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December 3, 2018 BSK Project No.: G16-054-11L

Ms. Sue Rice
Program Director
Vanir Construction Management, Inc.
24400 Amador Street
Hayward, California 94544

SUBJECT: Supplemental Geotechnical Information
Harder Elementary School Campus Reconstruction
495 Wyeth Road
Hayward, California

Dear Ms. Rice:

The purpose of this letter is to provide a discussion regarding the applicability of our existing geotechnical investigation and geologic hazards assessment report[1] (attached) for the planned campus reconstruction of Harder Elementary School located at 495 Wyeth Road in Hayward, California. It is our understanding that since the issuance of our geotechnical report, the layout of the planned campus buildings and relative improvements has changed. As shown on the attached Figure 1, Site Plan, the new campus consists of kindergarten buildings K1 and K2 (approximately 5,000 square feet [sf] each), buildings A, M, and L (combined footprint of approximately 20,000 sf), and Building B (approximately 32,500 sf).

DISCUSSION

Subsurface Exploration

Our geotechnical exploration for the project consisted of nine (9) exploratory soil borings (20 to 30 feet deep) and three (3) Cone Penetration Tests (50 feet deep). The locations of our borings and CPTs with respect to the proposed buildings are shown on Figure 1.

The borings and CPTs generally encountered interbedded fine-grained alluvial soils consisting of alternating layers of clays, silts, and clayey sands. The fine-grained soils were generally firm to hard, while

[1] Geotechnical Investigation Report and Geologic Hazard Assessment, Harder Elementary School Campus Reconstruction, Hayward, California, dated July 22, 2016, BSK Project No. G16-054-11L

the coarse-grained soils were medium dense with a high clay content. The subsurface conditions encountered are in general agreement with Witter et al (2006[2]) who mapped the campus as being underlain by Holocene age (less than 11,800 years) alluvial fan deposits consisting of sand, gravel, silt, and clay, and is moderately to poorly sorted, and moderately to poorly bedded.

Seismic Design Parameters

The geotechnical report presented seismic design parameters based on the 2013 California Building Standards Code (CBC [California Code of Regulations, Title 24]). These seismic design parameters are unchanged in the 2016 CBC and are therefore applicable for the planned campus reconstruction being currently designed to comply with the 2016 CBC.

CONCLUSIONS

Based on the above discussion, our review of the new campus layout plan and review of our existing geotechnical report, we believe that our exploration plan provides adequate coverage for the new layout to adhere to the requirements of the 2016 CBSC for California public schools, which is a minimum of two exploration points per building and one point per every 5,000 sf. The remainder of the geotechnical recommendations from our 2016 report remain valid for this project.

CLOSURE

We appreciate the opportunity of providing our services to the District on this project and trust this meets your needs currently. Please, contact us at (925) 315-3151 if you have any questions or require additional information.

Respectfully submitted,
BSK Associates

Omar K. Khan
Project Geologist

Carrie Foulk, PE, GE
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
CARRIE L. FOULK
No. 3016
Exp. 09/30/19
GEOTECHNICAL
STATE OF CALIFORNIA

Enclosures:
BSK's 2016 Geotechnical Investigation Report and Geologic Hazard Assessment (7/22/16)
Figure 1 – Site Plan

[2] Witter, R.C., Knudsen, K.L., Sowers, J.M., Wentworth, C.M., Koehler, R.D., and Randolph, C.E., 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California: U.S. Geological Survey in cooperation with the California Geological Survey, Open-File Report 2006-1037, scale 1:200,000

GEOTECHNICAL INVESTIGATION REPORT AND GEOLOGIC HAZARD ASSESSMENT

HARDER ELEMENTARY SCHOOL CAMPUS RECONSTRUCTION

BSK PROJECT NO. G16-054-11L

PREPARED FOR:

HAYWARD UNIFIED SCHOOL DISTRICT
24400 AMADOR STREET
HAYWARD, CALIFORNIA 94544

July 22, 2016

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July 22, 2016 BSK Project Number G16-054-11L

Vanir Construction Management, Inc.
24400 Amador Street
Hayward, California 94544

Attention: Mike Regan (Mike.Regan@vanir.com)
Senior Project Manager

Subject: **Geotechnical Investigation Report and Geologic Hazards Assessment**
Campus Reconstruction
Harder Elementary School
Hayward, California

Dear Mr. Regan:

We are pleased to submit four bound copies of our geotechnical investigation report and geologic hazards assessment for the planned re-construction of Harder Elementary School within the Hayward Unified School District in Hayward, California. The enclosed report describes the geotechnical investigation performed and presents our geotechnical recommendations for foundations, retaining walls, earthwork and pavements. A geologic and seismic hazards assessment is included as an appendix to this report. We had previously provided a draft geotechnical report entitled *Geotechnical Investigation Report and Geologic Hazard Assessment* dated May 17, 2016 (File No. G16-054-11L). Since the time of that report we have received new information of the layouts of the building, and therefore needed to perform additional exploration to meet the Division of State Architect (DSA) and the 2013 California Building Code standards. This report supersedes our May 17, 2016 draft report.

In summary, it is our opinion that the site does not pose significant geotechnical concerns that would preclude the planned development provided the recommendations presented in our report are incorporated in design and construction. The main geotechnical concern for the project site is the presence of moderately to highly expansive surface clays. The buildings can be supported on spread footings, deepened to mitigate the expansive soils at the site. The building floor slabs will need to be supported on “non-expansive” or lime-treated soils to reduce the impact of expansive soils at the site. A minor amount of liquefaction-induced settlement has been calculated; however, the magnitude of settlement is not anticipated to affect the design of the buildings.

These and other geotechnical recommendations pertaining to the proposed project are discussed in the report. The apparent geologic hazard for the project, other than those mentioned above, is the potential for strong ground shaking, which is typical of the entire San Francisco Bay Area. A summary of the geologic hazards is presented in the main text of this report and a detailed Geologic and Seismic Hazards Assessment that complies with State criteria is included in Appendix D.

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 soil 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.

Carrie L. Foulk, PE, GE #3016
Senior Geotechnical Engineer

Bradley E. Steen, PE, GE #2839
Principal Geotechnical Engineer

Martin Cline, CEG #2084
Senior Engineering Geologist

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Log of Borings B-1 through B-9

APPENDIX B – Cone Penetrometer Tests & Liquefaction Analyses

CPT-1 through CPT-3 logs and Liquefaction Analysis Results

APPENDIX C – Laboratory Test Results

Plate C-1 – Atterberg Limits

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APPENDIX D – Geologic and Seismic Hazards Assessment

APPENDIX E – Corrosivity Test Results by CERCO Analytical

APPENDIX F – Exhibit 1 - Summary of Compaction Requirements

1. INTRODUCTION

This report presents the results of our geotechnical investigation for the planned reconstruction of the Harder Elementary School within the Hayward Unified School District. A Vicinity Map showing the location of the project site is presented on Plate 1. Our investigation has been performed for the Hayward Unified School District (HUSD) and was coordinated with Mr. Mike Regan of Vanir Construction. This report contains a description of our site investigation methods and findings, including field and limited laboratory data. It provides geotechnical recommendations for the project and also presents a geologic and seismic hazards assessment for the campus.

1.1 Project Description

The proposed project will include the complete demolition of the existing campus and the construction of six structures, including: a multipurpose building, a library and computer lab, multi-story classrooms, a kindergarten building, a pre-K building and an administration office located at 495 Wyeth Road in Hayward, California. There will be new playfields surrounding the school and an outdoor dining area between the kindergarten and the multipurpose/kitchen building. Paved parking lots and a drop off area will be constructed at the northeast and southwest limits of the school property. We have based our work on the conceptual site plan provided in a bond measure planning phase presentation entitled "Preliminary Plans and 3d View.pdf" by Lionakis Architecture dated April 29, 2016. The Site Improvement Plan, Plate 2, shows the approximate locations of planned improvements overlain on a Google Earth image, showing the existing campus.

The one-story admin and classroom structures will be either wood-frame or steel-frame construction supported on a shallow foundation system. Exterior and interior wall loads are anticipated to be about 1 to 2 kips per lineal foot and column loads will be less than 50 kips. The two-story classroom and multipurpose buildings will like be steel-frame structures with exterior wall loads of about 1 kip per foot and column loads ranging from 65 to 160 kips. Although a grading plan is not currently available for the project, we anticipate that site grades will remain close to existing elevations and that cuts and fills during construction will be limited to less than a few feet. Excavations for the removal of existing and installation of new underground utilities are expected to be up to 5 feet deep.

If the actual project differs significantly from that described above, specifically if the grading differs from that we assumed 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 was to explore and evaluate the subsurface conditions at the site in order to provide geotechnical input for the design and construction of the planned improvements for

this project. The scope of services, as outlined in our March 9, 2016 proposal (File Number: GL16-13262) and our addendum dated May 10, 2016, field exploration, laboratory testing, engineering analysis, and preparation of this report. A geologic and seismic hazards evaluation for the entire school was also performed concurrently and is presented in Appendix D.

This investigation specifically excludes the assessment of site environmental characteristics, particularly those involving hazardous substances.

2. SITE INVESTIGATION

2.1 Field Exploration

Exploration locations and frequency were chosen to meet the requirements of the Division of State Architect (DSA) and the 2013 California Building Code, which requires a minimum of two borings per building and at least one per every 5,000 square feet of foundation footprint area.

Our subsurface investigation was performed on March 31 and June 29, 2016 to evaluate the subsurface conditions at the site for the planned re-construction. As outlined in our proposal, the field investigation consisted of drilling 9 borings and advancing 3 Cone Penetrometer Tests (CPTs) at the approximate locations shown on Plate 2. Middle Earth Geo Testing of Fremont, California was subcontracted to provide CPT services and Exploration GeoServices of San Jose was subcontracted to provide boring services. Two bulk samples were obtained in future pavement areas (R-1 and R-2) for Resistance (R)-value testing for use in pavement design.

Prior to subsurface exploration, Underground Service Alert (USA) was contacted to provide utility clearance and each exploration location was cleared for detectable underground utilities by GeoTech Utility Locating of Moraga, California. A drilling permit was obtained from the Alameda County Public Works Department (County). Upon completion of the field investigation, the borings and CPTs were backfilled with grout and capped with Quikrete in paved areas. Excess cuttings generated during drilling were disposed of at the site in unimproved areas near the locations of the borings.

The locations of the borings and CPTs were estimated by our field representative based on rough measurements from existing features at the site. Elevations shown on the boring logs were estimated using the elevation information available on Google Earth Pro. As such the elevations and locations of the borings 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 depths of approximately 20 to 30 feet below the existing ground surface. The borings were logged by an engineer of BSK Associates (BSK).

Relatively undisturbed samples of the subsurface materials were obtained using a split spoon sampler with a 2.5-inch inside diameter (I.D.) and a 3-inch outside diameter (O.D.) fitted with stainless steel liners. The samplers were driven 18 inches using a 140-pound, semi-automatic trip hammer falling 30 inches, and blow counts for successive 6-inch penetration intervals were recorded. The blow counts were reported on the final boring logs. After the sampler was withdrawn from the borehole, the samples were removed, 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 hand-held pocket penetrometer. The results of these tests are shown adjacent to the samples on the boring logs.

