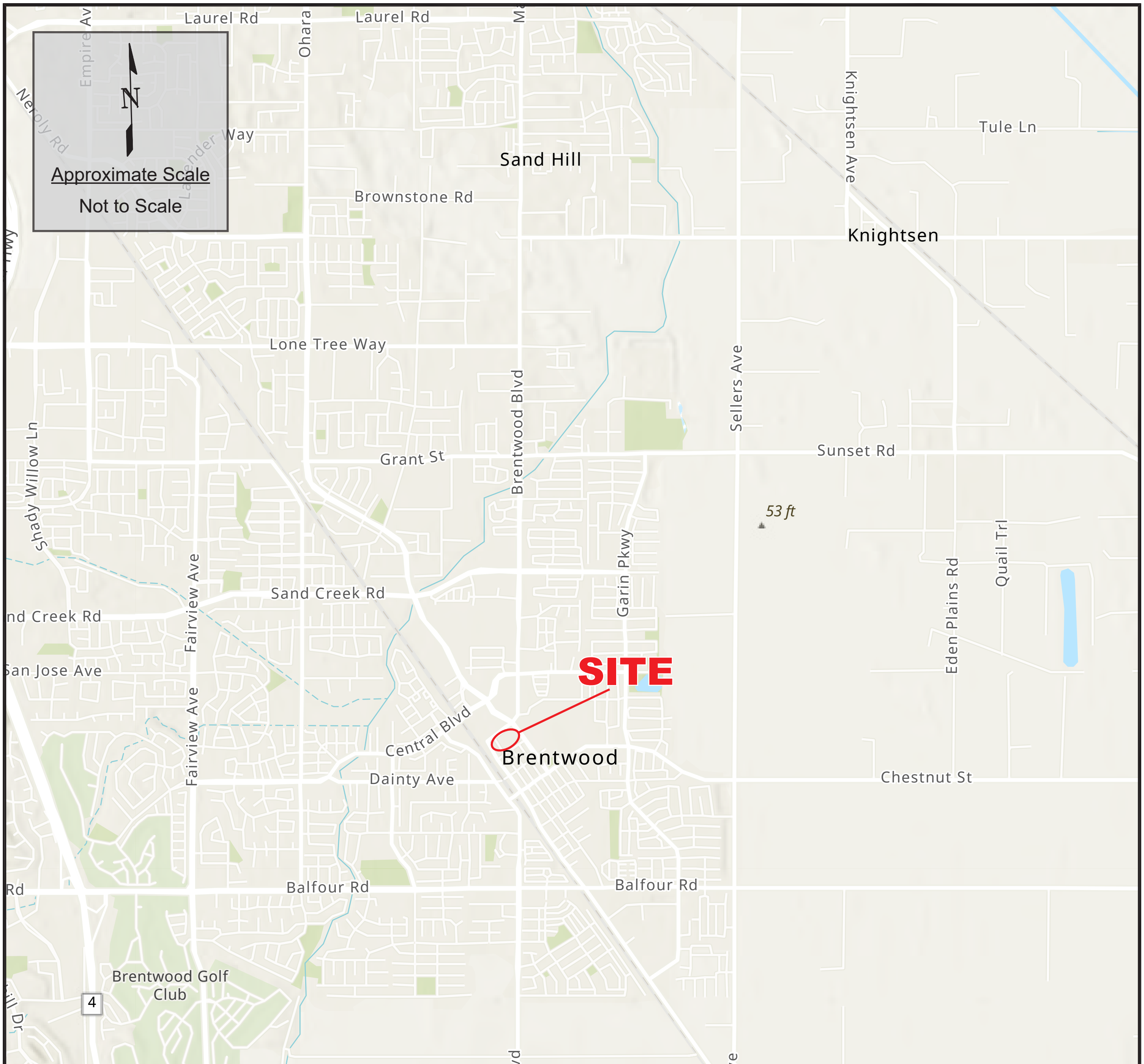
Also, land or facility use, on and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, BSK may recommend that additional work be performed and that an updated report be issued.

The scope of services for this subsurface investigation and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the air, soil, surface water, or groundwater at this Site.

BSK relied on our subsurface exploration and provided recommendations for this project. We understand that BSK will be given the opportunity to perform a formal geotechnical review of the final project plans and specifications. In the event BSK is not retained to review the final project plans and specifications to evaluate if our recommendations have been properly interpreted, we will assume no responsibility for misinterpretation of our recommendations.

We recommend that all earthwork during construction be monitored by a representative from BSK, including site preparation, foundation excavation, placement of engineered fill, and trench backfill. The purpose of these services would be to provide BSK the opportunity to observe the actual soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

FIGURES



References: 1. https://www.arcgis.com/apps/mapviewer/index.html, 2022

Note: Locations are approximate

BSK ASSOCIATES

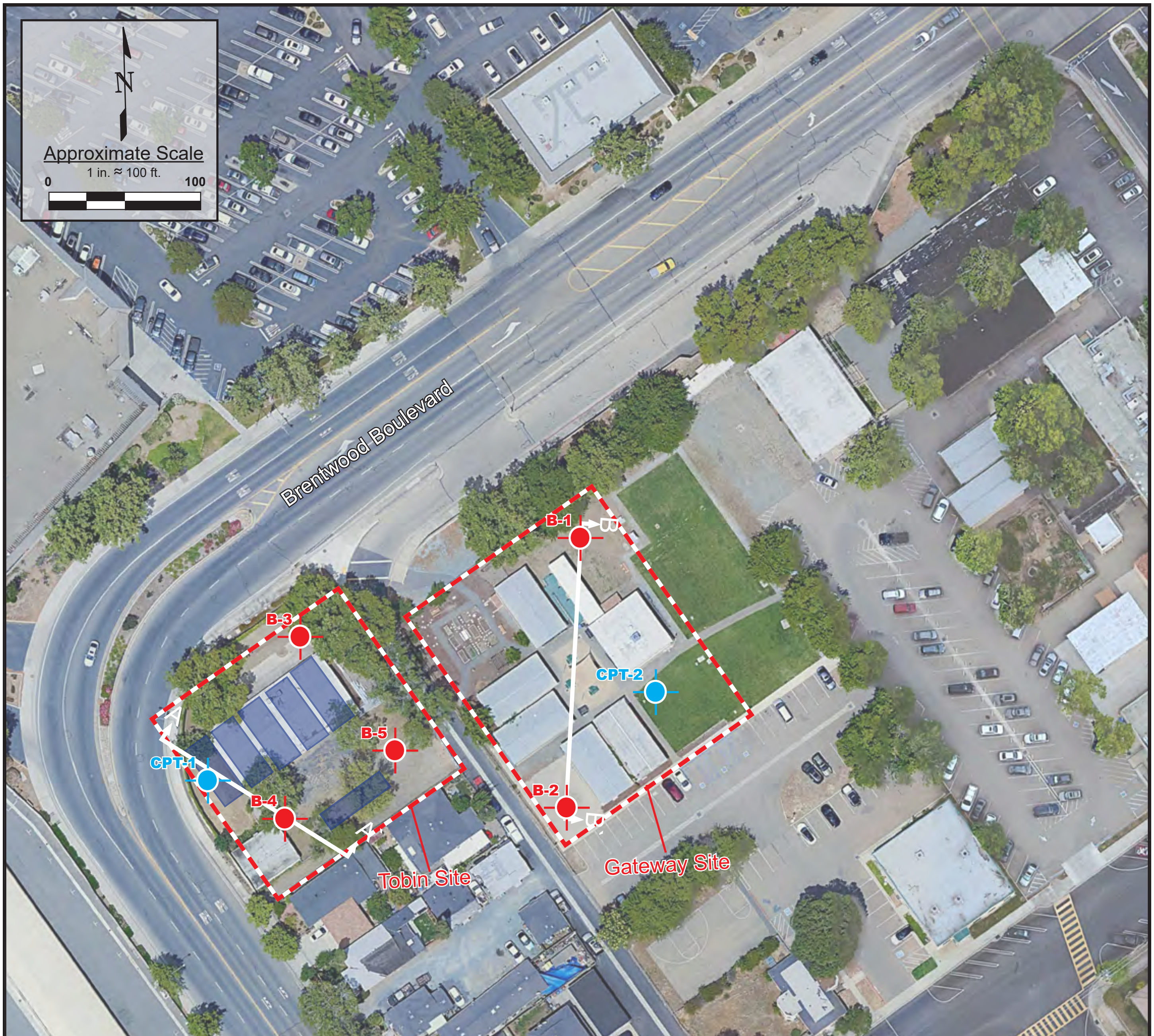
PROJECT NO.	G00000614
DRAWN:	07/18/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Foulk
FILE NAME:	Figures.indd

VICINITY MAP

Liberty Adult Community Education School
Gateway and Tobin Sites
Brentwood, California

FIGURE

1



References: 1. http://earth.google.com, 2023
Notes: Locations are approximate

Legend

B-1 Approximate Boring Location (BSK, 2023)

CPT-1 Approximate CPT Location (BSK, 2023)

Planned Portable Buildings

A A` Cross Section Lines (See Figure C-4)

BSK ASSOCIATES

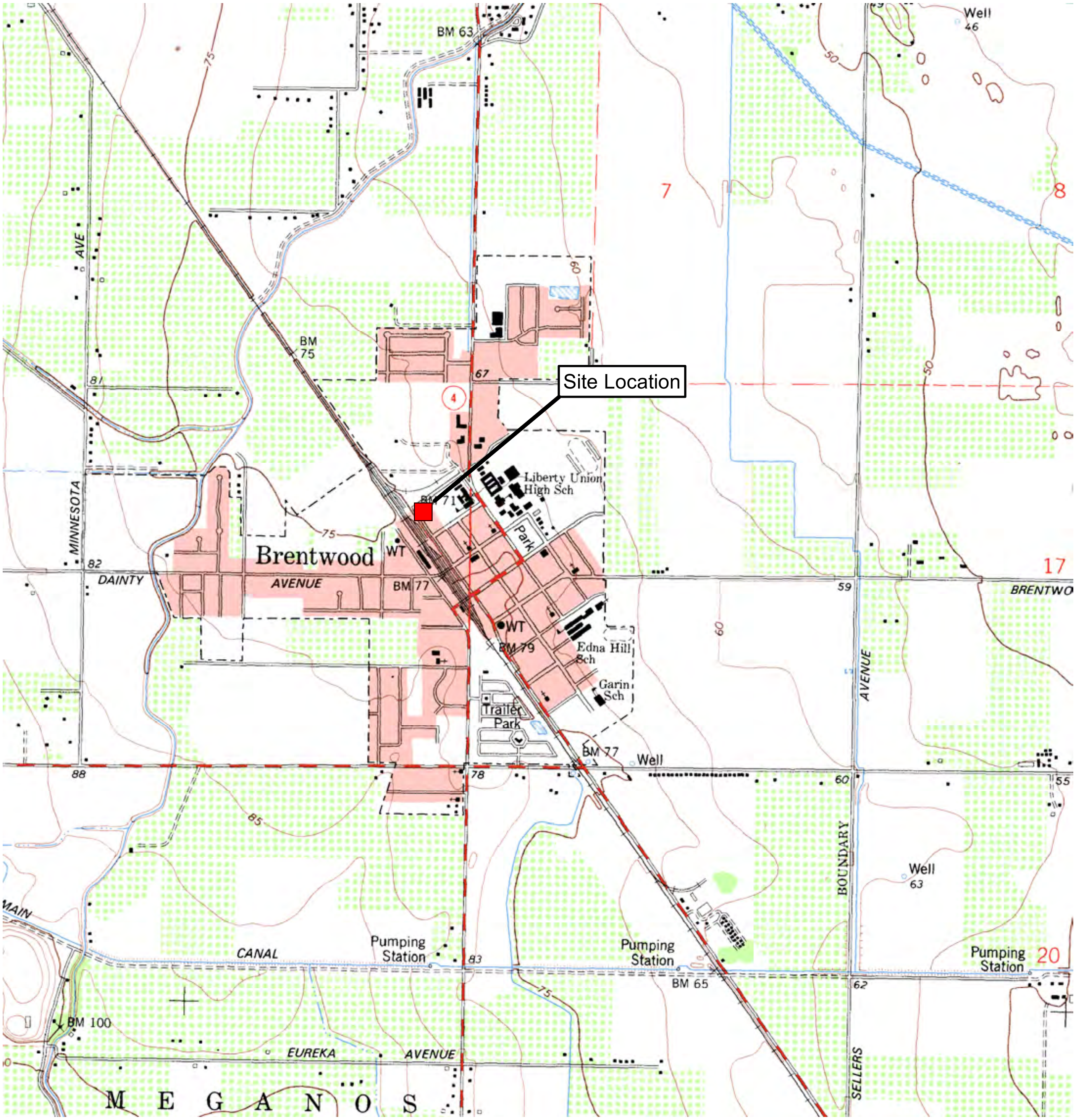
PROJECT NO.	G00000614
DRAWN:	07/10/23
DRAWN BY:	M. Romero
CHECKED BY:	REJ
FILE NAME:	Figures D-2 and D-6.indd

SITE PLAN

Liberty Adult Comunity Education School
Gateway and Tobin Sites
Brentwood, California

FIGURE

2



Reference: http://www.atlas.ca.gov/download.html#/casil/imageryBaseMaps
LandCover/baseMaps/drg/7.5_minute_series_albers_nad83_trimmed

N
W E
S

0 1,000 2,000 4,000 Feet

BSK ASSOCIATES

PROJECT NO. G00000614	**AREA TOPOGRAPHIC MAP**	FIGURE
Drawn: 07/11/2023		
DRAWN BY: MBC	Liberty Adult Community Education School Gateway and Tobin Sites Brentwood, California	3
CHECKED BY: REJ		
FILE NAME: Figure D-1 Topomap		

APPENDIX A

BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487/2488)

MAJOR DIVISIONS				*GRAPHIC LOG*		*TYPICAL DESCRIPTIONS*
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH <5% FINES	Cu ≥4 and 1≤Cc≤3		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/or 1>Cc>3		GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5 to 12% FINES	Cu ≥4 and 1≤Cc≤3		GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
					GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/or 1>Cc>3		GP-GM	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
					GP-GC	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH >12% FINES			GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
					GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
					GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤Cc≤3		SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/or 1>Cc>3		SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5 to 12% FINES	Cu ≥6 and 1≤Cc≤3		SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
					SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/or 1>Cc>3		SP-SM	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
					SP-SC	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH >12% FINES			SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
					SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
					SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)				ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY,
					CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
					CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
					OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)				MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
					CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
					OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487/2488)

FIGURE

A-1

SOIL DESCRIPTION KEY

MOISTURE CONTENT

DESCRIPTION	ABBR	FIELD TEST
Dry	D	Absence of moisture, dusty, dry to the touch
Moist	M	Damp but no visible water
Wet	W	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

PLASTICITY

DESCRIPTION	ABBR	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm) thread cannot be rolled at any water content.
Low (L)	LP	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	MP	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit
High (H)	HP	It takes considerable time rolling and kneeding to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit

STRUCTURE

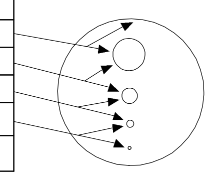
DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4 in. thick, note thickness
Laminated	Alternating layers of varying material or color with the layer less than 1/4 in. thick, note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	ABBR	FIELD TEST
Very Soft	VS	Thumb will penetrate soil more than 1 in. (25 mm)
Soft	S	Thumb will penetrate soil about 1 in. (25 mm)
Firm	F	Thumb will indent soil about 1/4 in. (6 mm)
Hard	H	Thumb wil not indent soil but readily indented with thumbnail
Very Hard	VH	Thumbnail will not indent soil

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12"	>12"	Larger than basketball-sized
Cobbles		3 - 12'	3 - 12"	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3"	3/4 -3"	Thumb-sized to fist-sized
	fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19"	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock salt-sized
	fine	#200 - #10	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029	Flour-sized and smaller

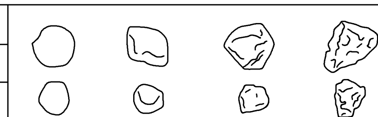


REACTION WITH HCL

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

ANGULARITY

DESCRIPTION	ABBR	CRITERIA
Angular	A	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	SA	Particles are similar to angular description but have rounded edges
Subrounded	SR	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	R	Particles have smoothly curved sides and no edges



APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	ABBR	SPT (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
Very Loose	VL	<4	<4	<5	0 - 15	Easily penetrated with 1/2-inch reinforcing rod by hand
Loose	L	4 - 10	5 - 12	5 - 15	15 - 35	Difficult to penetrate with 1/2-inch reinforcing rod pushed by hand
Medium Dense	MD	10 - 30	12- 35	15 - 40	35 - 65	Easily penetrated a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer
Dense	D	30 - 50	35 - 60	40 - 70	65 - 85	Difficult to penetrate a foot with 1/2-inch reinforcing rod driven with 5-lb. hammer
Very Dense	VD	>50	>60	>70	85 - 100	Penetrated only a few inches with 1/2-inch reinforcing rod driven with 5-lb. hammer

BSK®

SOIL DESCRIPTION KEY

FIGURE

A-2

LOG SYMBOLS

	BULK / BAG SAMPLE
	SPLIT BARREL SAMPLER (2-1/2 inch outside diameter)
	SPLIT BARREL SAMPLER (3 inch outside diameter)
	STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)
	CONTINUOUS CORE
	SHELBY TUBE
	ROCK CORE
	GROUNDWATER LEVEL (encountered at time of drilling)
	GROUNDWATER LEVEL (measured after drilling)
	SEEPAGE

-4	PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)
-200	PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)
LL	LIQUID LIMIT (ASTM Test Method D 4318)
PI	PLASTICITY INDEX (ASTM Test Method D 4318)
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (EM 1110-1-1906)/ASTM Test Method D 2850
EI	EXPANSION INDEX (UBC STANDARD 18-2)
COL	COLLAPSE POTENTIAL
UC	UNCONFINED COMPRESSION (ASTM Test Method D 2166)
MC	MOISTURE CONTENT (ASTM Test Method D 2216)

GENERAL NOTES

Boring log data represents a data snapshot.