Soil classifications made in the field from auger cuttings and samples were re-evaluated in the laboratory after further examination and testing. The soils were classified in the field in general accordance with the Unified Soil Classification System (Visual/Manual Procedure - ASTM D2488). Where laboratory tests were performed, the designations reflect the laboratory test results in general accordance with ASTM D2487 as presented on Plate A-1. The Soil Description Key and Log Key are presented on Plates A-2 and A-3. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the soil boring logs, which are also presented in Appendix A. 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

We advanced three CPTs to further assess subsurface conditions at the site. The CPTs were performed using an integrated electronic cone system in accordance with ASTM D 3441. 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 0.85. The cone bearing (Qc) and sleeve friction (Fs) were measured and recorded during the tests at every 5 centimeter (about 2 inch) depth intervals.

The cone was pushed using a special rig, having a down pressure capacity of approximately 20 tons. The penetration tests were carried to a depth of approximately 50 feet below the existing ground surface. Our field engineer specified the CPT locations. The information gathered from the CPTs was used for identifying potential liquefiable and soft soils (to a depth of 50 feet), and for foundation design. The CPT data (cone bearing, sleeve friction, friction ratio, and equivalent Standard Penetration Test blow counts, N) versus penetration depth below the existing ground surface are presented on the attached report (Appendix B).

The stratigraphic interpretation of the CPT data was performed based on relationships between cone bearing and sleeve friction versus penetration depth. The friction ratio (R_f), which is sleeve friction divided by cone bearing, is a calculated parameter which is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone bearing and generate large excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing 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, 1986 and 1990, and Lunne, Robertson & Powell, 1997. It should be noted that it is not always possible to clearly identify a soil type based on cone bearing (Qc) and sleeve friction (Fs). In these situations, experience and judgment and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type.

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, unconsolidated-undrained triaxial compression (TXUU), and R-value tests. Most of the laboratory test results are presented on the individual boring logs. The results of the Atterberg Limits, TXUU, and R-Value tests are presented graphically in Appendix C.

Analytical testing was performed on a sample of near-surface soils (boring B-4) to assist in evaluating the corrosion potential of the on-site soils. The corrosivity testing was performed by CERCO Analytical of Concord, California using ASTM methods as described in CERCO Analytical's report. The corrosion results are presented in Appendix E.

3. SITE CONDITIONS

3.1 Site Description

Harder Elementary School is located at 495 Wyeth Road in Hayward, California. The campus sits on an approximately 7½-acre parcel located within a mixed use area of residential and commercial retail and light industrial development. The site is essentially flat at an elevation of about 57 feet above Mean Sea Level.

3.2 Subsurface Conditions

The underlying stratigraphy of the campus is interbedded fine-grained, alluvial soils. Based on our exploration, this alluvium consists of alternating layers of clays, silts and clayey sands. The clays and silts are typically firm to hard layers exhibiting low to high plasticity. The underlying clayey sands are generally medium dense with a high clay content. The current laboratory test results are indicative of soils with medium to high expansion potential when subjected to changes in moisture content.

Groundwater was recorded at depths of about 18½ to 20 feet below ground surface (bgs) in some of the borings. 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 log data in Appendix A.

It should be noted that soil and subsurface conditions can deviate from those conditions encountered at the boring 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. DISCUSSION AND CONCLUSIONS

4.1 General

Based on the results of our field investigation, it is our opinion that the proposed improvements are geotechnically feasible and that the site may be developed as presently planned. This conclusion is based on the assumption that the recommendations presented in this report will be incorporated in the design and construction of this project. The significant geotechnical issue for the proposed campus is the presence of moderately to highly expansive surface clays. While these soils pose additional challenges to the proposed improvements, there are conventional methods which can aid in mitigating the effects of these existing conditions.

4.2 Geologic and Seismic Hazards Summary

As required by the State of California in Title 24 of the California Building Code, a geologic and seismic hazard evaluation is needed for school developments. BSK has provided an evaluation of the campus, along with a discussion of the geology of the site and its vicinity in a separate geologic and seismic hazards assessment report presented in Appendix D. In this assessment, we conclude that the planned structures are free of most geologic and seismic hazards except for: 1) the presence of moderately to highly expansive surface clays, 2) the potential for minor liquefaction-induced settlements during a significant seismic event, and 3) the potential for strong ground shaking, which is typical of the entire San Francisco Bay Area. Expansive soils are discussed briefly above and recommendations for mitigation will be discussed later in this report.

4.2.1 Faulting and Seismic Shaking

The San Francisco Bay Area is seismically dominated by the active San Andreas Fault system. This fault system movement is distributed across a complex system of generally strike-slip, right-lateral parallel and sub-parallel faults including, among others, the San Andreas, San Gregorio, Hayward and Calaveras faults.

The site is not located within an Alquist-Priolo Earthquake Fault Zone and no mapped active fault traces are known to transverse the site. Nevertheless, the site is located in a seismically active area of California. We expect the site to be subjected to substantial ground shaking due to a major seismic event on the active faults in the Bay Area and surrounding regions during the design life of the project.

According to a recent study[1], there is a 63 percent probability that one or more magnitude M6.7 or greater earthquakes will occur in the San Francisco Bay Area within the next approximately 30 years (between 2007 and 2036).

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. Therefore, the design of building structures for this project should incorporate the seismic design parameters presented in the geologic and seismic hazard report presented in Appendix D.

4.2.2 Soil Liquefaction

Liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase, resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil is not confined. Soils most susceptible to liquefaction are loose, clean, uniformly graded, silt and fine sand, as well as some lean clay deposits. Based on the subsurface exploration performed for the investigation, the site is underlain by interbedded alluvial soils consisting primarily of firm to hard sandy and silty clays, and medium dense clayey sand.

In order for liquefaction 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
- Ground shaking is of sufficient intensity to act as a triggering mechanism

In addition, after 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 submerged silty/clayey sands and less plastic clays below the water table are considered to be potentially liquefiable.

We evaluated liquefaction potential across the site in our current CPTs for the project (CPT-1 through CPT-3). Based on the shallow historical depth of groundwater and the relative density of the layers of

[1] Field, E.H., Miler, K.R., and the 2007 Working Group on California Earthquake Probabilities (2008), Forecasting California's Earthquakes – What Can We Expect in the Next 30 Years?: U.S. Geological Survey, Fact Sheet 2008-3027, 4 p. (http//:pubs.usgs.gov/fs/2008/3027/).

more granular soils and less plastic fine-grained soils that underlie the planned classroom building, we anticipate that the cyclic stress associated with the design peak horizontal ground acceleration (PGA_M) is sufficient to result in some liquefaction-induced settlement at this site. Based on this information, we performed liquefaction analyses for the CPTs using the methods proposed by Boulanger and Idriss (2014)[2]. For our analyses, we used peak ground accelerations of 0.98g (based on site-specific ground motion analysis), associated with an earthquake magnitude of M7.0. Historically high groundwater depths mapped by California Geological Survey show the site to be in an area where groundwater has been as high as 20 to 30 feet below grade. Groundwater was encountered in our borings at depths of about 18½ to 20 feet below the ground surface. Because of this discrepancy, the more conservative groundwater level used in our analyses was an assumed depth of 15 feet bgs to account for fluctuations in the groundwater table. All 3 CPTs predicted magnitudes of potential settlement of less than ½ inch based on the design level event. The results of these analyses are presented in Appendix C.

As noted in Special Publication 117A, "Guidelines for Evaluating and Mitigating Seismic Hazards in California, 2008", CGS states that *"it is very difficult to reliably estimate the amount of localized differential settlement likely to occur as part of the overall predicted settlement: localized differential settlements on the order of up to two-thirds of the total settlements anticipated should be assumed unless more precise predictions of differential settlements can be made"*. This would approximate our predicted differential settlement to less than 1/3 inch. Based on Ishihara (1985) and Youd and Garris (1995), we believe that the potential for ground surface disruption (such as sand boils, ground fissures, etc.) to occur at site is low due to the presence of the non-liquefiable clayey soils above the sandy layers and the lateral discontinuity of the liquefiable layers.

4.2.3 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. Due to the composition and apparent relative density of the soils above the water table within the maximum depth of our exploration (about 30 feet), we conclude that the potential for dynamic compaction/seismic settlement during a seismic event is low.

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[2] 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.

5. RECOMMENDATIONS

Presented below are recommendations for foundations, seismic considerations, concrete floor slabs, exterior flatwork, retaining walls, earthwork, construction considerations, site drainage, and pavements for this project.

5.1 Foundations

5.1.1 Shallow Footings

Based on our investigation, the loads for the proposed 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, depth of embedment, and width of footings are presented below.

Footing Bearing Capacity Recommendations			
Footing Type	**Allowable Bearing Pressure (psf)***	**Minimum Embedment (in)****	**Minimum Width (in)**
Exterior Continuous Footings	3,500	24	18
Isolated Interior Footings	3,500	24	18x18
Isolated Exterior Footings	3,500	24	18x18
* Pounds per square foot, dead plus live load. Includes factor of safety (FS) of at least 2.			
**Below lowest adjacent grade defined as bottom of slab on the interior and finish grade at the exterior.			

Allowable soil bearing pressures may be increased by one-third for transient loads such as wind and seismic loads. Where footings are located adjacent to below-grade structures or near major underground utilities, the footings should extend 9 inches below a 2H:1V (horizontal to vertical) plane projected upward from the structure footing or bottom of the underground utility to avoid surcharging the below grade structure and underground utility with building loads. Also, where utilities cross <u>under</u> 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 “Earthwork” section of this report. In addition, where utilities cross <u>through</u> 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.

Concrete for footings should be placed neat against firm native soil or engineered fill. The footing excavations should not be allowed to dry before placing concrete. If shrinkage cracks appear in the footing excavations, the excavations should be thoroughly moistened to close all cracks prior to concrete placement. The footing excavations should be monitored by a representative of BSK for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials. If soft or loose materials are encountered at the bottom of the footing excavations, they should be removed and

replaced with lean concrete or engineered fill. BSK should also be present during the overexcavation. Unit prices for such overexcavation and backfilling should be obtained during contractor bidding for this project.

5.1.2 Drilled Piers

New drilled pier supporting covered walkways (not fence posts) should be at least 12 inches in diameter, spaced at least three pier diameters center to center (for axial capacity), and extend at least 5 feet below grade. Drilled piers that extend into firm soil may be designed using an allowable skin friction of 450 psf for axial compressive and 300 psf for uplift loads. These values include a factor of safety of two, and may be increased by one-third for resisting total loads, including wind and seismic. The upper 2 feet of soil should be ignored for calculation of skin friction unless the ground surface is confined by paving or a slab.

We recommend steel reinforcement and concrete be placed within about 4 to 6 hours upon completion of each drilled pier hole; as a minimum, the holes should be poured the same day they are drilled. 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. Our borings indicate that shallow drilled piers can be drilled with a standard flight auger using a standard rig with Kelly bar, subject to access restrictions. The bottom of the drilled holes should be cleaned such that no more than 2 inches of loose soil remains in the hole prior to placement of concrete. A representative from BSK should be present to observe drilled holes to confirm bottom conditions prior to placing steel reinforcement.

If groundwater is encountered within the drilled pier holes, no more than 6 inches of standing water should be present during concrete placement. Otherwise, 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. In order to develop the design skin friction value previously provided, concrete used for pier construction should have a slump of 6 to 8 inches.

5.1.3 Resistance to Lateral Loads

Lateral loads applied against footings may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundation. The frictional and passive resistance may be assumed in design to act concurrently. An allowable friction coefficient of 0.30 between the foundations and supporting subgrade soils may be used. For passive resistance at this site, an allowable equivalent fluid pressure (unit weight) of 300 pounds per cubic foot (pcf) may be used against the sides of foundations. The friction coefficient and passive pressure values include factors of safety of about 1½. We based these lateral load resistance values on the assumption that the concrete for footings are either placed directly against undisturbed

soils or that the voids created from the use of forms are backfilled with properly compacted onsite soil per the recommendations in the "Earthwork" section of this report or other approved material, such as lean concrete.