This data represents subsurface characteristics only to the extent encountered at the location of the boring.

The data inherently cannot accurately predict the entire subsurface conditions to be encountered at the project site relative to construction or other subsurface activities.

Lines between soil layers and/or rock units are approximate and may be gradual transitions.

The information provided should be used only for the purposes intended as described in the accompanying documents.

In general, Unified Soil Classification System designations presented on the logs were evaluated by visual methods.

Where laboratory tests were performed, the designations reflect the laboratory test results.

The Responsible Geotechnical Engineer, Professional Engineer, or Professional Geologist uses professional judgement and visual-manual procedures in general conformance with ASTM D2488 to classify soil when the full classification suite of tests per ASTM D2487 is not conducted.

LOG KEY

FIGURE

A-3

LOG OF BORING NO. B-1

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

Project Name: **Liberty Adult Community Education Center**
Project Number: **G00000614**
Project Location: **929 2nd St, Brentwood, CA 94513**
Logged by: **K. Yang**
Checked by: **M. Romero**

Surface El.: **75 feet**
Location: **Approximately: 37.9355828, -121.6977265**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetro-meter, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0	**Sandy Lean CLAY (CL):** brown, moist, soft, low to medium plasticity, fine sand									
	slightly porous TXUU (see figure B-2) c= 1,440 psf	1A 1B 1C	5 4 3	2.5 1.75		98	21	38	19	19
5	light brown	2A 2B 2C	5 8 12	1.5		111	24			
10	firm, medium plasticity	3A 3B 3C	6 8 10	1.5		97	26			
14	**Lean CLAY (CL):** light yellowish brown, moist, hard, medium plasticity, trace fine sand									
15		4A 4B 4C	14 28 38	4.5						
19	**Sandy Lean CLAY (CL):** olive yellow, moist, firm to hard, medium plasticity, fine sand									
20		5A 5B 5C	12 14 24	3.5 4.5						

Completion Depth: 40.0
Date Started: 6/12/23
Date Completed: 6/12/23
California Sampler: 2.5-in inner diameter
SPT Sampler:

Drilling Equipment: Exploration GeoServices Mobil B-53R
Drilling Method: Hollow Stem Auger
Drive Weight: 140 lbs
Hole Diameter: 8-in
Drop: 30-in
Remarks: Semi-Automatic Hammer

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

LOG OF BORING NO. B-1

Project Name: **Liberty Adult Community Education Center**
Project Number: **G00000614**
Project Location: **929 2nd St, Brentwood, CA 94513**
Logged by: **K. Yang**
Checked by: **M. Romero**

Surface El.: **75 feet**
Location: **Approximately: 37.9355828, -121.6977265**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
	Sandy Lean CLAY (CL): olive yellow, moist, firm to hard, medium plasticity, fine sand *(continued)*									
25	soft, increased sand content, calcium carbonate	6A	14	1.5						
	Lean CLAY (CL): light yellowish brown, moist, firm, medium plasticity, homogeneous, calcium carbonate veins	6B 6C	26 37	3.25						
30	**Sandy Lean CLAY (CL):** light yellowish brown, wet, soft, medium plasticity, fine sand ▽	7A 7B 7C	15 16 19	1.5	69			33	18	15
35	hard, manganese oxide staining	8A 8B 8C	12 36 29	4.25 4.5						
	very moist, hard, seam of silty sand present	9A 9B 9C	12 18 33	2.25 4.25						
40	Boring terminated at approximately 40 feet. Free groundwater was observed at 31 feet. Boring was backfilled with cement grout.									
45										

Completion Depth: 40.0
Date Started: 6/12/23
Date Completed: 6/12/23
California Sampler: 2.5-in inner diameter
SPT Sampler:

Drilling Equipment: Exploration GeoServices Mobil B-53R
Drilling Method: Hollow Stem Auger
Drive Weight: 140 lbs
Hole Diameter: 8-in
Drop: 30-in
Remarks: Semi-Automatic Hammer

BSK ASSOCIATES
BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

LOG OF BORING NO. B-2

Project Name: **Liberty Adult Community Education Center**
Project Number: **G00000614**
Project Location: **929 2nd St, Brentwood, CA 94513**
Logged by: **K. Yang**
Checked by: **M. Romero**

Surface El.: **76 feet**
Location: **Approximately: 37.9350856, -121.6977869**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / 6 inches	Pocket Penetro-meter, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		FILL: approximately 4 inches of possible aggregate base										
		Sandy Lean CLAY (CL): dark brown, moist, soft, medium plasticity, fine sand										
				1A 1B 1C	5 5 6	2.0		98	21			
5		TXUU (see figure B-2) c= 960 psf		2A 2B 2C	3 5 6	1.5		95	25			
10		soft to firm, increased sand content slightly porous		3A 3B 3C	7 8 7	2.5 1.5		97	21			
15		very moist, soft		4A 4B 4C	9 6 10	1.75 1.5						
		Lean CLAY (CL): light yellowish brown, moist, firm to hard, medium plasticity										
20				5A 5B 5C	12 19 37	4.0 2.5						
		Boring terminated at approximately 21.5 feet. No free groundwater was observed. Boring was backfilled with cement grout.										

Completion Depth: 21.5
Date Started: 6/12/23
Date Completed: 6/12/23
California Sampler: 2.5-in inner diameter
SPT Sampler:

Drilling Equipment: Exploration GeoServices Mobil B-53R
Drilling Method: Hollow Stem Auger
Drive Weight: 140 lbs
Hole Diameter: 8-in
Drop: 30-in
Remarks: Semi-Automatic Hammer

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

LOG OF BORING NO. B-3

Project Name: **Liberty Adult Community Education Center**
Project Number: **G00000614**
Project Location: **929 2nd St, Brentwood, CA 94513**
Logged by: **K. Yang**
Checked by: **M. Romero**

Surface El.: **76 feet**
Location: **Approximately: 37.9353612, -121.6984454**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0	**Sandy Lean CLAY (CL):** dark brown, moist, firm to hard, medium plasticity, fine to coarse sand, trace fine gravel (possible fill)									
	TXUU (see figure B-2) c= 2,500 psf	1A 1B 1C	12 9 10	3.5		106	20			
	Lean CLAY with Sand (CL): dark brown, moist, firm to hard, medium plasticity, fine sand									
5		2A 2B 2C	7 10 9	2.5 2.0		96	23			
	Sandy Lean CLAY (CL): light yellowish brown, moist, firm, low to medium plasticity, fine sand									
10		3A 3B 3C	7 9 10	1.5 3.0		108	20			
15	hard, decreased sand content	4A 4B 4C	15 27 39	3.75 4.25						
20	soft, low plasticity, increased sand content	5A 5B 5C	7 6 20	1.5						
	Boring terminated at approximately 21.5 feet. No free groundwater was observed. Boring was backfilled with cement grout.									

Completion Depth: 21.5
Date Started: 6/12/23
Date Completed: 6/12/23
California Sampler: 2.5-in inner diameter
SPT Sampler:

Drilling Equipment: Exploration GeoServices Mobil B-53R
Drilling Method: Hollow Stem Auger
Drive Weight: 140 lbs
Hole Diameter: 8-in
Drop: 30-in
Remarks: Semi-Automatic Hammer

LOG OF BORING NO. B-4

BSK ASSOCIATES
BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

Project Name:	**Liberty Adult Community Education Center**
Project Number:	**G00000614**
Project Location:	**929 2nd St, Brentwood, CA 94513**
Logged by:	**K. Yang**
Checked by:	**M. Romero**

Surface El.: **76 feet**

Location: **Approximately: 37.9351052, -121.6983612**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / 6 inches	Pocket Penetro-meter, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0		**Lean CLAY (CL):** dark brown, moist, hard, medium to high plasticity										
				1A 1B 1C	7 12 15	4.0				43	20	23
5		trace fine sand slightly porous Collapse Potential= 0.05% (see figure B-3)		2A 2B 2C	7 12 13	3.0		101	21			
		Sandy Lean CLAY (CL): light yellowish brown, moist, firm to hard, medium plasticity, fine sand, porous										
10		Collapse Potential= 11.67% (see figure B-4)		3A 3B 3C	7 7 9	4.0 2.0		83	10			
		Clayey SAND (SC): light yellowish brown, moist, medium dense, fine sand										
15				4A 4B 4C	12 14 19	3.0 3.5						
		Sandy Lean CLAY (CL): light yellowish brown, moist, firm, medium plasticity, fine sand										
		Lean CLAY (CL): yellowish brown, moist, medium plasticity, hard, trace fine sand										
20				5A 5B 5C	12 20 32	4.5						

Completion Depth:	40.0	**Drilling Equipment:**	Exploration GeoServices Mobil B-53R
Date Started:	6/12/23	**Drilling Method:**	Hollow Stem Auger
Date Completed:	6/12/23	**Drive Weight:**	140 lbs
California Sampler:	2.5-in inner diameter	**Hole Diameter:**	8-in
SPT Sampler:		**Drop:**	30-in
		Remarks:	Semi-Automatic Hammer

BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

LOG OF BORING NO. B-4

Project Name: **Liberty Adult Community Education Center**
Project Number: **G00000614**
Project Location: **929 2nd St, Brentwood, CA 94513**
Logged by: **K. Yang**
Checked by: **M. Romero**

Surface El.: **76 feet**
Location: **Approximately: 37.9351052, -121.6983612**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetro-meter, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
	Lean CLAY (CL): yellowish brown, moist, medium plasticity, hard, trace fine sand *(continued)*									
25	manganese oxide staining	6A 6B 6C	11 18 34	4.25 4.5						
29	**Sandy Lean CLAY (CL):** dark yellowish brown, wet, soft to firm, medium plasticity, fine sand, manganese oxide staining									
30		7A 7B 7C	18 22 37	1.5 2.0						
34	**Lean CLAY (CL):** yellowish brown, moist, medium plasticity, manganese oxide staining									
35		8A 8B 8C	8 18 30	4.5						
38	**Lean CLAY with Sand (CL):** dark yellowish brown, moist, soft to firm, medium plasticity, fine sand	9A 9B 9C	7 18 12	2.5 3.0	83			35	16	19
40	Boring terminated at approximately 40 feet. Free groundwater was observed at approximately 29 feet. Boring was backfilled with cement grout.									
45										

Completion Depth: 40.0
Date Started: 6/12/23
Date Completed: 6/12/23
California Sampler: 2.5-in inner diameter
SPT Sampler:

Drilling Equipment: Exploration GeoServices Mobil B-53R
Drilling Method: Hollow Stem Auger
Drive Weight: 140 lbs
Hole Diameter: 8-in
Drop: 30-in
Remarks: Semi-Automatic Hammer

LOG OF BORING NO. B-5

BSK ASSOCIATES
BSK Associates
399 Lindbergh Avenue
Livermore, CA 94551
Telephone: (925) 315-3151

Project Name:	**Liberty Adult Community Education Center**
Project Number:	**G00000614**
Project Location:	**929 2nd St, Brentwood, CA 94513**
Logged by:	**K. Yang**
Checked by:	**M. Romero**

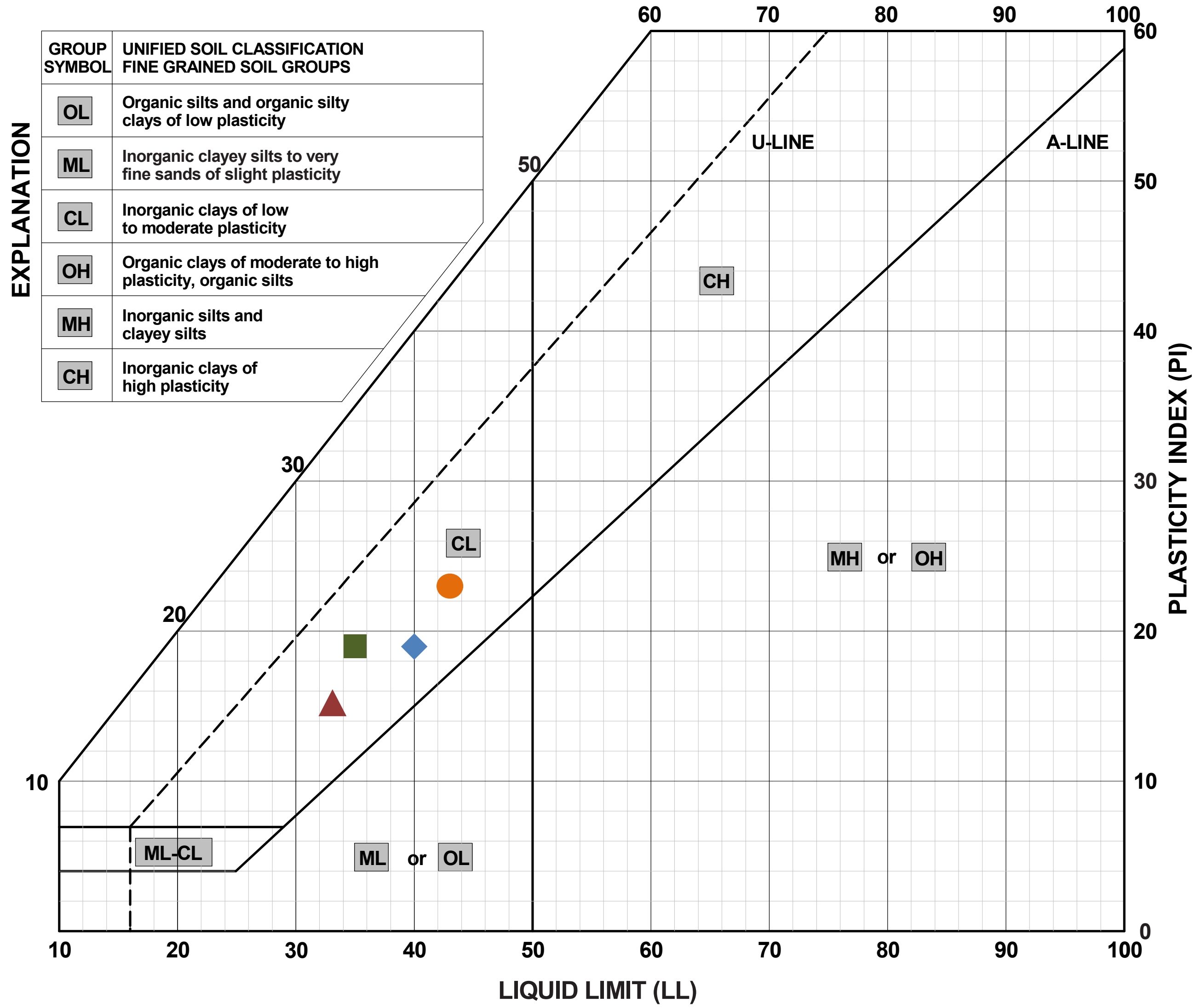
Surface El.: **76 feet**
Location: **Approximately: 37.9351803, -121.6980886**

Depth, feet	MATERIAL DESCRIPTION	Sample Number	Penetration Blows / 6 inches	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
	ASPHALT: approximately 4 inches of asphalt									
	FILL: approximately 5 inches of fill, possibly aggregate base									
	Lean CLAY with Sand (CL): dark brown, moist, firm, medium plasticity, fine sand, roots present									
	TXUU (see figure B-2) c= 2,040 psf	1A 1B 1C	7 8 9	2.0 3.0		101	21			
5		2A 2B 2C	7 8 12			89	22			
	Sandy Lean CLAY (CL): dark yellowish brown, moist, soft to firm, medium plasticity, roots present									
10		3A 3B 3C	6 7 11	1.5		102	21			
	Lean CLAY (CL): dark yellowish brown, moist, hard, medium plasticity, roots present									
15		4A 4B 4C	13 21 34	4.5						
	Boring terminated at approximately 16.5 feet. No free groundwater was observed. Boring was backfilled with cement grout.									
20										

Completion Depth:	16.5	**Drilling Equipment:**	Exploration GeoServices Mobil B-53R
Date Started:	6/12/23	**Drilling Method:**	Hollow Stem Auger
Date Completed:	6/12/23	**Drive Weight:**	140 lbs
California Sampler:	2.5-in inner diameter	**Hole Diameter:**	8-in
SPT Sampler:		**Drop:**	30-in
		Remarks:	Semi-Automatic Hammer

APPENDIX B

LABORATORY TEST RESULTS



LEGEND:	SOURCE	DEPTH (ft)	LL	PL	PI	DESCRIPTION
◆	B-1	2.5	38	19	19	Sandy Lean Clay (CL)
▲	B-1	31.0	33	18	15	Sandy Lean Clay (CL)
●	B-4	3.0	43	20	23	Lean Clay (CL)
■	B-4	39.5	35	16	19	Lean Clay with Sand (CL)

BSK ASSOCIATES

PROJECT NO. G00000614
DRAWN: 07/18/23
DRAWN BY: D. Tower
CHECKED BY: C. Foulk
FILE NAME: Figures.indd

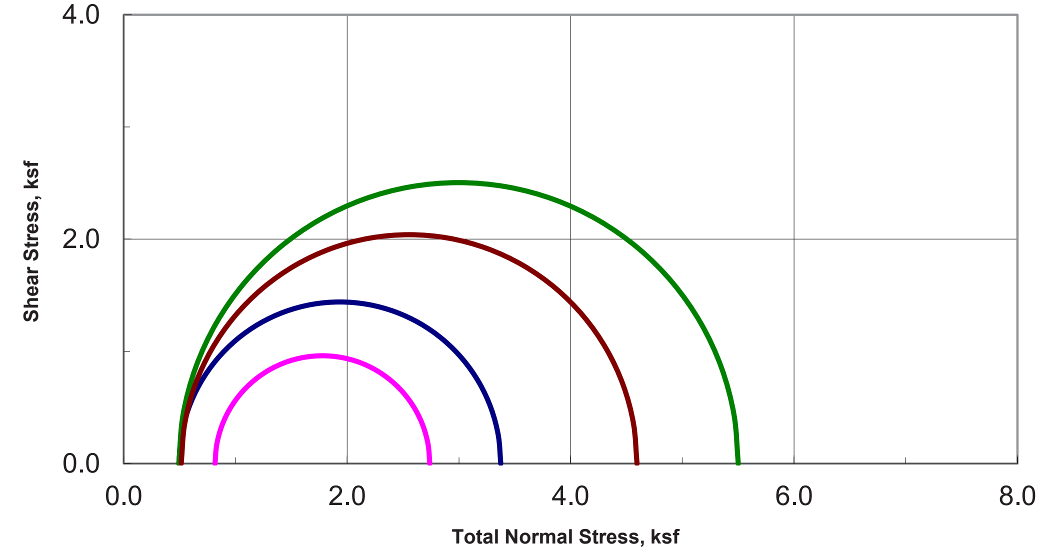
ATTERBERG LIMITS

Liberty Adult Community Education School
Gateway and Tobin Sites
Brentwood, California

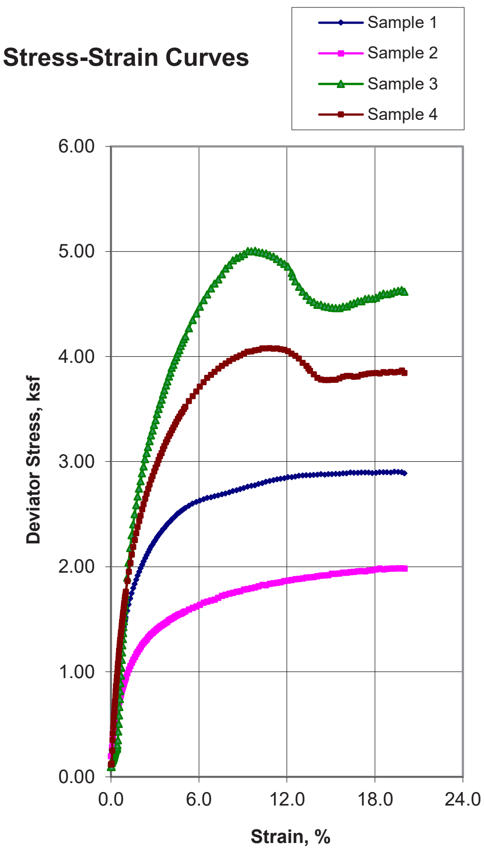
FIGURE
B-1

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test
ASTM D2850



Stress-Strain Curves



Sample Data	1	2	3	4
Moisture %	21.2	25.0	20.1	21.4
Dry Den,pcf	98.3	95.1	105.6	100.8
Void Ratio	0.714	0.772	0.597	0.672
Saturation %	80.3	87.4	91.0	85.9
Height in	5.00	5.02	5.00	5.01
Diameter in	2.40	2.37	2.40	2.40
Cell psi	3.4	5.7	3.4	3.6
Strain %	15.00	15.00	9.83	10.82
Deviator, ksf	2.881	1.922	5.008	4.080
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.050	0.050	0.050	0.050
Job No.:	664-490			
Client:	BSK Associates			
Project:	G0000614			
Boring:	B-1	B-2	B-3	B-5
Sample:	1C	2C	1C	1C
Depth ft:	3	6	3	3

Visual Soil Description

Sample #	
1	Sandy Lean Clay (CL)
2	Sandy Lean Clay (CL)
3	Sandy Lean Clay (CL)
4	Lean Clay with Sand (CL)

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

BSK ASSOCIATES

PROJECT NO. G00000614
DRAWN: 07/18/23
DRAWN BY: D. Tower
CHECKED BY: C. Foulk
FILE NAME: Figures.indd

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Liberty Adult Community Education School
Gateway and Tobin Sites
Brentwood, California

FIGURE
B-2

Collapse Potential of Soils
ASTM D5333-03

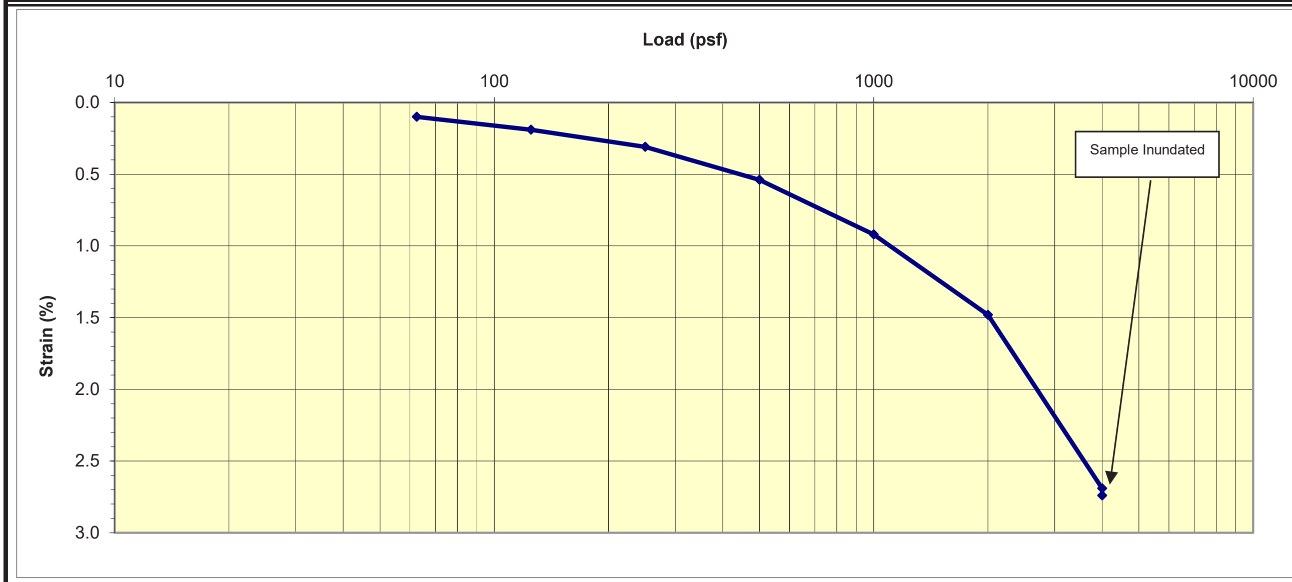
COOPER TESTING LABORATORY

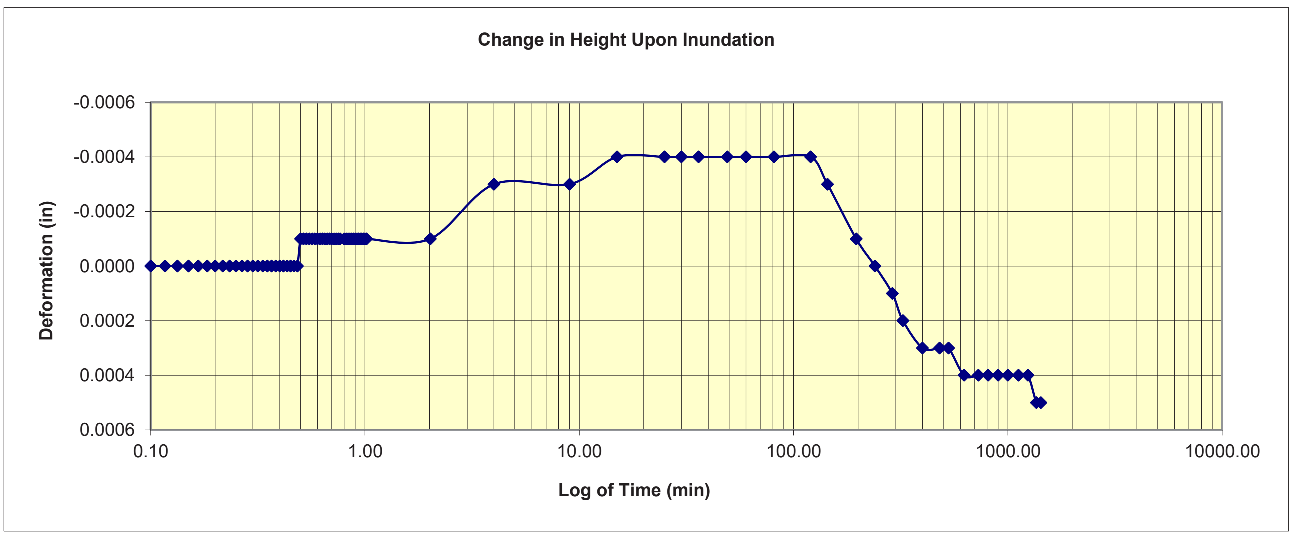
Job No.:	664-490	**Boring:**	B-4	**Date:**	6/28/2023
Client:	BSK Associates	**Sample:**	2C	**Tested By:**	MD
Project:	Gateway and Tobin **Proj. No.:** G0000614	**Depth, ft.:**	6	**Checked:**	PJ/DC
Soil Description:	Lean CLAY (CL)				

Load, psf	62.5	125	250	500	1000	2000	4000	Inundation
Deformation, in.:	0.001	0.0009	0.0012	0.0023	0.0038	0.0056	0.0121	0.0005

	Initial	**Final**
Moisture Content %	21.1	22.4
Dry Density, pcf	100.9	103.7
Void Ratio	0.671	0.625
Saturation %	85.1	97.0
Assumed Specific Gravity	2.70	

Vertical Stress at Inundation (psf)	**Collapse Potential (%)**
4000	**0.05**





BSK ASSOCIATES

PROJECT NO.	**G00000614**
DRAWN:	07/18/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Foulk
FILE NAME:	Figures.indd

COLLAPSE POTENTIAL OF SOILS

Liberty Adult Community Education School
Gateway and Tobin Sites
Brentwood, California

FIGURE

B-3

Collapse Potential of Soils
ASTM D5333-03

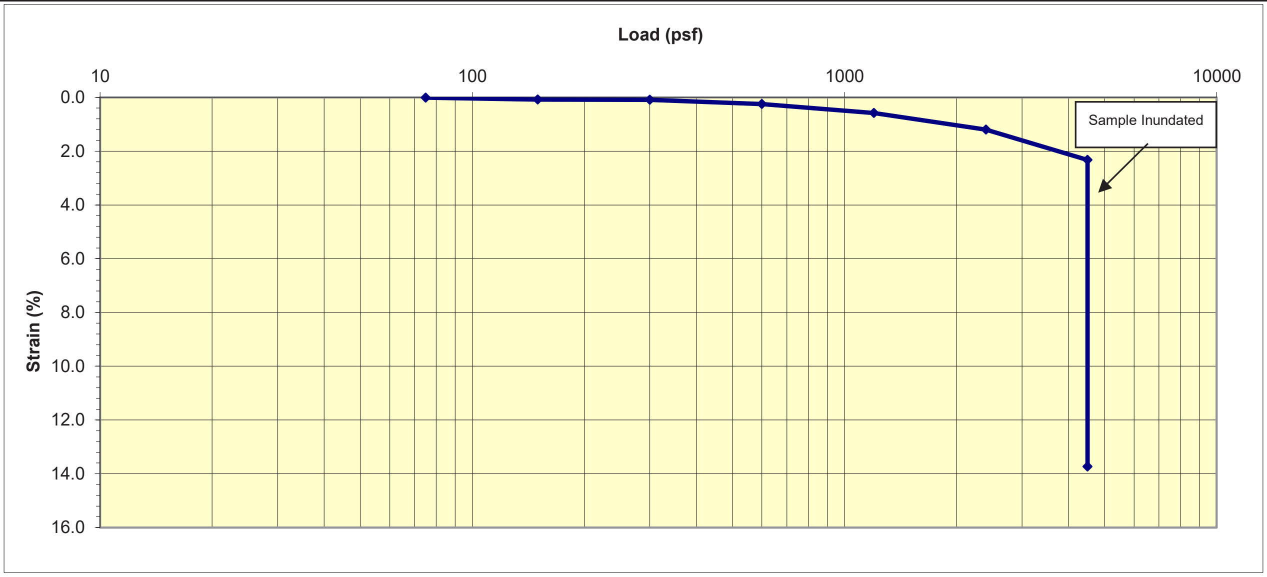
Job No.:	664-490	**Boring:**	B-4	**Date:**	6/28/2023		
Client:	BSK Associates	**Sample:**	3C	**Tested By:**	MD		
Project:	Gateway and Tobin **Proj. No.:** G0000614	**Depth, ft.:**	11	**Checked:**	PJ/DC		