Resistance to lateral loads for drilled piers can be provided by passive resistance against the piers using an allowable equivalent fluid pressure of 300 pcf up to a maximum of 2,000 psf. The passive resistance may be applied to a width of twice the diameter of the piers. Piers should be spaced at least 6 diameters apart (center to center) or lateral resistance capacity reductions may be necessary. The passive pressure value includes a factor of safety of about 1½.

The passive pressure may be increased by one-third for wind and/or seismic loading. Passive resistance in the upper foot of soil cover below finished grades should be neglected unless the ground surface is confined by concrete slabs, pavements, or other such positive protection.

5.1.4 Modulus of Subgrade Reaction

A modulus of subgrade reaction, K_{V1}, of 115 pounds per square inch per inch (pci) of deflection (based on a one square foot bearing plate) is considered applicable to the new footings and grade beams. The modulus of subgrade reaction is typically reduced for foundation or slab sizes larger than 1 square foot. For various slab sizes, the subgrade modulus may be calculated using the following formulas:

Square: $$K_S = (K_{V1}) \times \left(\frac{1\text{ foot}}{B}\right)$$

Rectangular: $$K_R = (K_{V1}) \times \left(\frac{1\text{ foot}}{B}\right) \times \left(\frac{m+0.5}{1.5\times m}\right)$$

Where:

- K_{V1} is the modulus of subgrade reaction for a 1 square foot plate (in units of pci);
- B is the width of the foundation/slab (in units of feet);
- m is the ratio of the foundation/slab length divided by its width (unitless); and
- K_S and K_R are the adjusted modulus of subgrade reaction based on the actual dimensions of the foundation/slab (in units of pci).

If a computer program is used to design the foundations for this project and it requires the input of a modulus of subgrade reaction for the site, the designer should check whether the program requires input of the unadjusted (i.e., K_{V1}) or adjusted (i.e., K_S or K_R) modulus of subgrade reaction.

5.2 2013 Seismic Design Criteria

The seismicity of the region surrounding the site is discussed in the "Faulting and Seismicity" Section of this report. From that discussion, it is important to note that the site is in a region of high seismic activity

and will likely be subjected to major shaking during the life of the project. As a result, structures to be constructed on the site should be designed in accordance with applicable seismic provisions of the building codes.

Based on our findings, we conclude that the Seismic Design Category E should be assigned to the planned buildings because S_1 is greater than 0.75. Therefore, as required by Section 1616A.1.3 of the 2013 CBC, we performed a site-specific-ground motion hazard analysis for this project. For details about the seismic design criteria to be used for this project and how it was developed, please refer to the geologic and seismic hazards assessment report in Appendix D.

5.3 Slabs-on-Grade

Slabs-on-grade for this project will consist of concrete floor slabs and 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 interior concrete slabs and exterior flatwork be underlain by 18 inches and 12 inches of “non-expansive” engineered fill, respectively, 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. See below for additional criteria for interior floor slabs.

High calcium quicklime treatment of the in-situ soils is recommended as an alternative to “non-expansive” fill. If this alternative is utilized, extensive quality control is needed as well as laboratory testing to evaluate the appropriate lime treatment mixture. The client needs to understand the risk of this approach if selected, as quicklime treatment requires extensive quality control. For estimating purposes, approximately 18 inches and 12 inches of soil would need to be treated for interior slabs and exterior flatwork, respectively, provided that the moisture content of the soils below that is at least 3 percent over optimum moisture. Our experience has indicated that about 5 percent high calcium quicklime by weight is typically needed for treatment. We should perform additional laboratory tests to refine this estimate. The negative impact of quicklime treatment on future vegetation should be considered in whether it should be used, and what mitigation measures are needed.

The “non-expansive” fill or quicklime-treated soil should extend a minimum horizontal distance of 5 feet beyond all building areas, where feasible, including the outer edge of perimeter footings and footings extending beyond perimeter walls, where flatwork is planned. The horizontal limits of treatment can be reduced to 3 feet elsewhere, such as for exterior flatwork. The over-build of the quicklime-treatment can be eliminated where landscaping is planned; however, it is important that the lime-treatment extends to the edge of the structural improvements. Therefore, special care should be exercised during surveying and staking of the building limits during construction. 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 or lime-treated soil is placed, as subgrades will dry out even through “non-expansive” fills or quicklime-treated soil. 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 or quicklime-treated soil prior to slab construction.

Where “non-expansive” fill or quicklime-treated soil is removed to install utilities inside the building, this layer should be replaced with new imported “non-expansive” fill.

5.3.1 Interior Floor Slabs

Concrete slabs-on-grade will include building interior floor slabs and exterior flatwork. All interior slabs should be supported over properly prepared subgrade soils, as described in the “Earthwork” section of this report. Due to the presence of expansive soils, interior floor slabs should be underlain by at least 18 inches of “non-expansive” fill or lime-treated soil.

Concrete floor slabs should be supported on at least 6 inches of angular gravel or crushed rock to enhance subgrade support for the slab. This material should not be considered part of the required minimum of 18 inches of “non-expansive” fill. If this material is desired to be used as a capillary break, it should be ¾ inch maximum size with no more than 10 percent by weight passing the #4 sieve. It is important that placement of this material and concrete be done as soon as possible after compaction of the “non-expansive” or lime-treated subgrade materials to reduce drying of the subgrade.

Floor slabs should have a minimum thickness of 5 inches. A Structural Engineer should design reinforcing and slab thickness. Because the floor slabs are to be supported on imported "non-expansive" granular material or lime-treated soil, a modulus of subgrade reaction is difficult to estimate. For estimating purposes, a value of 150 pounds per cubic inch may be used. This number will need to be confirmed when the source and type of the imported soil has been determined.

Special care should be taken so that reinforcement is placed at the slab mid-height. The floor slab should be separated from footings, structural walls, and utilities and provisions made to allow for settlement or swelling movements at these interfaces. If this is not possible from a structural or architectural design standpoint, it is recommended that the slab connection to footings be reinforced such that there will be resistance to potential differential movement.

5.3.2 Floor Slab Moisture

Subsurface moisture and moisture vapor naturally migrate upward through the soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. To reduce the impact of the subsurface moisture and potential impact of future introduced moisture (such as landscape irrigation or precipitation) the current industry standard is to place a vapor retarder on the compacted crushed rock layer. This membrane typically consists of visqueen or polyvinyl plastic sheeting at least 10 to 15 mils in thickness. It should be noted that although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering

manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building and all elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect indoor air quality.

Various factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the on-site soils affect slab moisture and can control future performance. In many cases, floor moisture problems are the result of either improper curing of floors slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding your proposed flooring applications.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) manual.

It is emphasized that we are not floor moisture proofing experts. We make no guarantee nor provide any assurance that use of capillary break/vapor retarder system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for floor slab moisture protection.

Exterior grading will have an impact on potential moisture beneath the floor slab. Recommendations for exterior draining are provided in the “Site Drainage and Storm Water Infiltration” section of this report.

5.3.3 Exterior Concrete Flatwork

Slabs-on-grade for this project will consist of exterior flatwork. 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 effect 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 Exhibit 1 (Appendix F). In addition, all exterior concrete slabs should be supported on a minimum of 12 inches of “non-expansive” imported soil, quicklime-treated on-site soils, Class 2 Aggregate Sub-Base (ASB), or Class 2 Aggregate Base (AB). Even with the 12 inches of “non-expansive” material, some movement of exterior slabs may occur. Where concrete flatwork is to be exposed to vehicle traffic, the upper 6 inches of fill should be Class 2

Aggregate Base as specified in the current California Department of Transportation Standard Specifications. This may need to be increased if concrete flatwork is to be exposed to heavy truck traffic. Exterior flatwork will be subjected to edge effects due to the drying out of subgrade soils. Because of the expansive soils, flatwork should have control joints on no greater than 8 feet centers. To protect against edge effects adjacent to unprotected areas, such as vacant or landscaped areas, lateral cutoffs, such as inverted curbs, are recommended. Prior to construction of the flatwork, the upper 12 inches of "non-expansive" fill, quicklime-treated soil, ASB or AB, should be moisture conditioned to near optimum moisture content. If the "non-expansive" fill, ASB or AB is not covered within 30 days after placement, the soils below this material will need to be checked for appropriate moisture of at least 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.

5.3.4 Quad Area Flatwork

We understand that a permeable concrete flatwork may be considered for the new quad area. This design methodology attempts to accommodate portions of surface stormwater runoff through infiltration into the near-surface soils. This design is especially appropriate for coarser-grained soils (sands and gravels). Typically, concrete flatwork designs are based on minimizing the introduction of water into fine-grained soil subgrades. This is because saturated subgrades can begin to yield (pump) over time. By definition, permeable concrete pavements allow saturation of subgrade soils during periods of rainfall or other introduction of surface water. Therefore, there is an increased risk of reduced flatwork life unless otherwise mitigated. If modular pavers are considered for the quad area, the joints between the pavers allow migration of surface water into the soil subgrade.

The quad area flatwork should follow the same recommendations for subgrade preparation as presented in Section 5.3.3. Given the potential mobilization of the lime-treatment operation for the building area, we would also recommend that the quad area be underlain by 12 inches of lime-treated soil, to enhance support and mitigate the potential for reduced design life of quad area due to surface water infiltration. Consideration could also be given to sloping the soil subgrade beneath the pavements towards collection trenches along the perimeter of the quad area.

5.3.5 Effect of Plants on Foundation and Flatwork Performance

Because of the highly expansive nature of the on-site soils, trees and other large plants can significantly contribute to differential settlement of a foundation, flatwork and other paved areas. The roots of trees and large plants can absorb the moisture from the soil, causing the soil to shrink much faster than other soil areas exposed to the weather. The soil where the moisture is lost more rapidly will sink lower than the surrounding soil, causing differential settlement in overlying or adjacent improvements. Certain trees and plants are known to be more hydrophilic (water-loving) than others. Research studies indicate that a tree should be at least as far away from a building as the mature height of the tree to minimize

the effect of drying caused by the tree. A plant and tree specialist should be consulted to avoid the issues described herein.

5.4 Retaining Walls

It is our understanding that low retaining walls may be used in adjacent planting areas or elevator pits. These walls may also be supported on continuous spread footings.

The retaining walls should be designed to resist static earth pressures due to the adjacent soil, and any surcharge effects caused by loads adjacent to the walls. It is recommended that the walls be designed for lateral earth pressures as presented below, which are expressed as equivalent fluid pressures.

LATERAL EARTH PRESSURES FOR MODERATELY EXPANSIVE ON-SITE SOILS WITH BACKFILL SLOPES OF 6 TO 1 (HORIZONTAL TO VERTICAL) OR LESS	
Earth Pressures	**Equivalent Fluid Density, pcf**
Active	45
At-rest	60
Passive (allowable)	300

The passive pressure includes a factor of safety of about 1½.

Walls whose tops are not free to deflect (such as elevator pits) should be designed for an at-rest condition, while an active case can be applied for walls that are free to deflect at the top. These values are unfactored, apply to horizontal backfill, and do not include hydrostatic pressures that might be caused by groundwater or water trapped behind the structure. Retaining walls should be well-drained to reduce hydrostatic pressure. Otherwise, hydrostatic pressure should be included in the design of the walls.

5.5 Demolition

5.5.1 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 Exhibit 1. This process should be observed and tested by a BSK representative.

5.5.2 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 the planned building provided they are cut off and capped at the building perimeters. Pipelines larger than 2 inches in diameter within the planned building should be removed or filled with a 1-sack sand-cement slurry mix. Active utilities to be reused should be carefully located and protected during demolition and during construction.

5.5.3 Existing Trees

Tree stumps and roots over 1 inch in diameter and/or over 3 feet in length should be removed within the building footprint and areas for planned improvements.

5.5.4 Reuse of On-site Concrete and Asphalt Concrete

Existing concrete and asphalt concrete (AC) may be pulverized or broken up and mixed with the underlying base for use as general engineered fill if it meets the following requirements:

GENERAL ENGINEERED FILL GRADATION	
Sieve Size	**Percentage Passing**
3 inch	100 min.
1½ inch	85 min.
No. 200	8 – 40

The processed concrete/AC debris and base material may be used as “non-expansive” fill if the material meets the gradation requirements above and has a plasticity index of 15 or less. The processed asphalt concrete/base material may be used as Class 2 Aggregate Subbase if it meets the following requirements:

CLASS 2 AGGREGATE SUBBASE GRADATION	
Sieve Size	**Percentage Passing**
3 inch	100 min.
2½ inch	90 – 100
No. 4	40 - 90
No. 200	0 – 25
CLASS 2 AGGREGATE SUBBASE QUALITY	
Sand Equivalent	18 min.
R-value	50 min.

The existing concrete or AC pavement may be left in place to provide for a stable staging/work area and for providing drainage relief. We have found that the subgrade soils beneath paved areas tend to be on the wet side and may require additional time to dry. Therefore, if the existing pavement is left as a staging area, then time should be factored into the construction schedule to allow for drying of the subgrade soils once the pavements are removed.

5.6 Earthwork

Earthwork at the site will generally consist of subgrade preparation and placement of concrete slabs and pavements support (including lime treatment), excavation and backfill of demolished foundations, and excavation 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 to 3 feet to create building pads 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 3 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, concrete flatwork, and pavements. In general, soft/loose or unsuitable materials encountered should be overexcavated, removed, and replaced with compacted engineered fill material.

5.6.1 Site Preparation and Grading

Prior to the start of grading and subgrade preparation operations, the site should first be cleared and stripped (minimum of 3 inches deep) to remove all surface vegetation, organic laden topsoil and debris generated during the demolition of existing pavements, concrete slabs and flatwork, foundations, 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.

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 F, Exhibit 1. Scarification and recompaction should extend laterally a minimum of 5 feet beyond the limits of structures and 3 feet beyond flatwork, where achievable.

The net change in volume from the excavation fill source to the compacted engineered fill is referred to as percent shrinkage. The percent shrinkage is defined as follows:

$$\text{Shrinkage} = [\{ (\gamma_d)_f / (\gamma_d)_c \}-1] \times 100\%$$

where $(\gamma_d)_f$ = average dry density of engineered fill

$(\gamma_d)_c$ = average dry density of cut (in-situ soil).

The Shrinkage Factor, SF, is defined as follows:

$$SF = 1 - [\% \text{ shrinkage}/100]$$

We estimate that the shrinkage factor at this site would be about 0.9. All fills should be compacted in lifts of 8-inch maximum uncompacted thickness. A summary of compaction requirements of the projects is presented in Exhibit 1. Laboratory maximum dry density and optimum moisture content relationships should be evaluated based on ASTM Test Designation D1557 (latest edition).

All site preparation and fill placement should be observed by a BSK representative. It is important that, during the stripping and scarification process, our representative 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.

5.6.2 Lime Treatment

Lime-treatment of the in-situ soils (if used) should be performed using high calcium quicklime. Extensive quality control is needed as well as laboratory testing to evaluate the appropriate lime treatment mixture. Our experience has indicated that about 5 percent high calcium quicklime by dry unit weight of the soil is typically needed for treatment. The negative impact of lime-treatment on future vegetation should be considered.

The high calcium quicklime treatment operation should be conducted in general accordance with Section 24 of the Caltrans Standard Specifications, 2010 edition. Quicklime-treatment typically consists of spreading the required amount of quicklime over the area to be treated, followed by initial mixing of the quicklime and water within the soil section to be treated. This initial mixing is then allowed to sit for a period of about 24 hours or longer to permit the resulting chemical reaction to break down the material and change it chemically. Following this “mellowing” period, the soil-quicklime section is re-mixed and additional water, if needed, is added. It is important that adequate water be added before final mixing to ensure complete hydration of the quicklime and to bring the soil moisture content to at least 3 percent above the optimum moisture content before compaction takes place.

After the quicklime-treated pad/subgrade is compacted, it should be allowed to harden (cure) until loaded dump trucks and other construction equipment can operate on it without rutting the surface. Throughout this curing period, the surface of the quicklime-treated soil should be kept moist to aid in strength gain. Alternatively, the quicklime-treated surface can be covered with 4 to 6 inches of capillary break or base rock material.

It is very important that the general steps outlined above be performed in a manner that introduces sufficient water to the soil-quicklime mix to allow the quicklime to thoroughly hydrate and react chemically with the soil subgrade. Likewise, it is equally important that proper curing of the quicklime-treated section take place.

5.6.3 Fill Material

Except for organic laden soil, the on-site soil is 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. Where imported "non-expansive" material is required, it is recommended that it be granular in nature, adhere to the above gradation recommendations and conform to the following minimum criteria:

IMPORTED "NON-EXPANSIVE" FILL CRITERIA	
Plasticity Index	15 or less
Liquid Limit	Less than 30%
% Passing #200 Sieve	8 % – 40%

Highly pervious materials such as pea gravel or clean sands are not recommended because they permit transmission of water to the underlying soils. Prior to transporting proposed import materials to the site, the contractor should make representative samples of the material available to the geotechnical engineer at least 5 working days in advance to allow the engineer 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 Exhibit 1.

Due to the expansive soil content within on-site soils, proper moisture conditioning is important. The moisture conditioning should be performed in accordance with Exhibit 1. Where low expansion potential soils or baserock in paved areas is used, it should immediately be placed over the prepared subgrade to avoid drying of the subgrade. Prior to placement of the capillary break or crushed rock material over the "non-expansive" or lime-treated fill subgrade for the building pad, the subgrade should be moisture conditioned to the moisture content indicated in Exhibit 1. 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.

5.6.4 Weather/Moisture Considerations

If earthwork operations and construction for this project are scheduled to be performed during the rainy season (usually November to May) or in areas containing saturated soils, provisions may be required for drying of soil or providing admixtures, such as lime-treatment, to the soil prior to compaction. Conversely, additional moisture may be required during dry months. Water trucks should be made available in sufficient numbers to provided adequate water during earthwork operations.

Since portions of the site are currently capped with concrete slabs or AC pavement, the moisture content of the subgrade soils in these areas may be significantly above the optimum moisture content. This occurrence is usually caused by the migration of irrigation water from landscaped areas into the aggregate base material and/or the entrapment of subsurface moisture underneath slab and pavement areas. As a result, the subgrade soils may need to be dried prior to undergoing recompaction. It is recommended that any landscape watering in the area be turned off at least two weeks prior to the start of grading activities at the site. If site grading is performed during the rainy months, the site soils could become very wet and difficult to compact without undergoing significant drying. This may not be feasible without delaying the construction schedule. For this reason, drier import soils could be required or lime treating may be needed if construction takes place during winter months.

5.6.5 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.

At the time of this geotechnical investigation, groundwater was encountered in some portions of the site at about a depth of 18½ feet. However, 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.

Backfill for trenches and other small excavations beneath slabs should be compacted as noted in Exhibit 1. 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, lean concrete or a 2-sack sand-cement slurry 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.

5.6.6 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 2013 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.

5.7 Pavements

5.7.1 Asphalt Concrete Pavements

Pavements for this project will mostly consist of asphalt-paved parking and driveways. We have made 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 our R-value testing in the area of the planned parking, we have used an R-value of 5 and we have developed the following recommendations for pavement sections.

Various alternative pavement sections for various different Traffic Indices (TIs) are presented below. 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. Three alternative pavement sections are given for the various TIs in the following table. They include 1) asphalt over baserock, 2) asphalt over baserock over aggregate subbase, and 3) asphalt over baserock over lime-treated soils.

ASPHALT CONCRETE PAVEMENT DESIGN Design R-Value = 5								
	Alternative 1		Alternative 2			Alternative 3		
Traffic index	AC	AB	AC	AB	ASB	AC	AB	LTS
4.0	2.5	7.5	2.5	4.0	4.0	2.5	4.0	12.0
4.5	2.5	9.0	2.5	4.5	5.0	2.5	4.0	12.0
5.0	2.5	11.0	2.5	5.0	6.5	2.5	4.0	12.0
5.5	3.0	12.0	3.0	5.5	7.0	3.0	4.5	12.0
6.0	3.0	13.5	3.0	6.5	8.0	3.0	4.5	12.0
6.5	3.5	14.5	3.5	6.5	9.0	3.0	6.0	12.0

Note: Thicknesses shown are in inches.
AC = Type B Asphalt Concrete
AB = Class 2 Aggregate Base (Minimum R-Value = 78)
ASB = Class 2 Aggregate Subbase (Minimum R-Value = 50)
LTS = Lime-Treated Subgrade (Minimum R-Value = 50)

Since a lime-treating alternative of the building pad and quad area is recommended, the third alternative may be cost effective for the asphalt-paved areas. This alternative, shown above, would consist of lime-treating the existing subgrade prior to placement of the pavement section. This would result in reduced AC and AB sections, as shown in Alternative 3 in the above chart.

5.7.2 Portland Cement Concrete Pavements

PCC pavement should have a minimum thickness of 6 inches supported over 6 inches of Caltrans Class 2 aggregate base compacted per Exhibit 1. Construction joints should be located no more than 10 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 cubic foot (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.

5.7.3 Pavement Drainage

Similar to slabs-on-grade, pavement subgrades will require mitigation of the expansive surface soils. We recommend that pavement subgrades be scarified to a depth of at least 12 inches, moisture conditioned to wet of optimum moisture content and recompacted per Exhibit 1.

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. Concrete curbs should extend a

minimum of 2 inches below the baserock and into the subgrade to provide a barrier against drying of the subgrade soils, or reduction of migration of landscape water, into the pavement section.

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

- All trench backfills, including utility and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade, in accordance with the compaction recommendations in Exhibit 1.
- If Alternative 3 above is selected for the pavement section, wherever lime-treated soil is removed to install utilities inside paved areas, this layer should be either backfilled with imported aggregate subbase or with aggregate base within the design pavement section.
- 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.

5.8 Storm Water Runoff Mitigation

Storm water runoff regulations require pretreatment of runoff and infiltration of storm water to the extent feasible. Typically, this results in the use of bioretention areas, vegetated swales, infiltration trenches, buried storm water detention/infiltration galleries, or permeable pavement near or within parking lots and at the location of roof run-off collection. These features are not well-suited to the moderately to highly expansive clay soils present at this site due to their relatively low permeability[3], which does not allow significant infiltration over short time periods. In addition, allowing water to pond on expansive clay soils can cause the soils to swell, which can cause distress to pavements, slabs, and lightly loaded structures.

Implementation of storm water infiltration criteria will likely result in increased distress and reduced service life of pavement and flatwork if not carefully designed in clay soils. In general, bioretention areas, vegetated swales and infiltration areas should be located in landscaped areas and well away from pavements, buildings, and slopes.