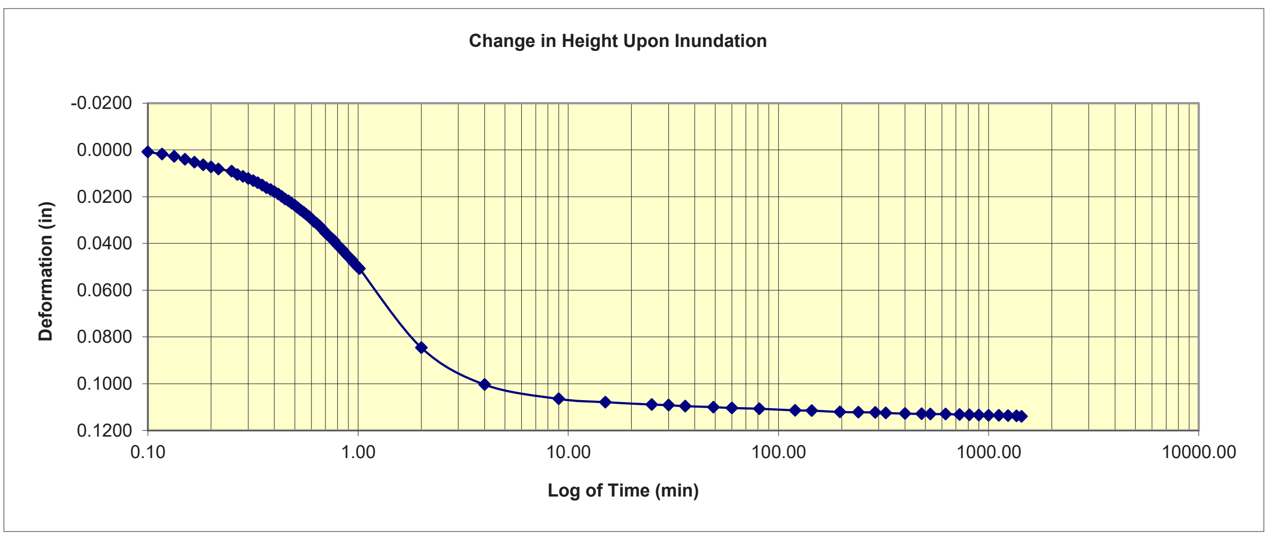
Soil Description: Sandy Lean Clay (CL)

Load, psf	75	150	300	600	1200	2400	4500	Inundation
Deformation, in.:	0.0001	0.0007	0.0001	0.0016	0.0033	0.0062	0.0113	0.114

	Initial	**Final**
Moisture Content %	10.3	24.0
Dry Density, pcf	82.9	96.1
Void Ratio	1.034	0.755
Saturation %	26.9	85.7
Assumed Specific Gravity	2.70	

Vertical Stress at Inundation (psf)	**Collapse Potential (%)**
4500	**11.67**





BSK ASSOCIATES

PROJECT NO.	G00000614
DRAWN:	07/18/23
DRAWN BY:	D. Tower
CHECKED BY:	C. Foulk
FILE NAME:	Figures.indd

COLLAPSE POTENTIAL OF SOILS

Liberty Adult Community Education School
Gateway and Tobin Sites
Brentwood, California

FIGURE

B-4

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

22 June, 2023

Job No. 2306037
Cust. No. 12667

Ms. Danaige Tower
BSK Associates Engineers & Laboratories
399 Lindbergh Avenue
Livermore, CA 94551

Subject: Project No.: G000000614
Project Name: Gateway and Tobin Site
Corrosivity Analysis – ASTM Test Methods

Dear Ms. Tower:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on June 19, 2023. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentrations are 18 mg/kg and 27 mg/kg. Both samples are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils are 7.45 and 8.61, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 270-mV and 280-mV. Both samples are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client:	BSK Associates Engineers & Laboratories
Client's Project No.:	G000000614
Client's Project Name:	Gateway and Tobin Site
Date Sampled:	16-Jun-23
Date Received:	19-Jun-23
Matrix:	Soil
Authorization:	Chain of Custody

Date of Report: 22-Jun-2023

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2306037-001	B-3 @ 3'	270	8.61	-	1,500	-	N.D.	27
2306037-002	B-5 @ 2'	280	7.45	-	1,500	-	N.D.	18

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	20-Jun-2023	20-Jun-2023	-	20-Jun-2023	-	20-Jun-2023	20-Jun-2023

Cheri McMillen
Chemist

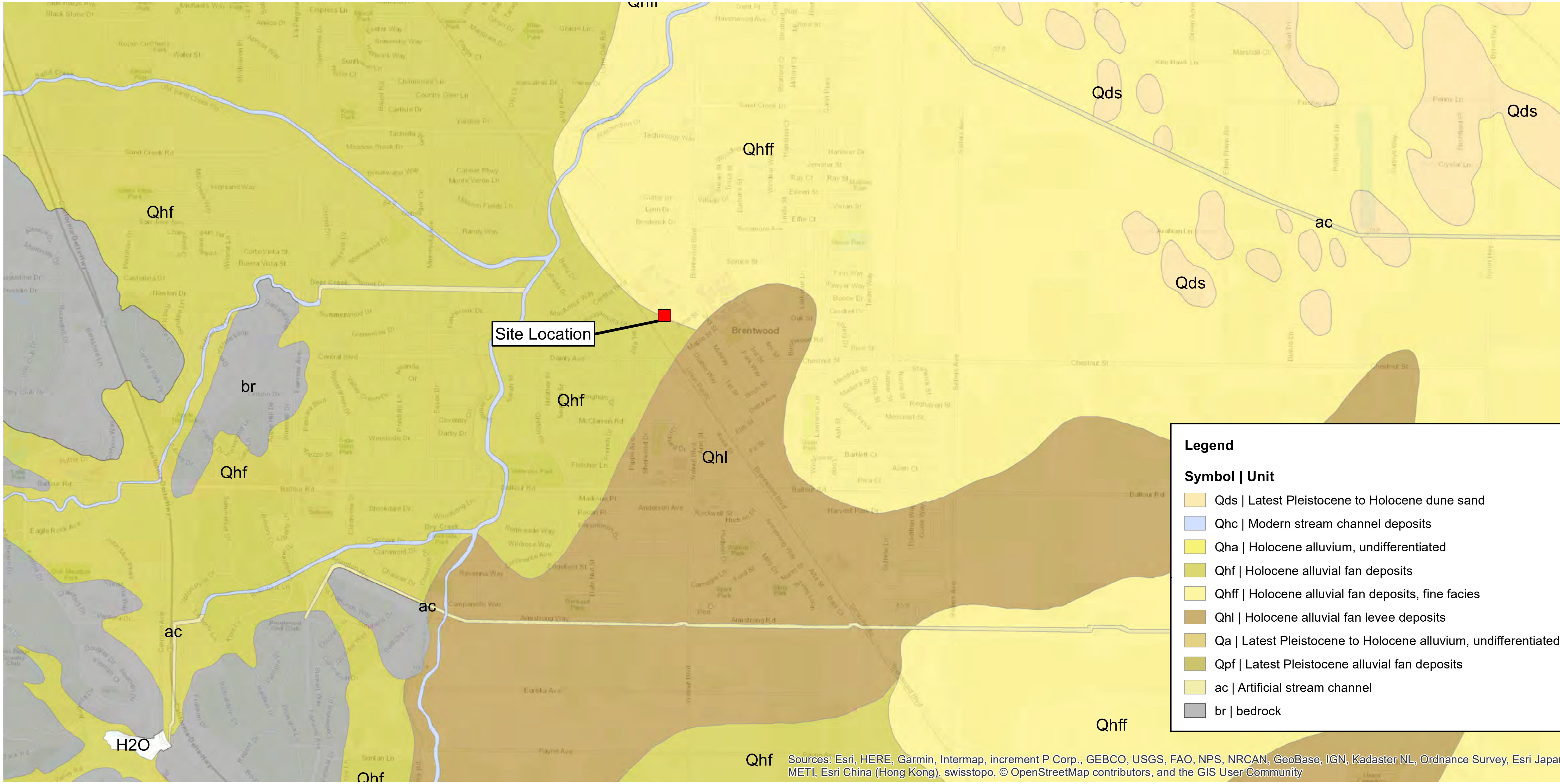
* Results Reported on "As Received" Basis

N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX C

GEOLOGIC AND SEISMIC HAZARD FIGURES



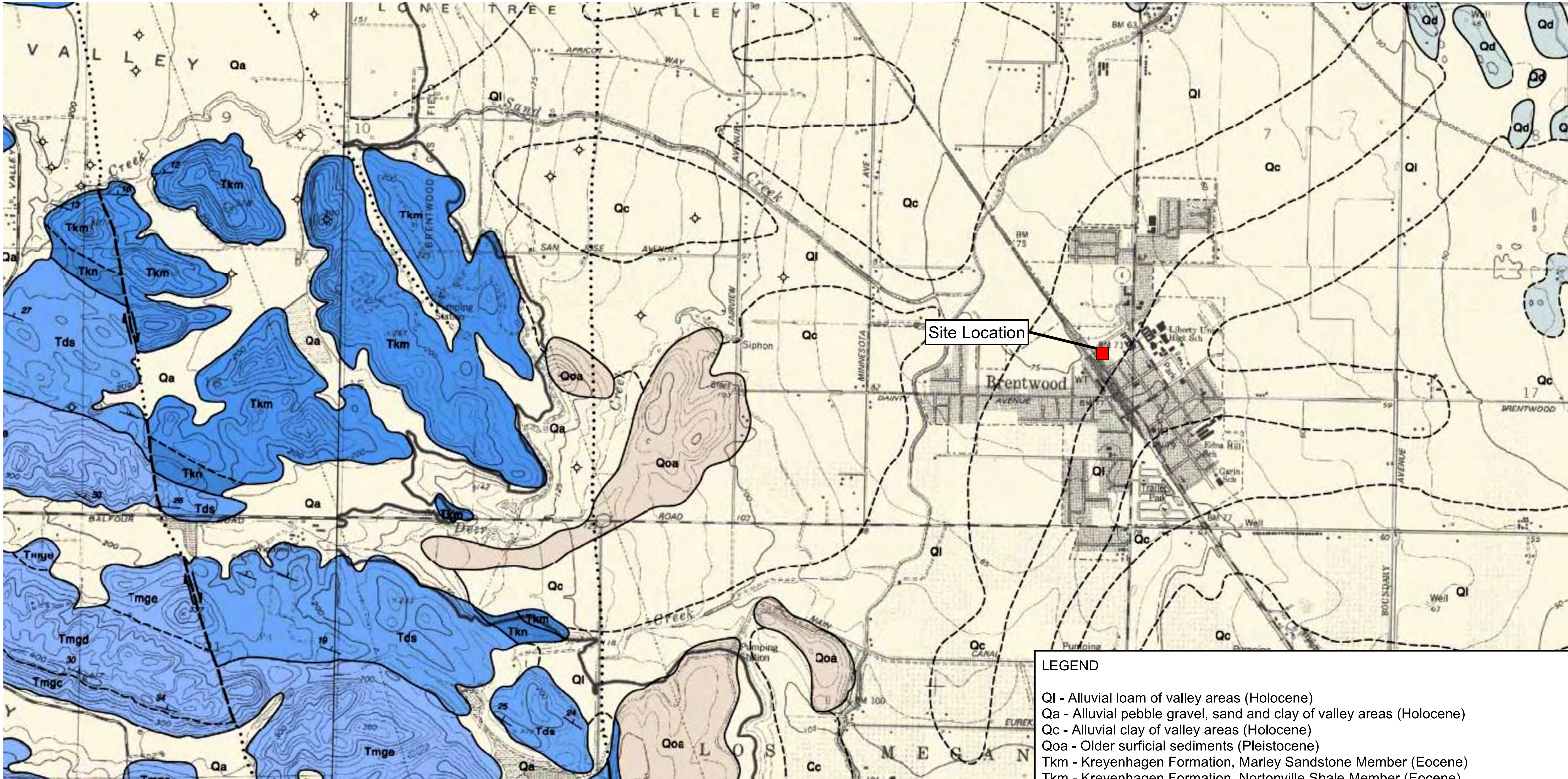
References: Knudsen, K.L., et all., 2000, Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California: A Digital Database, U.S. Geological Survey Open-File Report 00-444

BSK ASSOCIATES

PROJECT NO. G00000614	**QUATERNARY GEOLOGIC MAP**
Drawn: 07/11/2023	
DRAWN BY: MBC	Liberty Adult Community Education School Gateway and Tobin Sites Brentwood, California
CHECKED BY: REJ	
FILE NAME: Figure D-4 Geologic_map	

Figure

C-1



LEGEND

Ql - Alluvial loam of valley areas (Holocene)
Qa - Alluvial pebble gravel, sand and clay of valley areas (Holocene)
Qc - Alluvial clay of valley areas (Holocene)
Qoa - Older surficial sediments (Pleistocene)
Tkm - Kreyenhagen Formation, Marley Sandstone Member (Eocene)
Tkm - Kreyenhagen Formation, Nortonville Shale Member (Eocene)
Tds - Domengine Sandstone (Eocene)
Tmgd - Meganos Formation, Sandstone Member (Eocene)

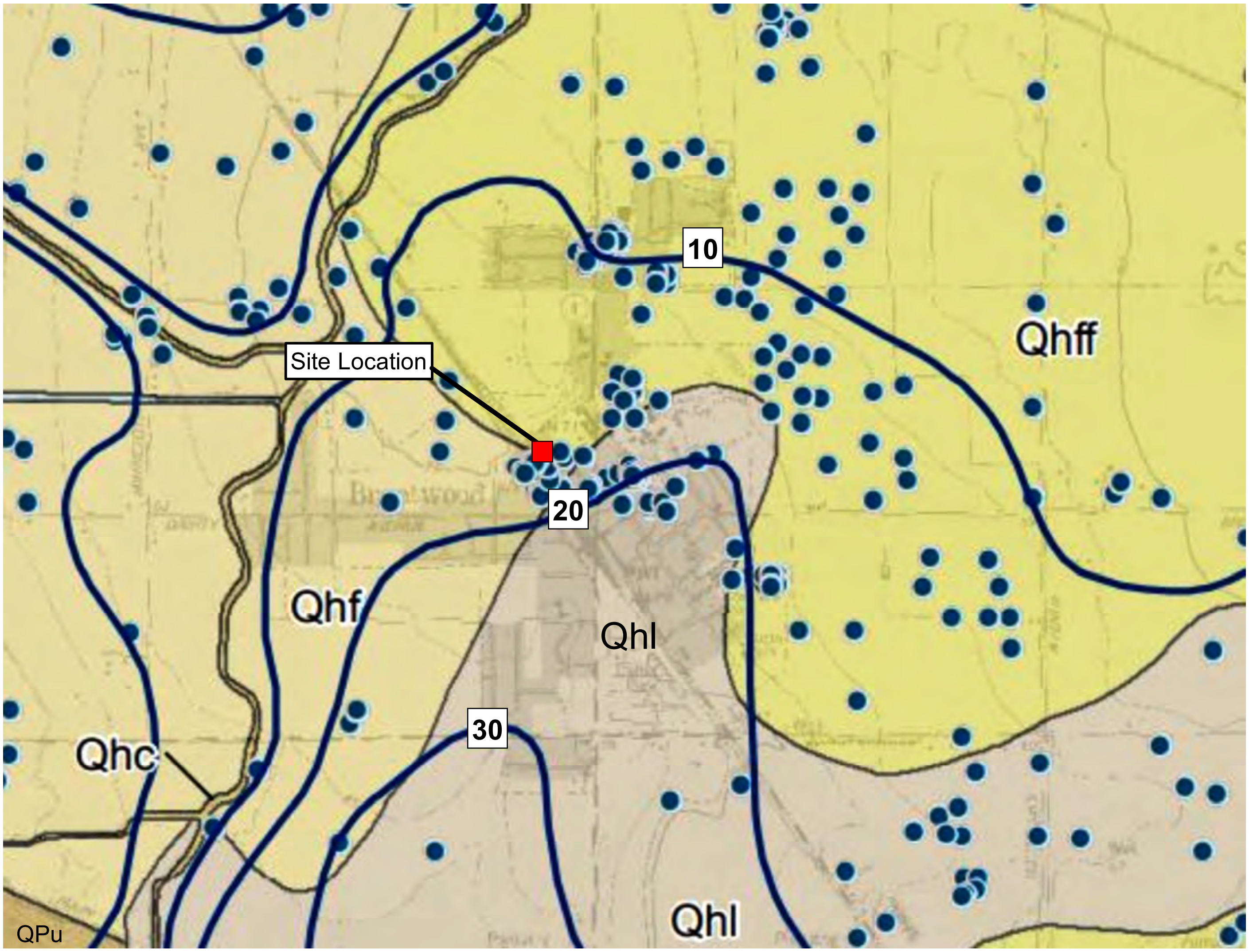
Dibblee, T.W., and Minch, J.A., 2006, Geologic map of the Antioch South & Brentwood quadrangles, Contra Costa County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-193