If it is not possible to locate these infiltration systems away from buildings and pavements, alternatives that isolate the infiltrated water, such as flow-through planters, could be considered. When using an

[3] Infiltration testing and/or laboratory permeability testing was not performed due to the dominant presence of highly plastic clays blanketing the site.

infiltration system in clay soils, underdrains should be used. In addition, the top of the swales should be a laterally separated a minimum of 12 inches from the curbs. To reduce potential for rotation of the curbs, curbs adjacent to the swales should extend a minimum of 12 inches below the bottom of the aggregate base course.

Due to the potential adverse effects on project performance, we should review the geotechnical aspect of the storm water infiltration system and its location.

5.9 Corrosivity Results

A soil sample was collected during our field investigation at a depth of approximately 1½ feet below the ground surface in boring B-4 and was submitted for corrosion testing. The sample was tested by CERCO Analytical, a State-certified laboratory in Concord, California, for redox potential, pH, resistivity, chloride content, and sulfate content in accordance with ASTM test methods. The test results are presented at the end of Appendix E. Also included is the evaluation by CERCO Analytical of the corrosion test results. Because we are not corrosion specialists, we recommend that a corrosion specialist be consulted for advice on proper corrosion protection for underground piping which will be in contact with the soils and other design details.

Based upon the resistivity measurements, the sample tested is classified as "corrosive" by CERCO Analytical. They recommend 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 all buried metallic pressure piping, such as ductile iron firewater pipelines, should be protected against corrosion.

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. For instance, any imported soil materials should not be any more corrosive than the onsite soils and should not be classified as being more corrosive than "moderately corrosive." Also, onsite cutting and filling may result in soils contacting concrete that were not anticipated at the time of this investigation.

5.10 Plan Review and Construction Observation

We also 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.

6. ADDITIONAL SERVICES AND LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of BSK's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. BSK makes no other representation, guarantee or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the District (Client) and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

The work performed was based on project information provided by the Client. If the Client does not retain BSK to review any plans and specifications, including any revisions or modifications to the plans and specifications, BSK assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, the Client must obtain written approval from BSK's engineer that such changes do not affect our recommendations. Failure to do so will vitiate BSK's recommendations.

The scope of services was limited to drilling and sampling nine borings and advancing three CPTs at the site, laboratory testing, and preparation of this recommendations report. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on subsurface exploration including 9 borings drilled to a maximum depth of 30 feet, 3 CPTs to a depth of 50 feet, groundwater level measurements, laboratory testing, and engineering analyses.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil or groundwater conditions could vary beyond the point explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that BSK is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated structure loads, and the design depths or locations of the foundations, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by BSK.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, BSK should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation

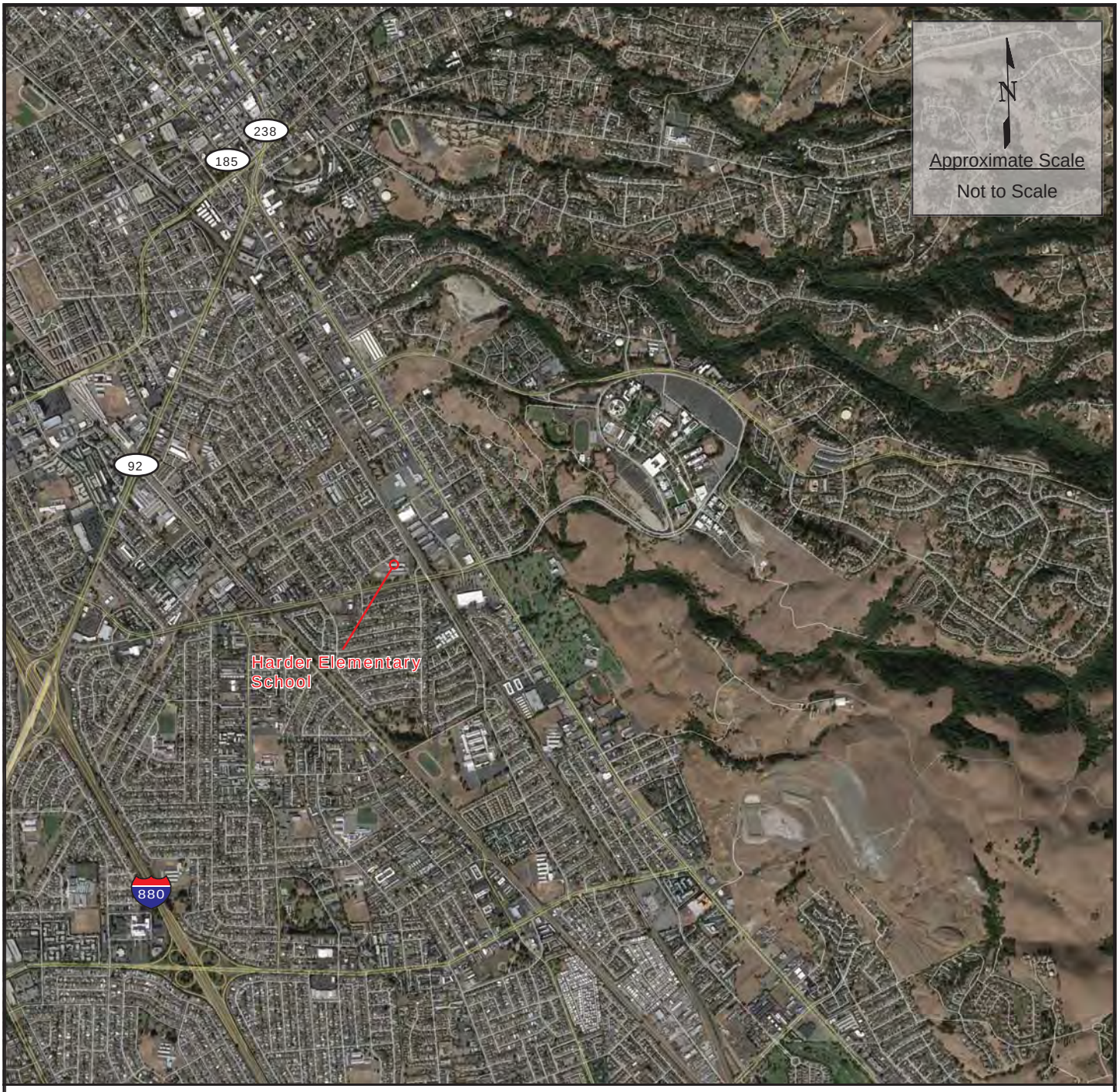
of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions are encountered. As a minimum BSK should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications;
- Observe and evaluate the site earthwork operations to confirm subgrade soils are suitable for construction of foundations, slabs-on-grade, pavements and placement of engineered fill;
- Confirm engineered fill for the structure and other improvements is placed and compacted per the project specifications; and
- Observe shallow foundation and drilled pier excavations to confirm conditions are as anticipated.

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

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that BSK's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems that may arise during earthwork and foundation construction.

PLATES



References: 1. http://earth.google.com, 2016

Note: Locations are approximate

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BSK Associates Engineers & Laboratories

PROJECT NO. G16-054-11L	VICINITY MAP	PLATE
DRAWN: 4/21/16		1
DRAWN BY: D. Tower		
CHECKED BY: B. Steen	Campus Reconstruction Harder Elementary School 495 Wyeth Road Hayward, California	
FILE NAME: Lab.indd		

References

1. http://earth.google.com, 2016
2. "Preliminary Plans and 3d View.pdf" by Lionakis dated April 29, 2016.

Legend

B-1 — Approx. Soil Boring Location

CPT-3 — Approx. CPT Location

R-1 — Resistance (R)-Value Sample Location

The information included on this graphic representation has been compiled from a variety of sources and is subject to change without notice. BSK makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. This document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.

BSK ASSOCIATES

PROJECT NO. G16-054-11L	SITE IMPROVEMENT PLAN	PLATE
DRAWN: 07/20/16		2
DRAWN BY: B. Steen		
CHECKED BY: C. Foulk	Campus Reconstruction Harder Elementary School 495 Wyeth Road Hayward, California	
FILE NAME: SitePlan.indd		

References

1. http://earth.google.com, 2016
2. "Preliminary Plans and 3d View.pdf" by Lionakis dated April 29, 2016.

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Legend

B-1 Approx. Soil Boring Location

CPT-3 Approx. CPT Location

R-1 Resistance (R)-Value Sample Location

BSK ASSOCIATES

PROJECT NO.	G16-054-11L
DRAWN:	07/20/16
DRAWN BY:	B. Steen
CHECKED BY:	C. Foulk
FILE NAME:	SitePlan.indd

SITE EXPLORATION PLAN

Campus Reconstruction
Harder Elementary School
495 Wyeth Road
Hayward, California

PLATE

3

APPENDIX A

BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487/2488)

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

BSK Associates Engineers & Laboratories

PROJECT NO. G16-057-11L	UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487/2488)
DRAWN: 4/28/16	
DRAWN BY: D, Tower	
CHECKED BY: B. Steen	Campus Reconstruction Harder Elementary School 495 Wyeth Road Hayward, California
FILE NAME: Legend.indd	

APPENDIX A

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

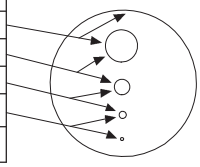
CONSISTENCY	ABBR	FIELD TEST
Very Soft	VS	Thumb will penetrate soil more than 1 in. (25 mm)
Soft	S	Thumb will penetrate soil about 1 in. (25 mm)
Firm	F	Thumb will indent soil about 1/4 in. (6 mm)
Hard	H	Thumb wil not indent soil but readily indented with thumbnail
Very Hard	VH	Thumbnail will not indent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

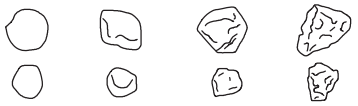
REACTION WITH HCL

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

LOG SYMBOLS

	BULK / BAG SAMPLE	-4	PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)
	SPLIT BARREL SAMPLER (2-1/2 inch outside diameter)	-200	PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)
	SPLIT BARREL SAMPLER (3 inch outside diameter)	LL	LIQUID LIMIT (ASTM Test Method D 4318)
	STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)	PI	PLASTICITY INDEX (ASTM Test Method D 4318)
	CONTINUOUS CORE	TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION (EM 1110-1-1906)/ASTM Test Method D 2850
	SHELBY TUBE	EI	EXPANSION INDEX (UBC STANDARD 18-2)
	ROCK CORE	COL	COLLAPSE POTENTIAL
	GROUNDWATER LEVEL (encountered at time of drilling)	UC	UNCONFINED COMPRESSION (ASTM Test Method D 2166)
	GROUNDWATER LEVEL (measured after drilling)		
	SEEPAGE	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.

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: 925-315-3151
Fax: 925-315-3152

LOG OF BORING NO. B-1

Project Name: **Harder Elementary School**
Project Number: **G16-054-11L**
Project Location: **495 Wyeth Rd**
Logged by: **M. McNally**
Checked by: **C. Foulk**

Surface El.: **57 ft.**
Location: **Harder E.S**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		LEAN CLAY WITH SAND (CH): greenish black, moist, firm to hard, fine grained sand, medium to high plasticity										
		TXUU (see Plate C-2) c=1,900 psf		1A 1B 1C	4 6 9	2.5		97	26			
5		greenish black mottled with olive gray, hard		2A 2B 2C	7 10 16	3.75		104	22			
10		yellowish brown, increase in fine sands, iron oxide staining		3A 3B 3C	8 12 16	3.75						
15		calcium carbonate nodules		4A 4B 4C	7 12 17	>4.5		107	20			
		SANDY LEAN CLAY (CL): brown, wet, firm to hard, medium plasticity, fine to coarse grained sand										
				5A 5B 5C	6 9 11	3.75 1.25						
20		Boring terminated at approximately 20 feet. Free groundwater was observed at approximately 18.5 feet. Boring was backfilled with cement grout.										
25												
30												

Completion Depth:	20.0	**Drilling Equipment:**	Exploration GeoServices Mobile B-40
Date Started:	3/31/16	**Drilling Method:**	Hollow Stem
Date Completed:	3/31/16	**Drive Weight:**	140 lbs
California Sampler:	2.5-inch inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-inch inner diameter	**Drop:**	30-in
		Remarks:	

LOG OF BORING NO. B-2

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: 925-315-3151
Fax: 925-315-3152

Project Name: **Harder Elementary School**
Project Number: **G16-054-11L**
Project Location: **495 Wyeth Rd**
Logged by: **M. McNally**
Checked by: **C. Foulk**

Surface El.: **59 ft.**
Location: **Harder E.S**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		SILT WITH SAND (ML): yellowish brown mottled with reddish brown, slightly moist, soft, medium plasticity,										
				1A	7	3.0						
		SILTY CLAY WITH SAND (CH): dark brown, moist, medium to high plasticity, firm to hard, fine grained sand TXUU (see Plate C-2) c=1,750 psf		1B	6			95	26			
				1C	9							
5				2A	5	2.75						
				2B	10							
				2C	18							
				3A	6	2.5						
				3B	12							
10		brown, increase in fine sand content, roots encountered		3C	13							
				4A	8	3.0						
		yellowish brown mottled with dark brown, slightly moist, fine to coarse sand, iron oxide staining, porous		4B	16	4.5						
15				4C	20							
				5A	4	2.5						
				5B	7			98	22			
20				5C	13							
		SANDY LEAN CLAY WITH GRAVEL (CL): yellowish brown, wet pockets, fine to coarse sand, fine subangular gravel		6A	6							
				6B	12		51			41	16	25
25				6C	16							
		CLAYEY SAND WITH GRAVEL (SC): yellowish brown, very moist, dense, fine to coarse sand, fine subrounded gravel										
				7A	10							
				7B	17							
30				7C	18							
		Boring terminated at approximately 30 feet. No free groundwater was observed. Boring was backfilled with cement grout.										

Completion Depth:	30.0	**Drilling Equipment:**	Exploration GeoServices Mobile B-40
Date Started:	3/31/16	**Drilling Method:**	Hollow Stem
Date Completed:	3/31/16	**Drive Weight:**	140 lbs
California Sampler:	2.5-inch inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-inch inner diameter	**Drop:**	30-in
		Remarks:	

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: 925-315-3151
Fax: 925-315-3152

LOG OF BORING NO. B-3

Project Name: **Harder Elementary School**
Project Number: **G16-054-11L**
Project Location: **495 Wyeth Rd**
Logged by: **M. McNally**
Checked by: **C. Foulk**

Surface El.: **58 ft.**
Location: **Harder E.S**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		ASPHALT: approximately 2.5-inches of asphalt										
		CLAYEY SAND WITH GRAVEL (SC): yellowish brown, slightly moist, dense, fine to coarse sand, fine subrounded gravel, fill-like material		1A	4	2.25						
		SILTY FAT CLAY WITH SAND (CH): greenish black mottled with gray, moist, medium to high plasticity, soft to firm, fine grained sand		1B 1C	5 9					61	21	40
5				2A 2B 2C	8 12 16	3.0		97	27			
		SILTY LEAN CLAY WITH SAND (CL): brown, moist, hard, low plasticity, increase in fine grained sand		3A 3B 3C	5 10 13	4.25						
10												
		medium plasticity, increased clay content, iron oxide staining		4A 4B 4C	6 18 23	4.0		108	21			
15												
		yellowish brown, increased fine grained sand content		5A 5B 5C	6 12 13	2.25 3.0						
20		Boring terminated at approximately 20 feet. Free groundwater was observed at approximately 18.5 feet. Boring was backfilled with cement grout.										
25												
30												

Completion Depth:	20.0	**Drilling Equipment:**	Exploration GeoServices Mobile B-40
Date Started:	3/31/16	**Drilling Method:**	Hollow Stem
Date Completed:	3/31/16	**Drive Weight:**	140 lbs
California Sampler:	2.5-inch inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-inch inner diameter	**Drop:**	30-in
		Remarks:	

GEO_TARGET HARDER E.S GINT BORE LOGS.GPJ GEOTECHNICAL 08.GDT 7/21/16

LOG OF BORING NO. B-4

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: 925-315-3151
Fax: 925-315-3152

Project Name: **Harder Elementary School**
Project Number: **G16-054-11L**
Project Location: **495 Wyeth Rd**
Logged by: **M. McNally**
Checked by: **C. Foulk**

Surface El.: **56 ft.**
Location: **Harder E.S**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		SILTY LEAN CLAY WITH SAND (CH): dark brown, slightly moist, soft, medium to high plasticity, fine grained sand, skeleton roots observed										
				1A 1B 1C	3 7 8	1.25 1.5		96	26			
5		firm TXUU (see Plate C-2) c=2,500 psf		2A 2B 2C	8 8 16	3.75 4.0		104	20			
		brown, increase in fine sand		3A 3B 3C	10 13 20	>4.5		108	21			
10												
		fine to coarse sand		4A 4B 4C	9 14 21	>4.5						
15												
		CLAYEY SAND (SC): yellowish brown, wet, medium dense, fine to coarse grained sand		5A 5B 5C	5 6 11	2.75 .75						
20												
				6A 6B 6C	8 10 11							
25		up to 1-inch fine subangular gravel										
		iron oxide staining		7A 7B 7C	6 10 12							
30		Boring terminated at approximately 30 feet. Free groundwater was observed at approximately 19.5 feet. Boring was backfilled with cement grout.										

Completion Depth:	30.0	**Drilling Equipment:**	Exploration GeoServices Mobile B-40
Date Started:	3/31/16	**Drilling Method:**	Hollow Stem
Date Completed:	3/31/16	**Drive Weight:**	140 lbs
California Sampler:	2.5-inch inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-inch inner diameter	**Drop:**	30-in
		Remarks:	

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: 925-315-3151
Fax: 925-315-3152

LOG OF BORING NO. B-5

Project Name: **Harder Elementary School**
Project Number: **G16-054-11L**
Project Location: **495 Wyeth Rd**
Logged by: **M. McNally**
Checked by: **C. Foulk**

Surface El.: **55 ft.**
Location: **Harder E.S**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		SILTY LEAN CLAY (CH): dark brown, slightly moist, hard, medium to high plasticity										
				1A 1B 1C	5 9 12	3.75						
5				2A 2B 2C	7 10 17	3.5 4.25		108	21			
		traces of fine grained sand, old roots found										
				3A 3B 3C	6 9 13	4.5		108	20			
10		fine and coarse grained sand, fine gravel up to 1/2-inch in diameter TXUU (see Plate C-2) c=4,200 psf										
		CLAYEY SAND WITH GRAVEL (SC): yellowish brown and multicolored, slightly moist, medium dense, low plasticity, fine grained sand, fine gravel up to 1/2-inch		4A 4B 4C	12 16 17			113	15			
15												
		SANDY LEAN CLAY WITH GRAVEL (CL): brown, slightly moist, hard, medium plasticity, fine to coarse snad, fine subrounded gravel up to 1/4-inch in diameter		5A 5B 5C	7 9 13							
20		Boring terminated at approximately 20 feet. No free groundwater was observed. Boring was backfilled with cement grout.										
25												
30												

Completion Depth:	20.0	**Drilling Equipment:**	Exploration GeoServices Mobile B-40
Date Started:	3/31/16	**Drilling Method:**	Hollow Stem
Date Completed:	3/31/16	**Drive Weight:**	140 lbs
California Sampler:	2.5-inch inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-inch inner diameter	**Drop:**	30-in
		Remarks:	

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: 925-315-3151
Fax: 925-315-3152

LOG OF BORING NO. B-6

Project Name: **Harder Elementary School**
Project Number: **G16-054-11L**
Project Location: **495 Wyeth Rd**
Logged by: **M. McNally**
Checked by: **C. Foulk**

Surface El.: **59 ft.**
Location: **Harder E.S**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		ASPHALT: approximately 3.5-inches of asphalt										
		SANDY FAT CLAY (CH): dark brown, slightly moist, firm to hard, medium to high plasticity, fine grained sand										
				1A 1B 1C	5 10 13	3.75				51	17	34
5		iron oxide staining		2A 2B 2C	8 11 18	4.25						
		SANDY LEAN CLAY (CL): yellowish brown, slightly moist, hard, medium to high plasticity, fine to coarse grained sand										
				3A 3B 3C	7 10 15	4.5						
10												
		moist, increased fine grained sand content		4A 4B 4C	9 12 13	3.25 2.5						
15												
		very moist, firm		5A 5B 5C	9 10 10	1.5		100	26			
20												
		increasing sand content		6A 6B 6C	7 11 16							
25												
				7A 7B 7C	6 8 12	2.25		104	24			
30		Boring terminated at approximately 30 feet. Free groundwater was observed at approximately 20 feet. Boring was backfilled with cement grout.										

Completion Depth:	30.0	**Drilling Equipment:**	Exploration GeoServices Mobile B-40
Date Started:	3/31/16	**Drilling Method:**	Hollow Stem
Date Completed:	3/31/16	**Drive Weight:**	140 lbs
California Sampler:	2.5-inch inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-inch inner diameter	**Drop:**	30-in
		Remarks:	

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: 925-315-3151
Fax: 925-315-3152

LOG OF BORING NO. B-7

Project Name: **Harder Elementary School**
Project Number: **G16-054-11L**
Project Location: **495 Wyeth Rd**
Logged by: **C. Foulk**
Checked by: **D. Tower**

Surface El.: **57 ft.**
Location: **Harder E.S**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		FAT CLAY WITH SAND (CH): greenish black mottled with dark reddish brown, slightly moist, very hard, high plasticity, fine grained sand, slightly porous, trace roots some 1-inch subrounded gravel		1A 1B 1C	14 18 30	>4.5						
5		**LEAN CLAY WITH SAND (CL):** dark reddish brown, slightly moist, firm to hard, medium plasticity, fine grained sand content		2A 2B 2C	13 18 18	3.0						
10		multi-colored, fine to coarse grained sand, fine subrounded gravel up to 1/2-inch, cemented		3A 3B 3C	6 10 14	4.5						
15		medium to high plasticity, less cemented, decreased sand content, fine grained sand		4A 4B 4C	13 16 18	2-2.5						
20				5A 5B 5C	10 12 14							
		Boring terminated at approximately 20 feet. No free groundwater was observed. Boring was backfilled with cement grout.										
25												
30												

Completion Depth:	20.0	**Drilling Equipment:**	Exploration GeoServices Mobile B-40
Date Started:	6/29/16	**Drilling Method:**	Hollow Stem
Date Completed:	6/29/16	**Drive Weight:**	140 lbs
California Sampler:	2.5-inch inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-inch inner diameter	**Drop:**	30-in
		Remarks:	

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: 925-315-3151
Fax: 925-315-3152

LOG OF BORING NO. B-8

Project Name: **Harder Elementary School**
Project Number: **G16-054-11L**
Project Location: **495 Wyeth Rd**
Logged by: **C. Foulk**
Checked by: **D. Tower**