N
W E
S

Feet
0 1,000 2,000 4,000

BSK ASSOCIATES

PROJECT NO. G00000614	**GEOLOGIC MAP**	Figure
Drawn: 07/11/2023		**C-2**
DRAWN BY: MBC	Liberty Adult Community Education School Gateway and Tobin Sites Brentwood, California	
CHECKED BY: REJ		
FILE NAME: Figure D-5 Geologic_map2		



LEGEND
Qhc - Holocene to Modern Stream Channel Deposits
Qhl - Holocene Alluvial Fan Levee Deposits
Qhf - Holocene Alluvial Fan Deposits
Qhff - Holocene Alluvial Fan Deposits, Fine Facies
QPu - Late to Early Pleistocene Sandstone, Siltstone, and Gravel - Undifferentiated

• Groundwater measurement location
—10— Depth to groundwater (in feet)

Reference: CGS, 2018, Seismic Hazard Zone Report for the Brentwood 7.5-Minute Quadrangle, Contra Costa County, California, Seismic Hazard Zone Report 124

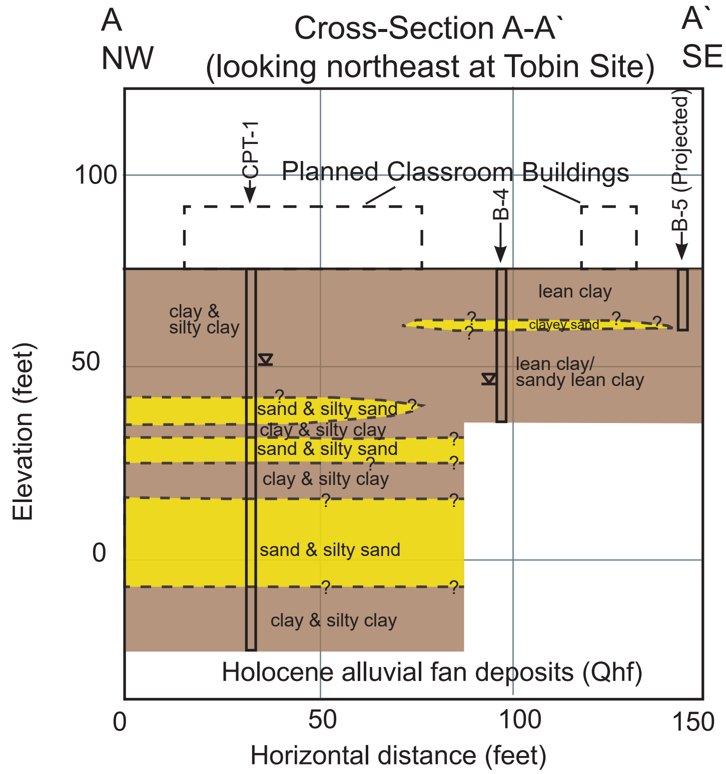
N W E S

0 1,000 2,000 4,000 Feet

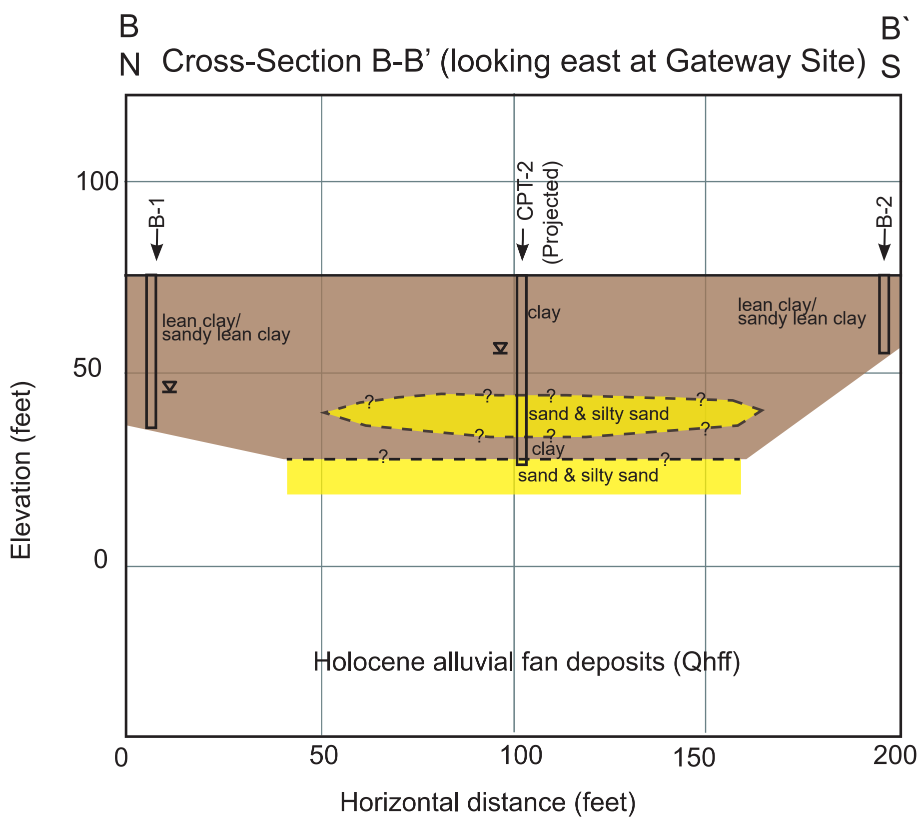
BSK ASSOCIATES

PROJECT NO. G00000614	**HISTORICALLY HIGH GROUNDWATER DEPTH**	Figure
Drawn: 07/11/2023		
DRAWN BY: MBC	Liberty Adult Community Education School Gateway and Tobin Sites Brentwood, California	C-3
CHECKED BY: REJ		
FILE NAME: Figure C-2 Historic GW		

Cross-Section A-A` (looking northeast at Tobin Site)



Cross-Section B-B' (looking east at Gateway Site)



LEGEND

▯ Soil boring or CPT Location

– –?– Unit contact

⊻ Groundwater level

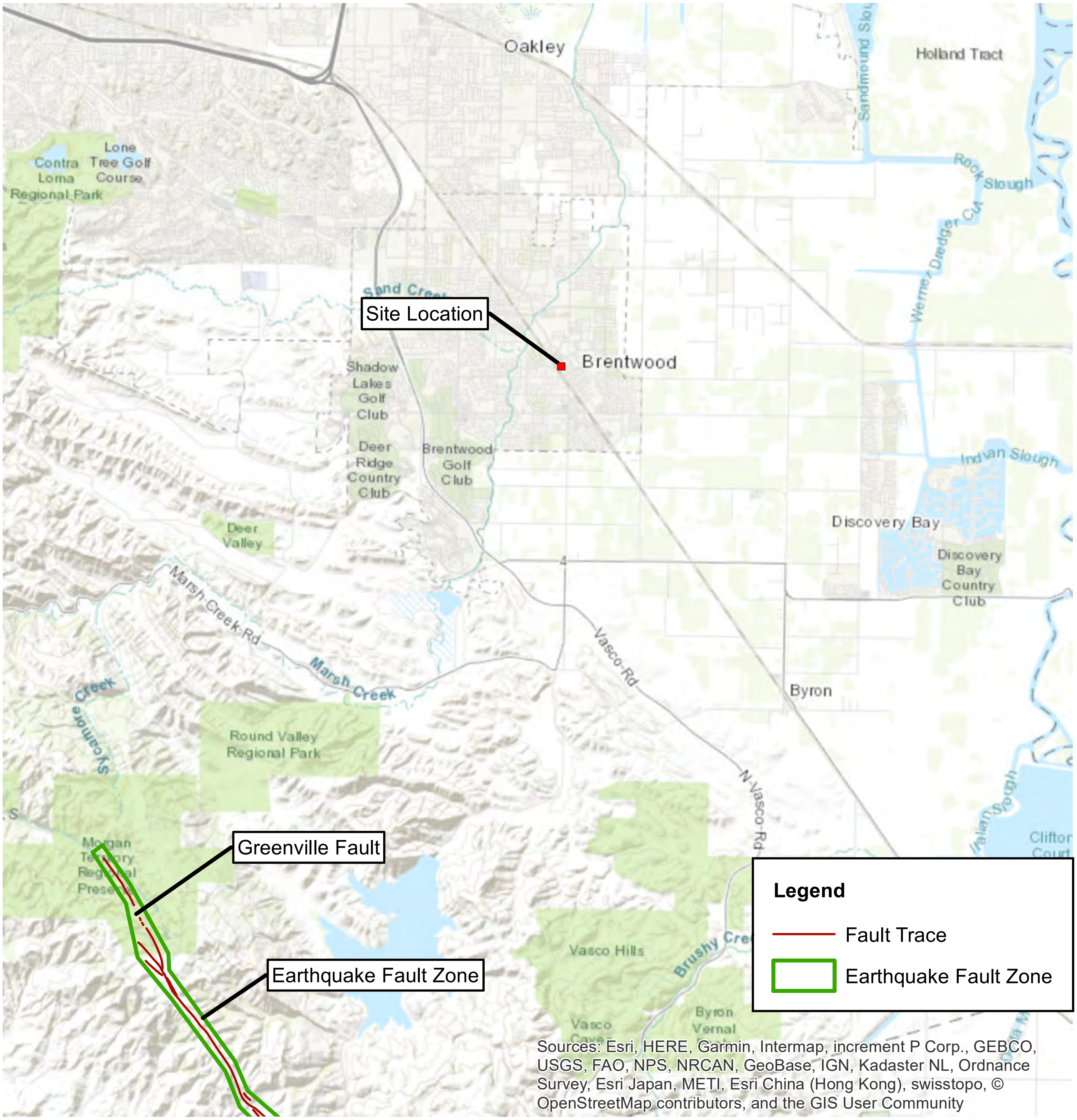
BSK ASSOCIATES

PROJECT NO.	G00000614
DRAWN:	07/10/23
DRAWN BY:	M. Romero
CHECKED BY:	REJ
FILE NAME:	Figures D-2 and D-6.indd

GEOLOGIC CROSS SECTIONS

Liberty Adult Comunity Education School
Gateway and Tobin Sites
Brentwood, California

Figure **C-4**



Reference: Special Studies Zones, Hayward Quad, CGS
http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps

BSK ASSOCIATES

PROJECT NO. G00000614	**ALQUIST-PRIOLO EARTHQUAKE FAULT ZONE**	Figure
Drawn: 07/11/2023		
DRAWN BY: MBC	Liberty Adult Community Education School Gateway and Tobin Sites Brentwood, California	**C-5**
CHECKED BY: REJ		
FILE NAME: Figure D-7 AP_Fault_map		

Legend

Landslide Hazard Zone

Liquefaction Hazard Zone

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

Reference: Special Studies Zones, Brentwood Quad, CGS
http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps

N W E S

0 1,000 2,000 4,000 Feet

BSK ASSOCIATES

PROJECT NO. G00000614

Drawn: 07/11/2023

DRAWN BY: MBC

CHECKED BY: REJ

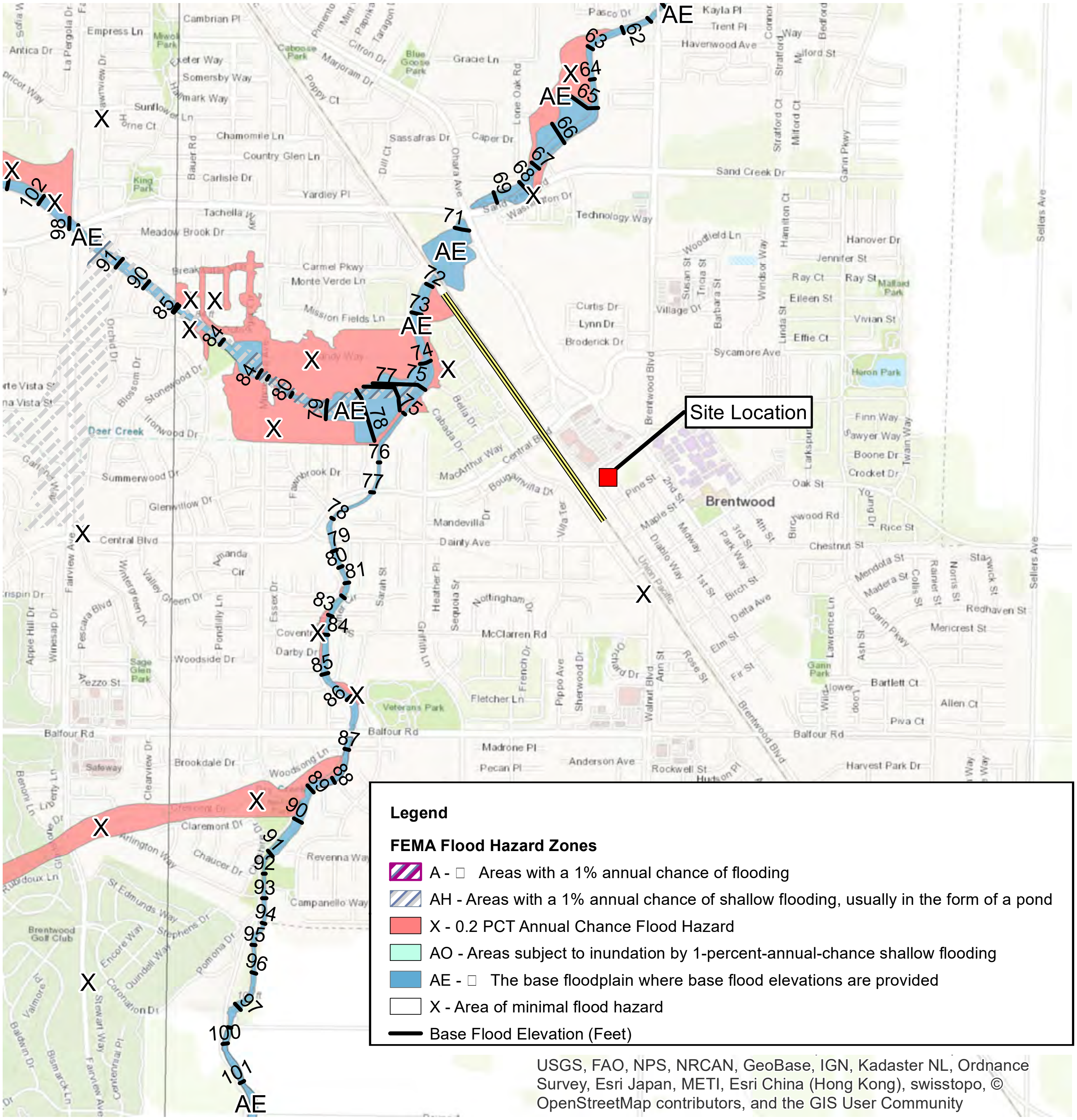
FILE NAME: Figure D-8 Seismic Hazard Zone

STATE SEISMIC HAZARD ZONES

Liberty Adult Community Education School
Gateway and Tobin Sites
Brentwood, California

Figure

C-6



Reference: FEMA Flood Hazard Layer, 06001C-NFHL, 5/7/2018

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N W E S

0 1,000 2,000 4,000 Feet

BSK ASSOCIATES

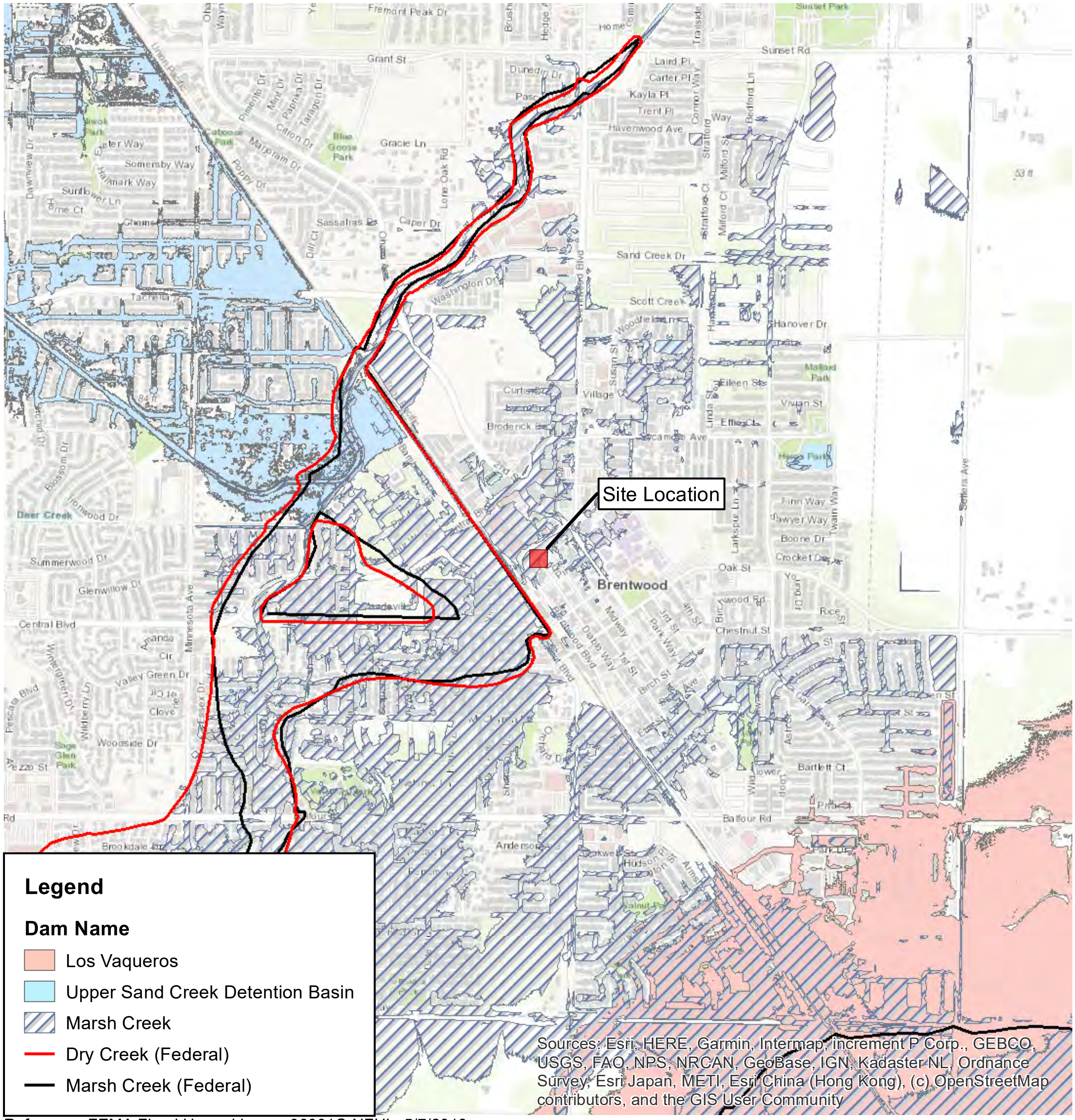
PROJECT NO. G00000614	
Drawn: 07/11/2023	
DRAWN BY: MBC	
CHECKED BY: REJ	
FILE NAME: Figure D-9 FEMA map	

FEMA FLOOD ZONES

Liberty Adult Community Education School
Gateway and Tobin Sites
Brentwood, California

Figure

C-7



Reference: FEMA Flood Hazard Layer, 06001C-NFHL, 5/7/2018
https://fmds.water.ca.gov/webgis/?appid=dam_prototype_v2
State of California OES Dam Inundation DVD

N W E S

Feet
0 1,000 2,000 4,000

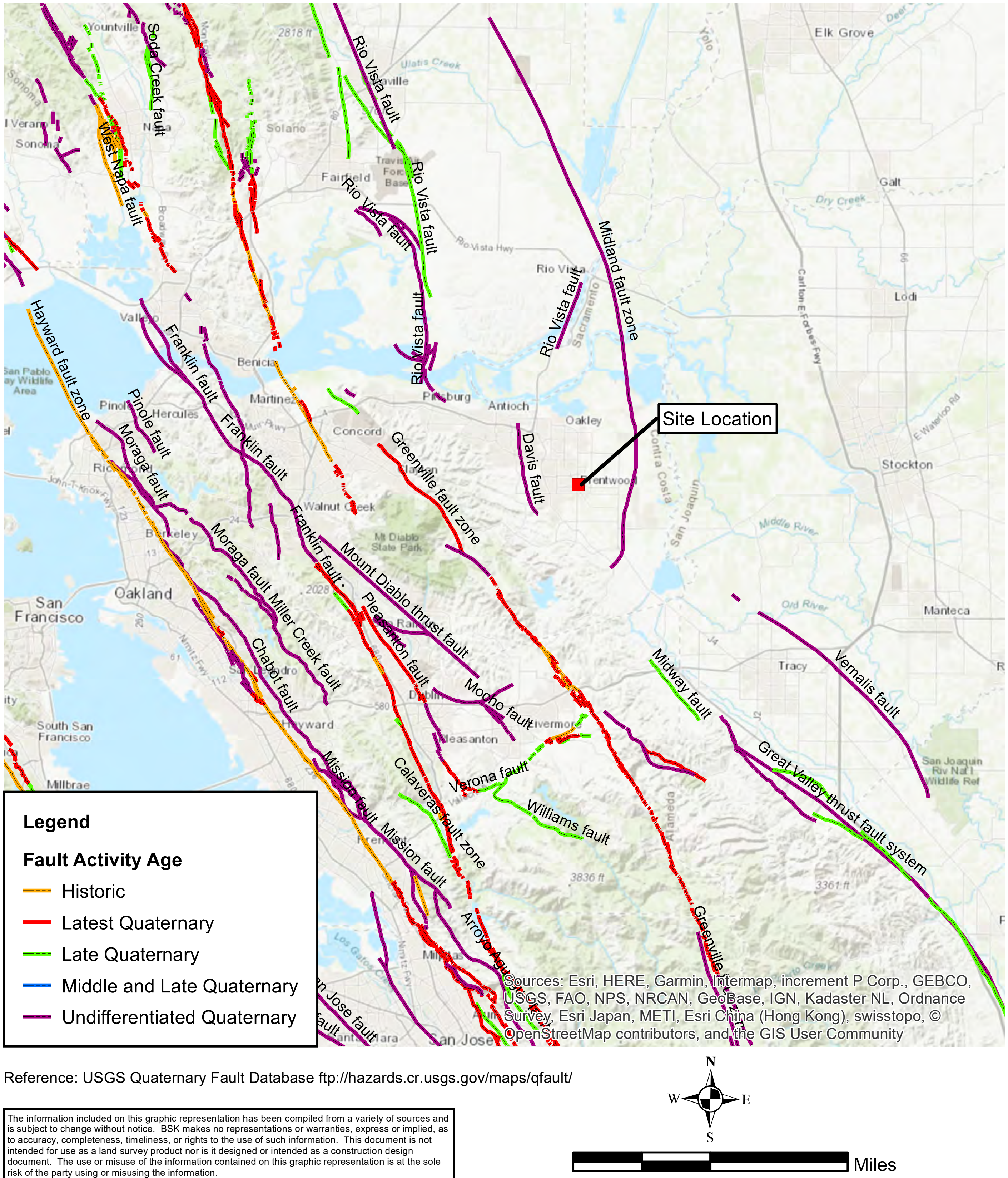
BSK ASSOCIATES

PROJECT NO. G00000614	
Drawn: 7/17/2023	
DRAWN BY: MJR	
CHECKED BY: REJ	
FILE NAME: Figure C-8 inundation	

DAM INUNDATION ZONES

Liberty Adult Community Education School
Gateway and Tobin Sites
Brentwood, California

Figure
C-8



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

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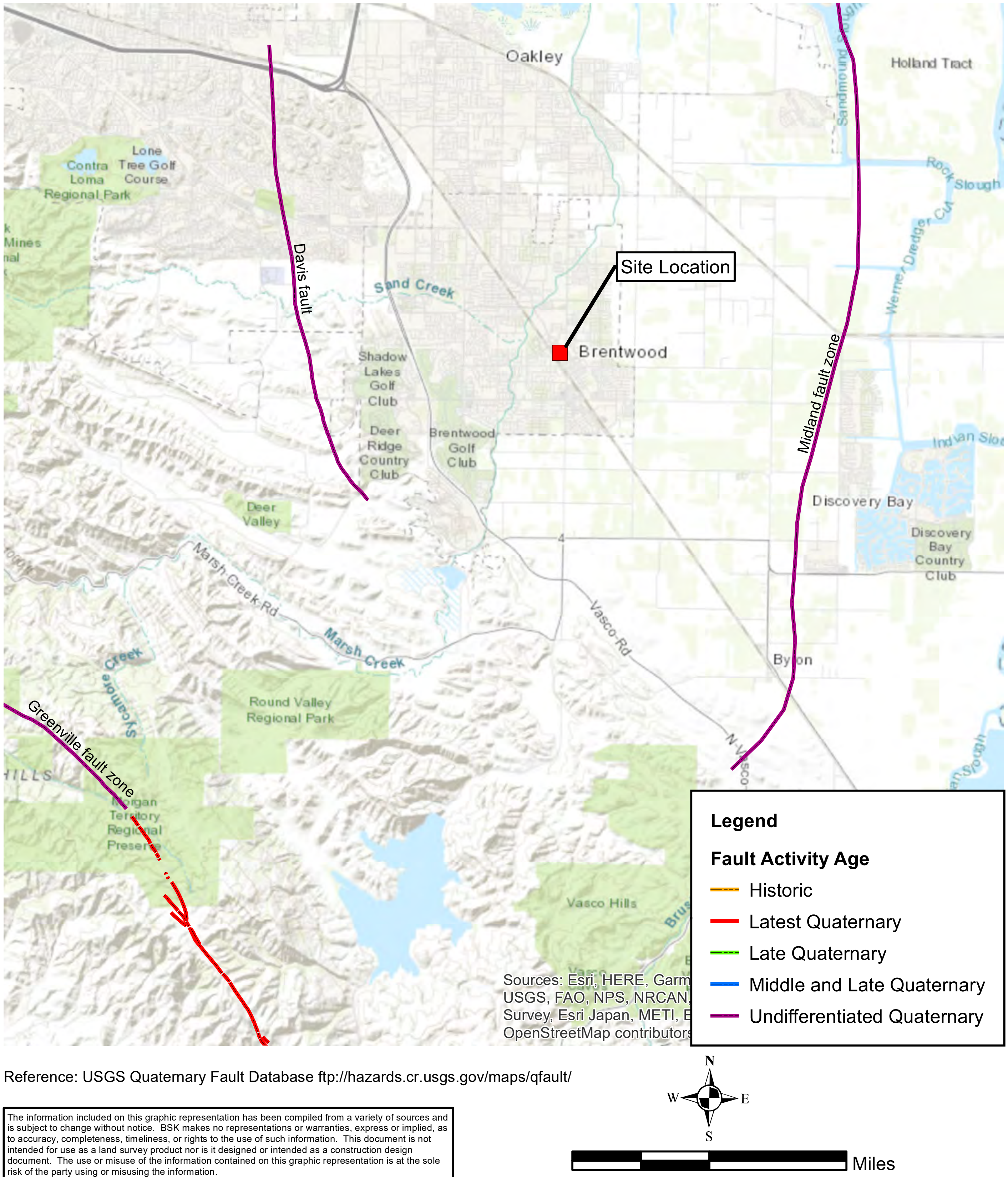
BSK ASSOCIATES

PROJECT NO. G00000614	**REGIONAL FAULT MAP**
Drawn: 07/11/2023	
DRAWN BY: MBC	Liberty Adult Community Education School Gateway and Tobin Sites Brentwood, California
CHECKED BY: REJ	
FILE NAME: Figure D-10 Regional Fault map	

Figure **C-9**



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

BSK ASSOCIATES

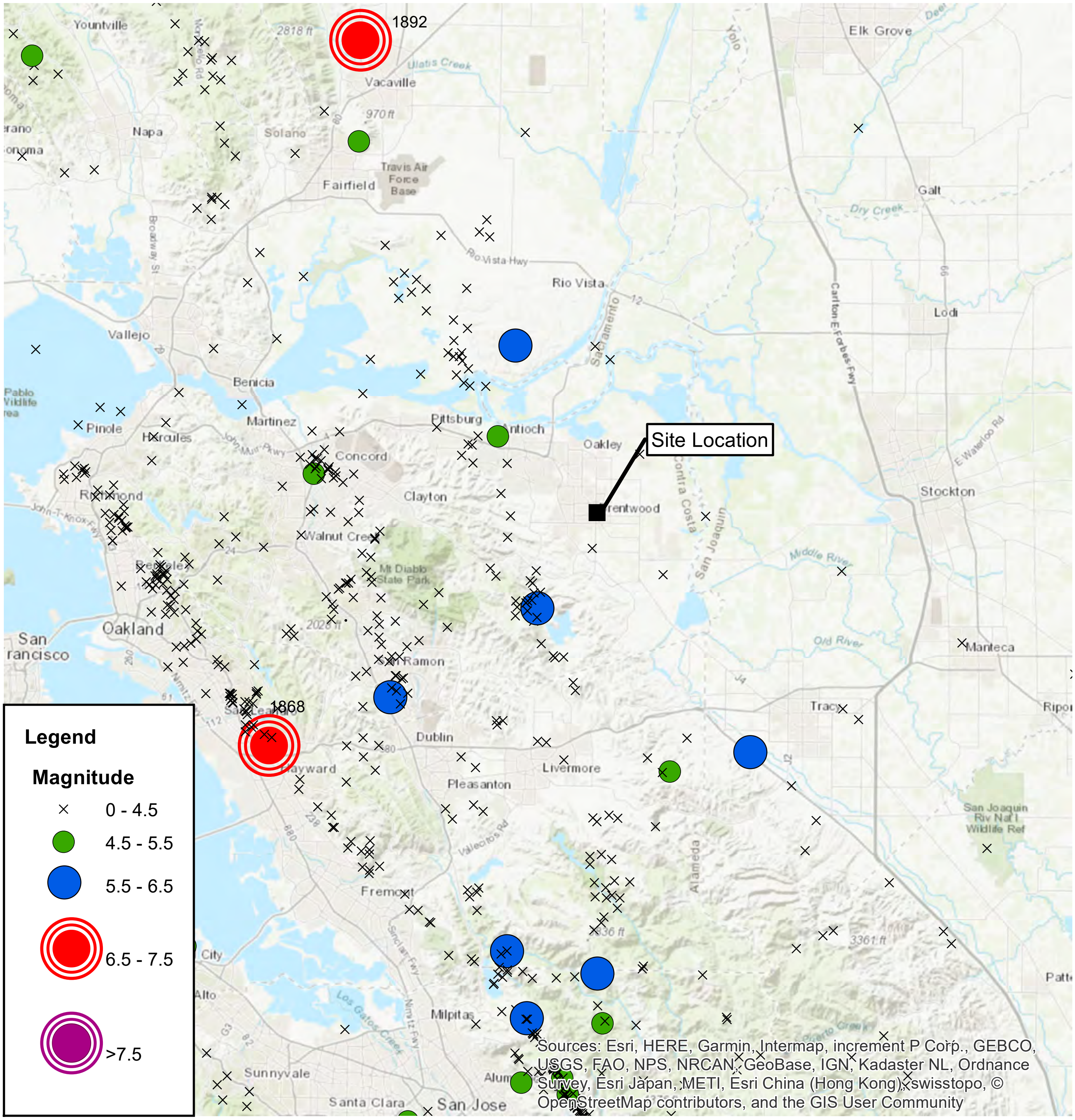
PROJECT NO. G00000614
Drawn: 07/11/2023
DRAWN BY: MBC
CHECKED BY: REJ
FILE NAME: Figure D-11 Local Fault map

LOCAL FAULT MAP

Liberty Adult Community Education School
Gateway and Tobin Sites
Brentwood, California

Figure

C-10



Source: National Seismic Hazard Model (NSHM) Earthquake Catalogs, 2014 NSHM Catalogs, USGS, https://github.com/usgs/nshmp-haz-catalogs

BSK ASSOCIATES

PROJECT NO. G00000614	
Drawn: 07/11/2023	
DRAWN BY: MBC	
CHECKED BY: REJ	
FILE NAME: Figure D-12 Historic Earthquakes map	

HISTORICAL EARTHQUAKES

Liberty Adult Community Education School
Gateway and Tobin Sites
Brentwood, California