Surface El.: **58 ft.**
Location: **Harder E.S**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		ASPHALT: approximately 2 inches										
		AGGREGATE BASEROCK: approximately 2 inches										
		FAT CLAY (CH): greenish black mottled with dark reddish brown, moist, hard, high plasicity										
				1A 1B 1C	8 9 12	3.5						
5				2A 2B 2C	8 9 12	3.5						
		dark brown mottled with dark reddish brown, firm to hard										
		LEAN CLAY WITH SAND (CL): dark reddish brown mottled with olive yellow, moist, hard, medium plasticity, fine to coarse grained sand, cemented		3A 3B 3C	10 14 18	3.5-4.5						
10												
		increase in fine to coarse grained sand, coarse subrounded gravel up to 1-inch round		4A 4B 4C	5 9 10	2.5						
15												
				5A 5B 5C	11 15 16	>4.5						
20		Boring terminated at approximately 20 feet. No free groundwater was observed. Boring was backfilled with cement grout.										
25												
30												

Completion Depth:	20.0	**Drilling Equipment:**	Exploration GeoServices Mobile B-40
Date Started:	6/29/16	**Drilling Method:**	Hollow Stem
Date Completed:	6/29/16	**Drive Weight:**	140 lbs
California Sampler:	2.5-inch inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-inch inner diameter	**Drop:**	30-in
		Remarks:	

LOG OF BORING NO. B-9

BSK Associates
324 Earhart Way
Livermore, CA 94551
Telephone: 925-315-3151
Fax: 925-315-3152

Project Name: **Harder Elementary School**
Project Number: **G16-054-11L**
Project Location: **495 Wyeth Rd**
Logged by: **C. Foulk**
Checked by: **D. Tower**

Surface El.: **58 ft.**
Location: **Harder E.S**

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		ASPHALT: approximately 2 inches										
		AGGREGATE BASEROCK: approximately 2 inches										
		FAT CLAY WITH SAND (CH): greenish black mottled with dark reddish brown, moist, firm to hard, medium to high plasticity, fine grained sand, iron oxide staining		1A 1B 1C	8 11 12	2-2.5						
5		dark reddish brown, increased in coarse grained sand content		2A 2B 2C	7 9 12	3-3.5						
		SANDY LEAN CLAY (CL): dark reddish brown, moist, hard, low to medium plasicity, fine to coarse grained sand, cemented-like structure		3A 3B 3C	17 22 27	>4.5						
10												
		firm, high plasticity, decrease in sand content		4A 4B 4C	7 11 13	2.5						
15												
		yellowish brown, hard, fine subrounded gravel up to 1/4-inch round		5A 5B 5C	11 14 16	3.75-4						
20		Boring terminated at approximately 20 feet. No free groundwater was observed. Boring was backfilled with cement grout.										
25												
30												

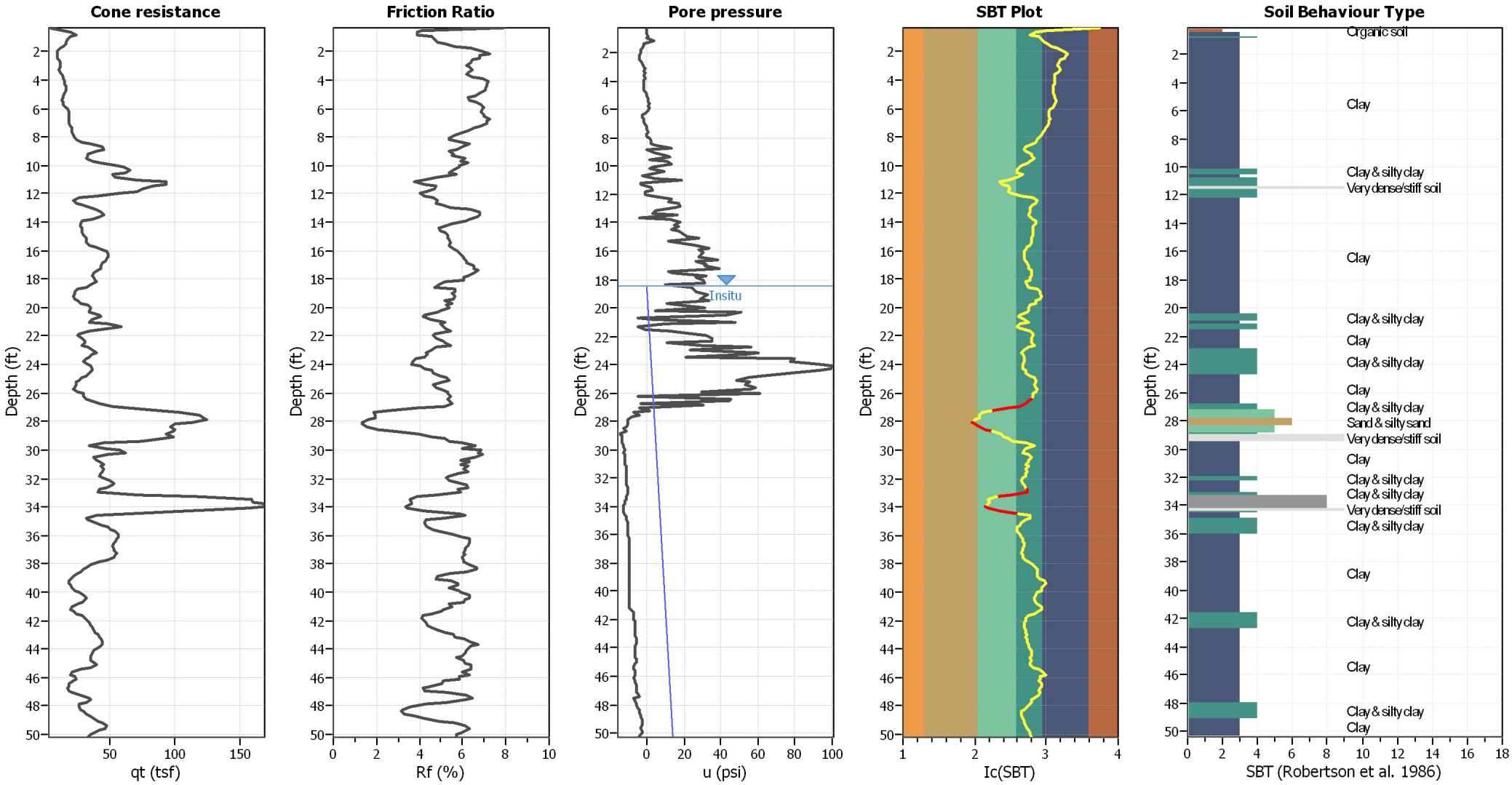
Completion Depth:	20.0	**Drilling Equipment:**	Exploration GeoServices Mobile B-40
Date Started:	6/29/16	**Drilling Method:**	Hollow Stem
Date Completed:	6/29/16	**Drive Weight:**	140 lbs
California Sampler:	2.5-inch inner diameter	**Hole Diameter:**	8-in
SPT Sampler:	1.4-inch inner diameter	**Drop:**	30-in
		Remarks:	

APPENDIX B

CONE PENETROMETER TEST RESULTS

AND LIQUEFACTION ANALYSES

CPT basic interpretation plots



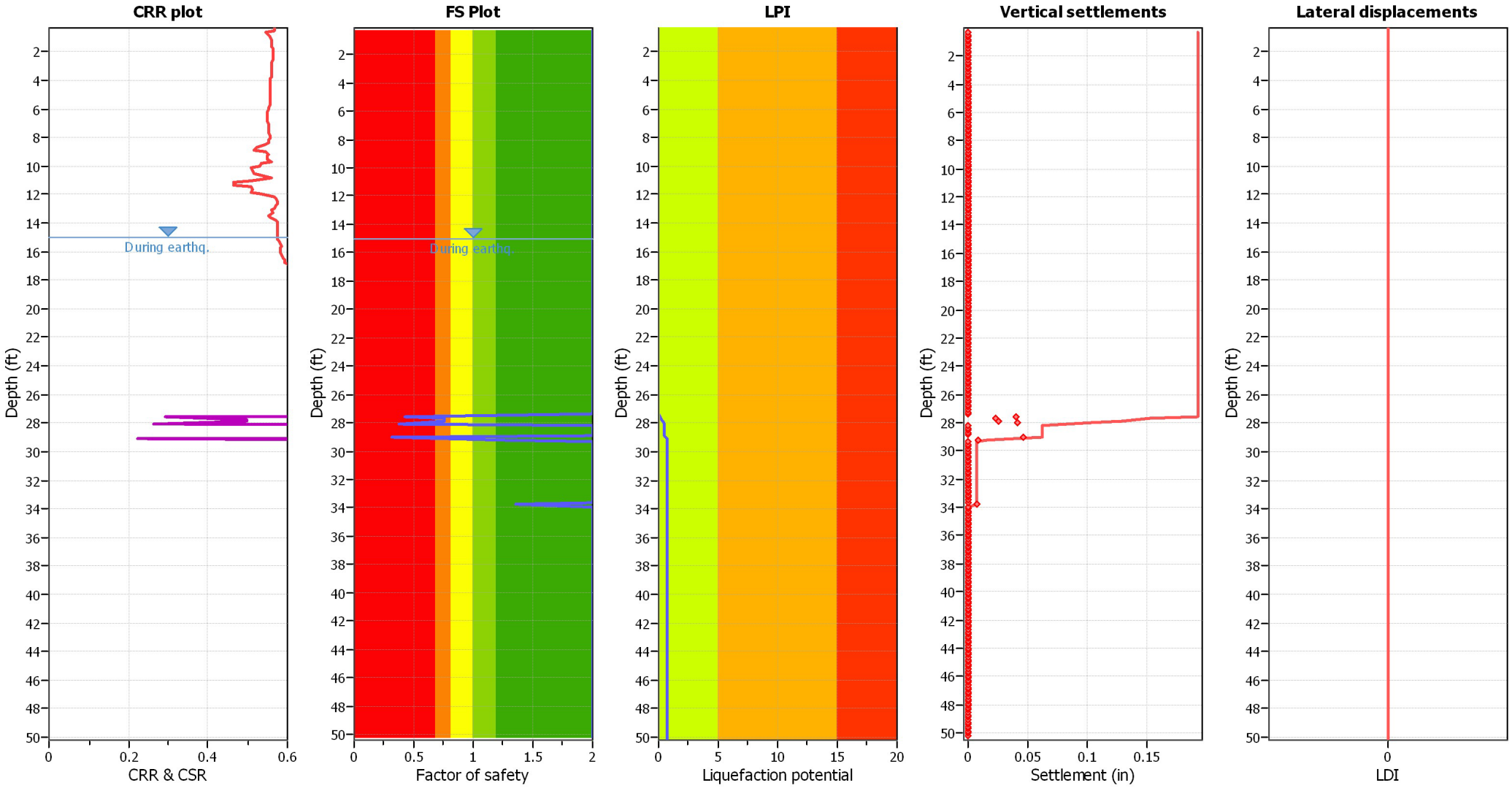
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	15.00 ft	Fill weight:	N/A
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.98	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	18.50 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



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	15.00 ft	Fill weight:	N/A
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.98	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	18.50 ft	Fill height:	N/A	Limit depth:	N/A