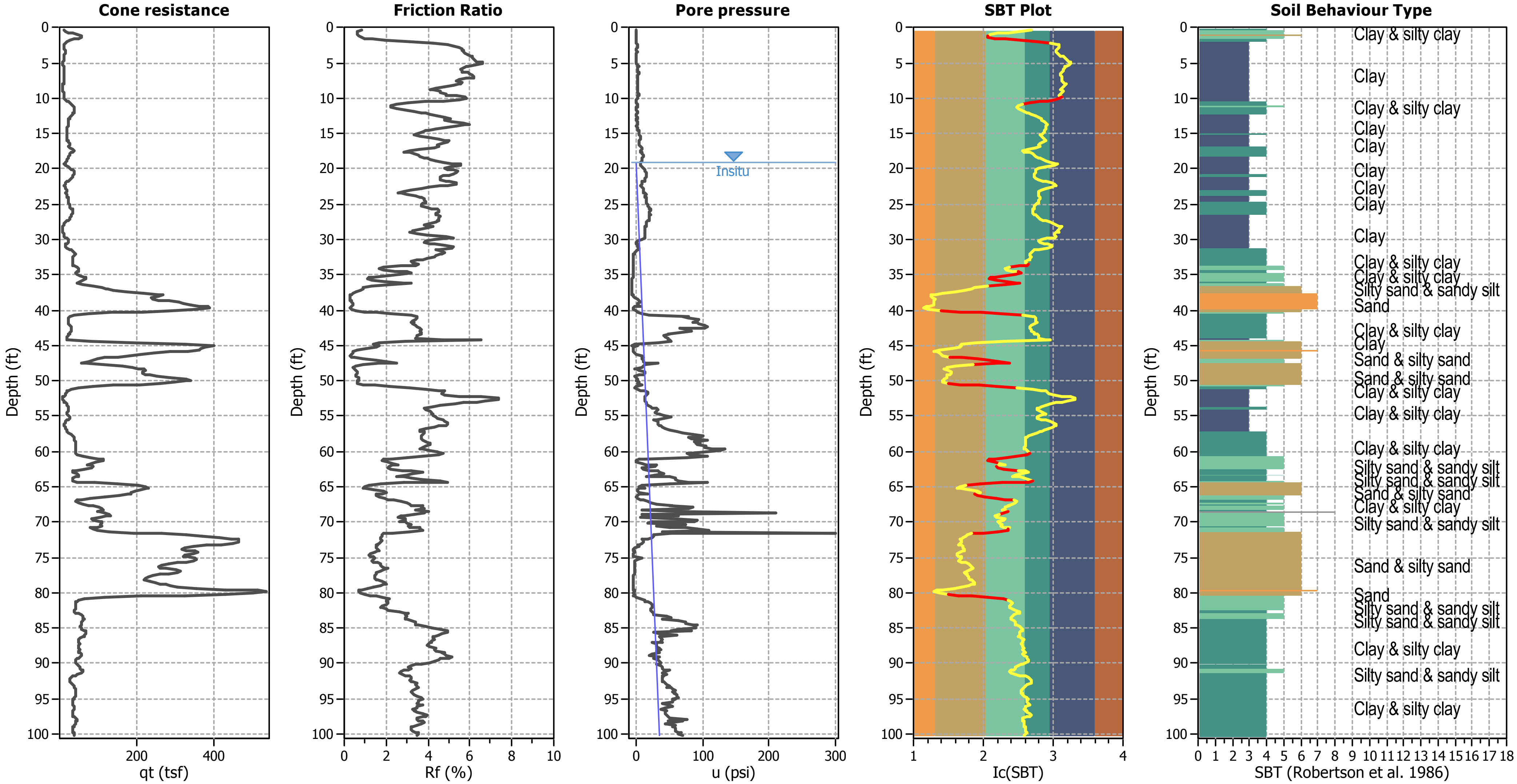
Figure

C-11

APPENDIX D

CONE PENETRATION TEST LOGS, LIQUEFACTION ANALYSIS, AND SHEAR WAVE VELOCITIES

CPT basic interpretation plots



Input parameters and analysis data

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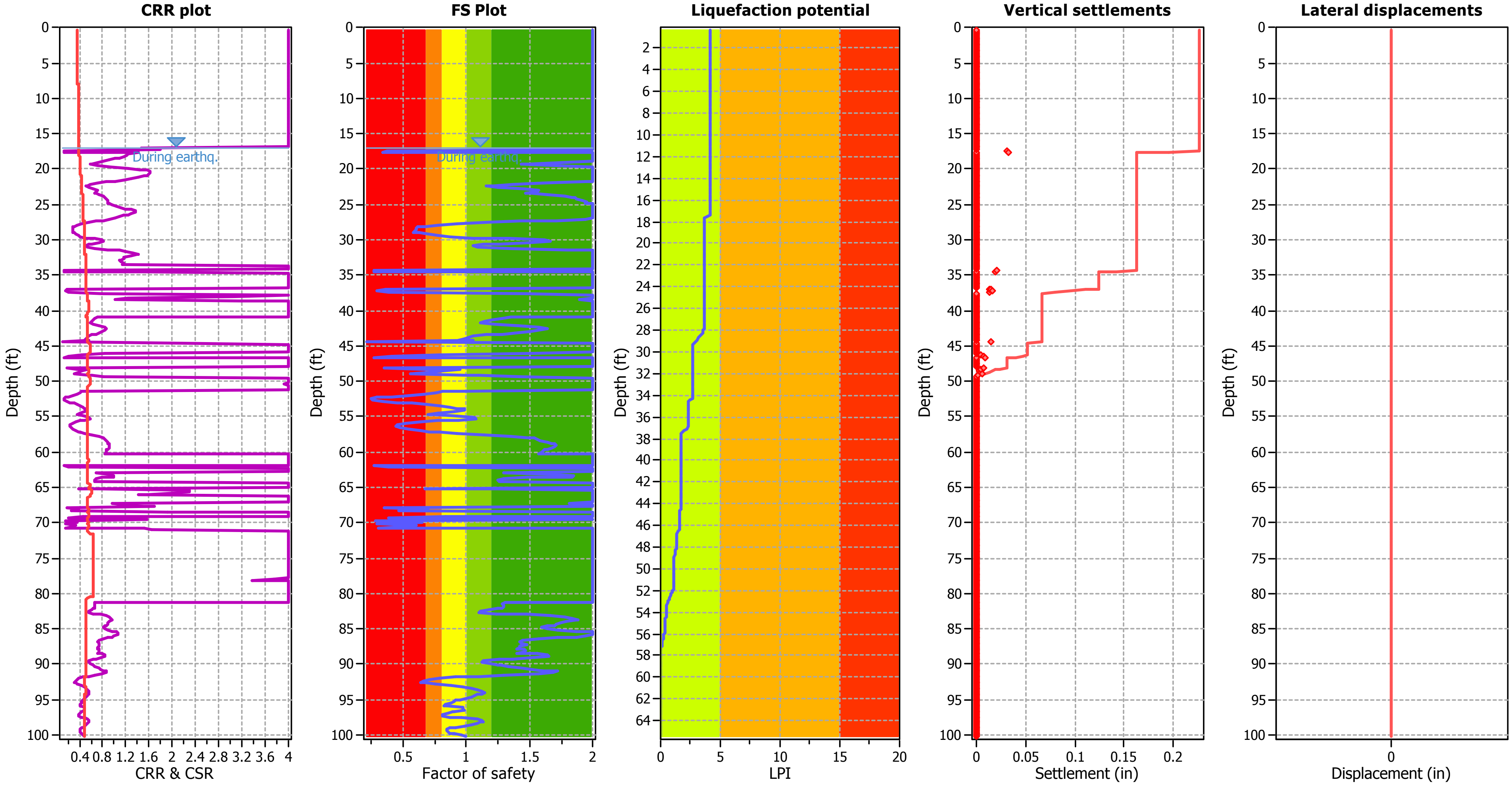
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	19.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots



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Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	19.00 ft	Fill height:	N/A	Limit depth:	N/A

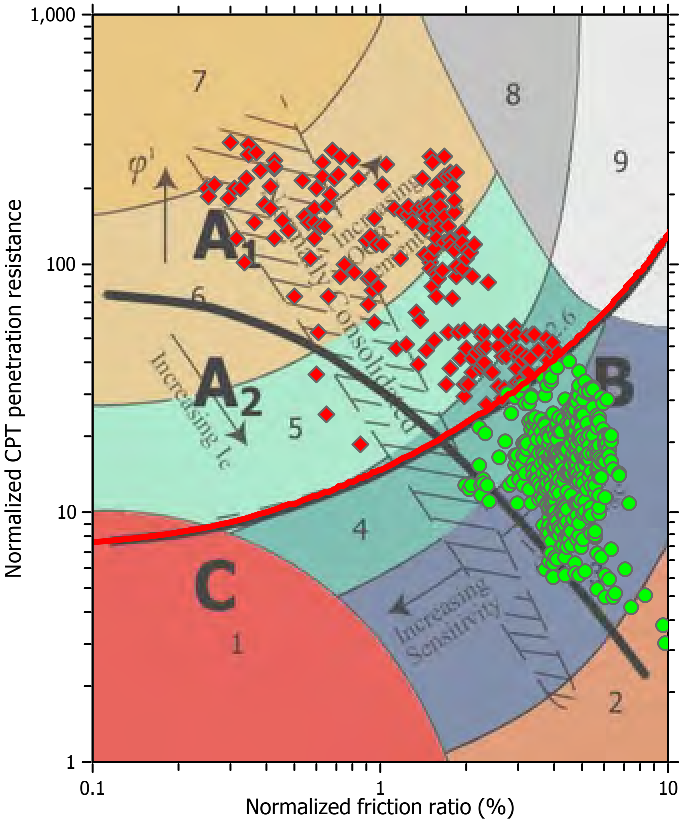
F.S. color scheme

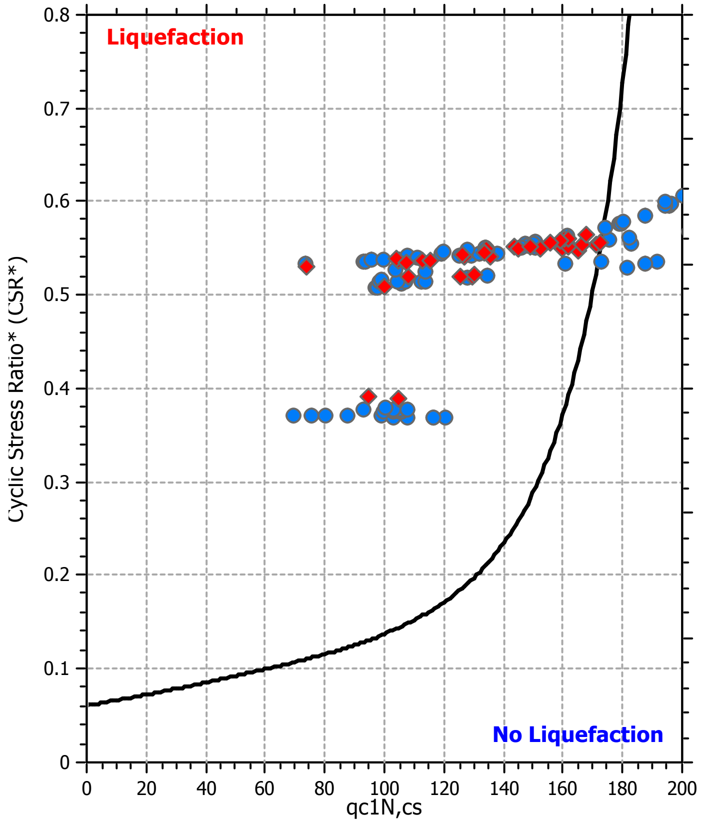
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

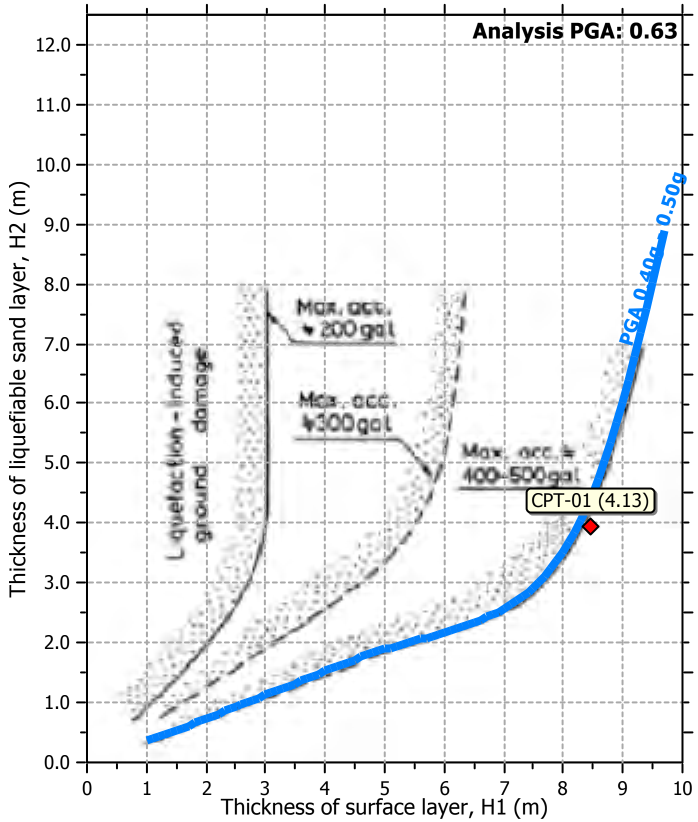
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots







Input parameters and analysis data

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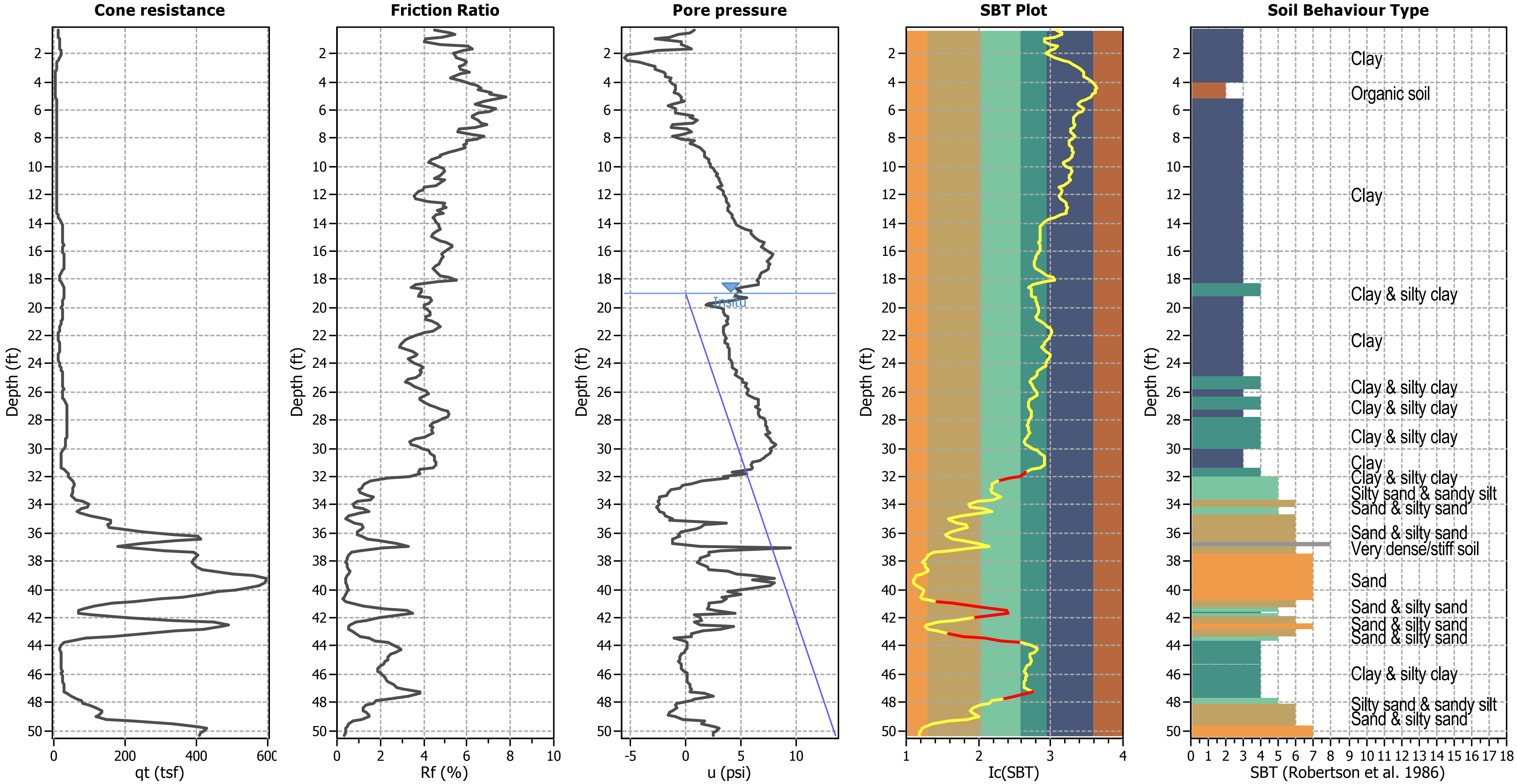
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	19.00 ft	Fill height:	N/A	Limit depth:	N/A

CPT basic interpretation plots



Input parameters and analysis data

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Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
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Depth to water table (insitu):	19.00 ft	Fill height:	N/A	Limit depth:	N/A

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4. Clayey silt to silty
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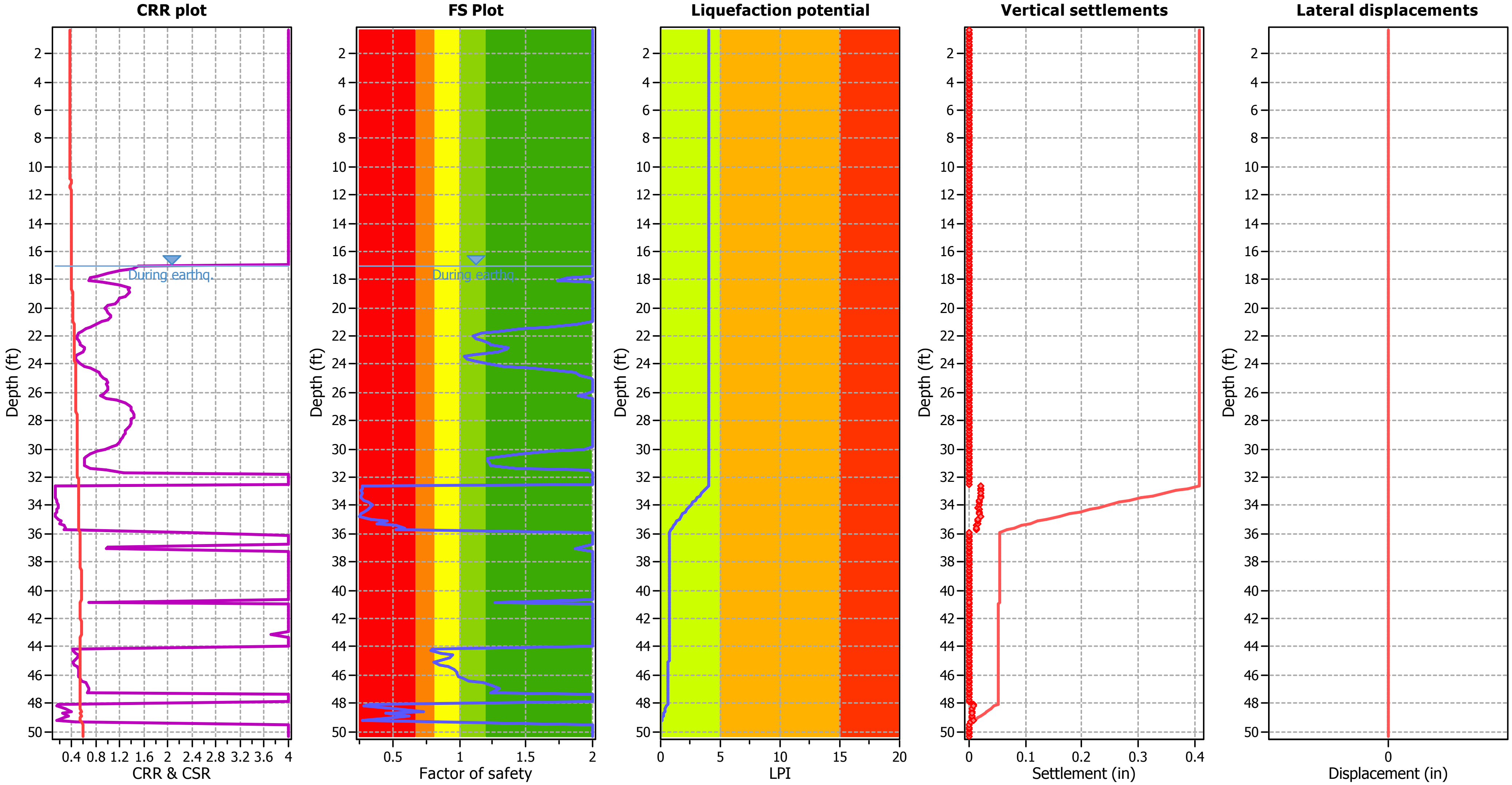
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

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Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
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Earthquake magnitude M_w:	7.40	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
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Depth to water table (insitu):	19.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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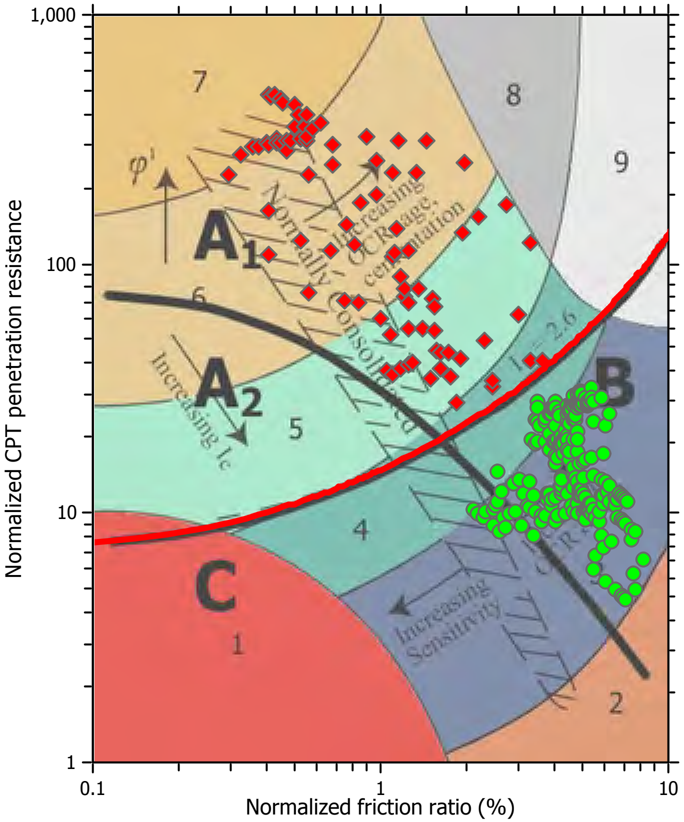
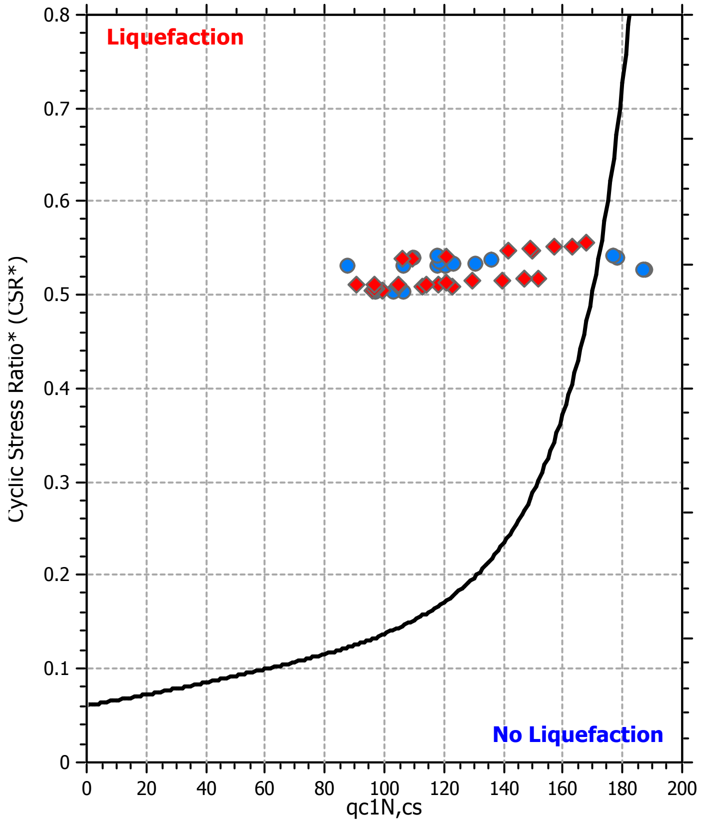
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

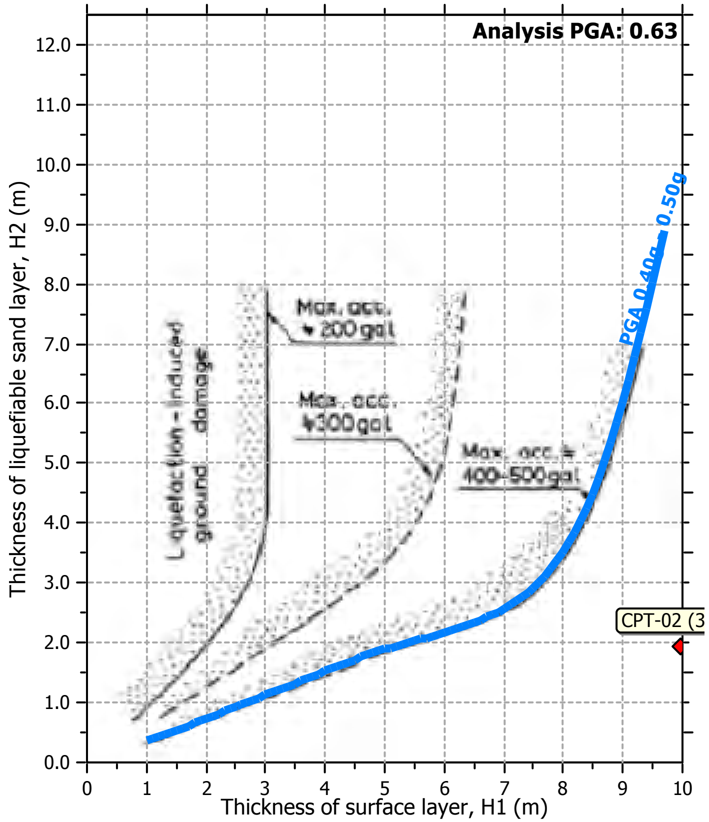
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots







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Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
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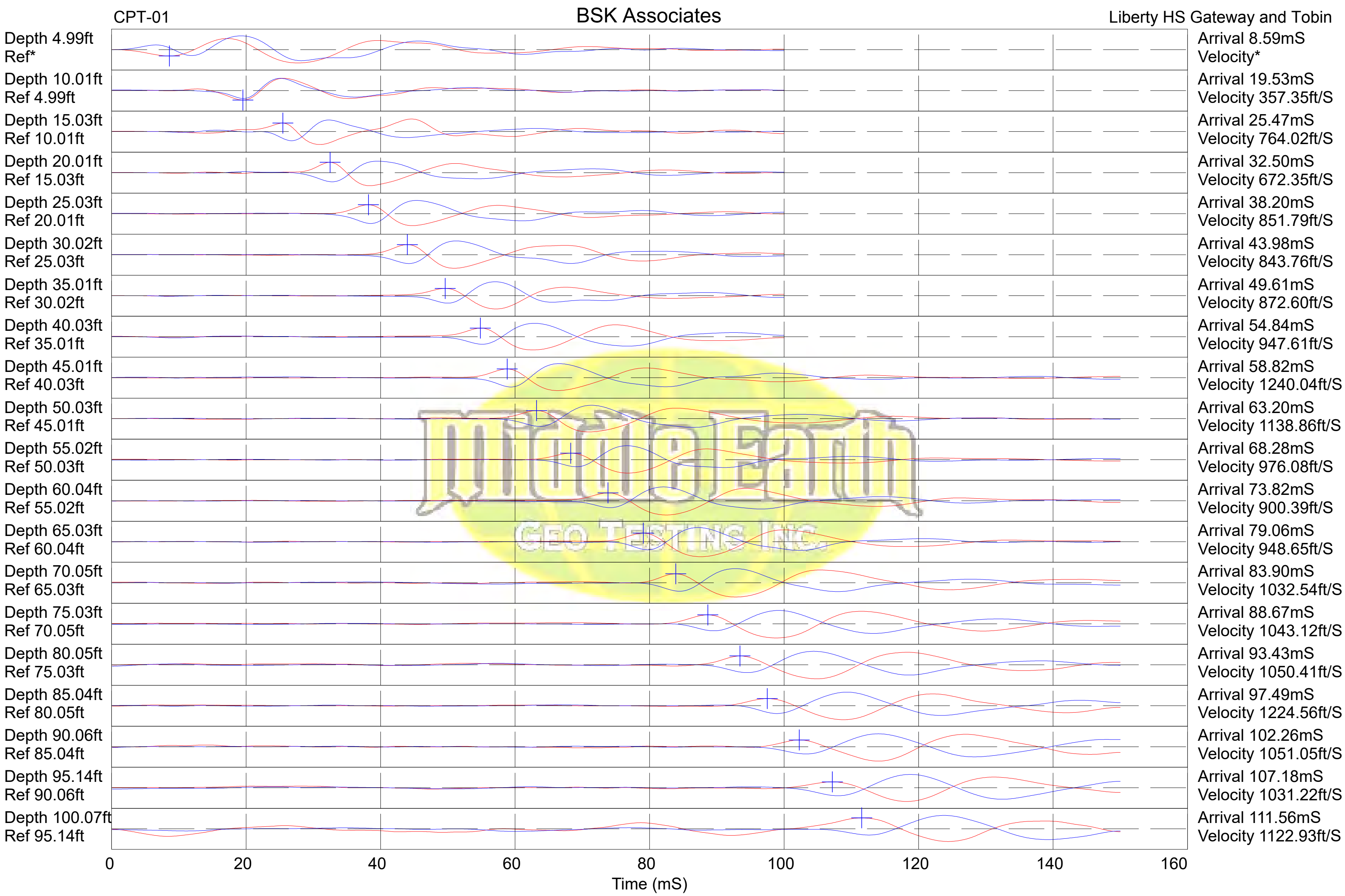
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	19.00 ft	Fill height:	N/A	Limit depth:	N/A



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

APPENDIX E

SUMMARY OF COMPACTION REQUIREMENTS

Area	Compaction Requirements (See Notes 1, 2, 3, 4, 6)
Subgrade Preparation and Placement of General Engineered Fill, Including Imported Fill	Compact upper 12 inches of exposed subgrade and entire fill to a minimum of 90 percent compaction near optimum content for granular soils and to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content for clayey soils.
Trenches[5]	Compact trench backfill to a minimum of 90 percent compaction near optimum moisture content for granular soils and to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content for clayey soils. Compact upper 12 inches of trench backfill to a minimum of 95 percent relative compaction near optimum moisture content for granular soils and Class 2 aggregate base and to a minimum of 92 percent relative compaction at least 2 percent over optimum moisture content for clayey soils within paved areas. Proper granular bedding and shading should be placed under and around new utilities.
Exterior Flatwork	Compact upper 12 inches of subgrade to a minimum of 90 percent compaction at near optimum moisture content for granular soils and to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content for clayey soils. Compact aggregate base to a minimum of 90 percent compaction at near optimum moisture content. Where exterior flatwork is exposed to vehicular traffic, compact as recommended below for pavements.
Pavements	Compact upper 12 inches of subgrade to a minimum of 95 percent compaction at near optimum moisture content for granular soils and to a minimum of 92 percent compaction at a minimum of 2 percent over optimum moisture content for clayey soils. Compact aggregate base to a minimum of 95 percent compaction near optimum moisture content.

Notes:

(1) Depths are below finished subgrade elevation.
(2) All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D1557.
(3) Fill material should be compacted in lifts not exceeding 8 inches in loose thickness.
(4) All subgrades should be firm and stable.
(5) In landscaping areas only, the percent compaction in trenches may be reduced to 85 percent.

APPENDIX F

IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will <u>not</u> likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will <u>not</u> be adequate to develop geotechnical design recommendations for the project.

Do <u>not</u> rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do <u>not</u> rely on an executive summary. Do <u>not</u> read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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