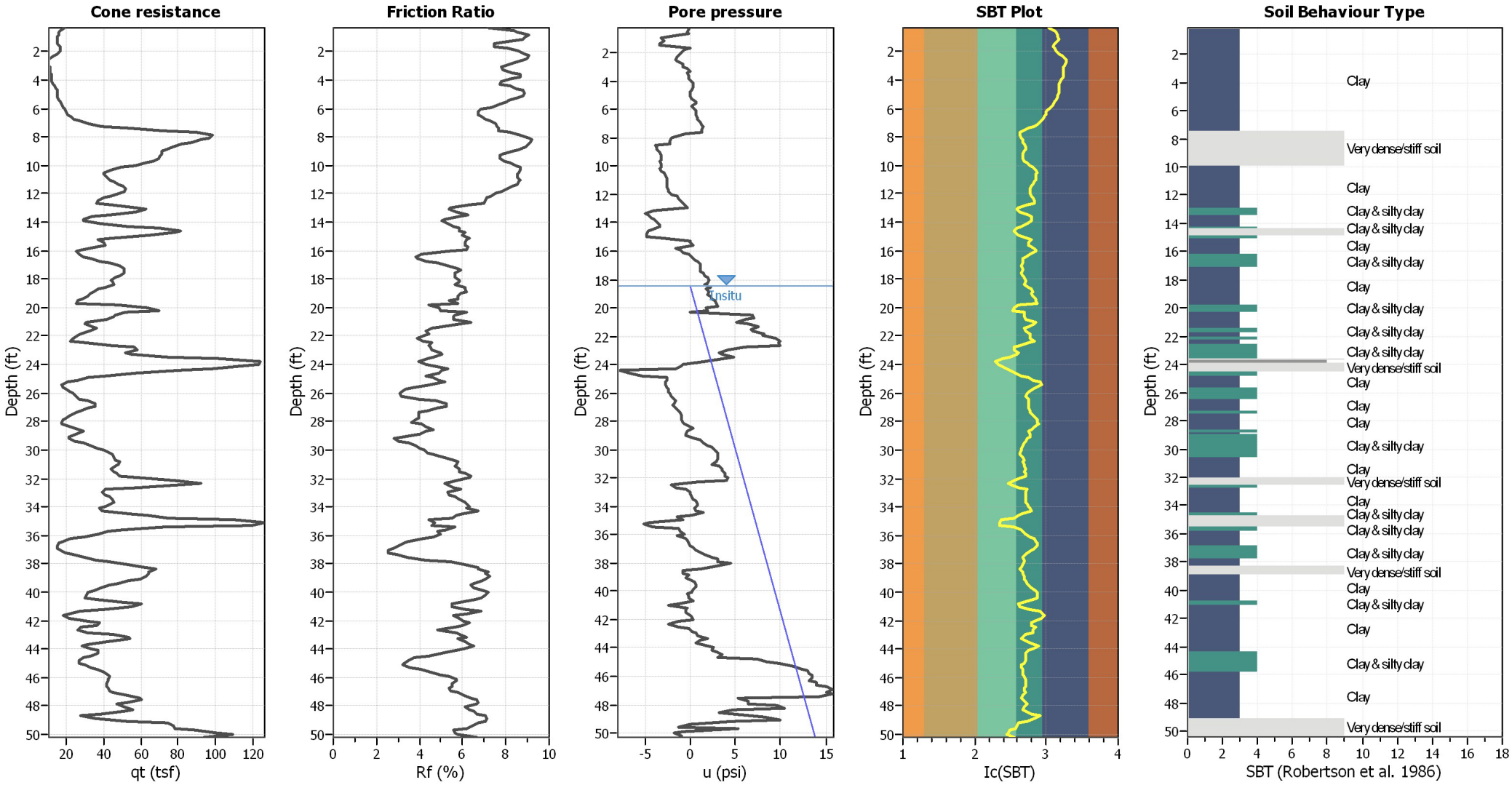
F.S. color scheme

- 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

CPT basic interpretation plots



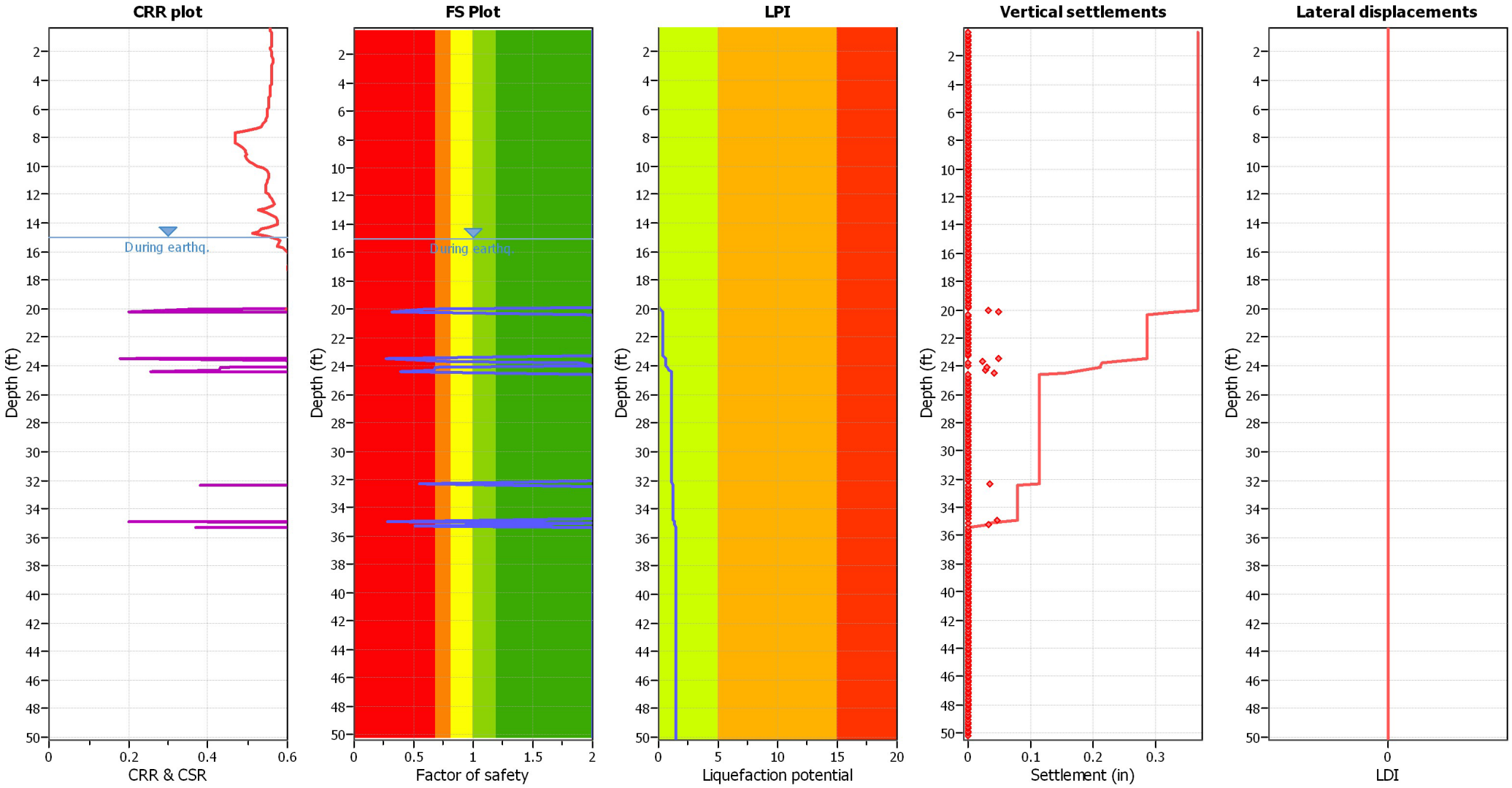
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	15.00 ft	Fill weight:	N/A
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.98	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	18.50 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



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	15.00 ft	Fill weight:	N/A
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.98	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	18.50 ft	Fill height:	N/A	Limit depth:	N/A

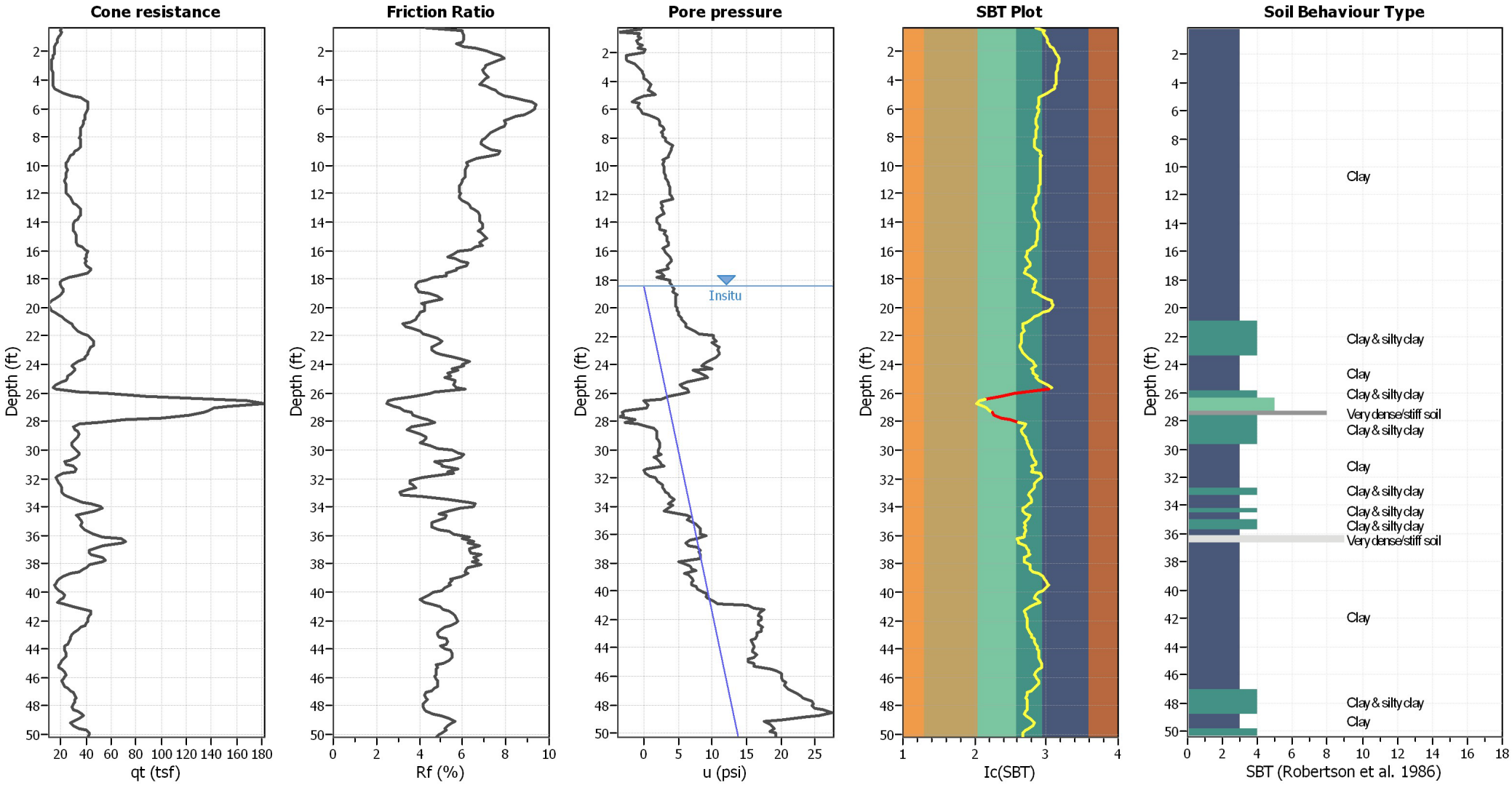
F.S. color scheme

- 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

CPT basic interpretation plots



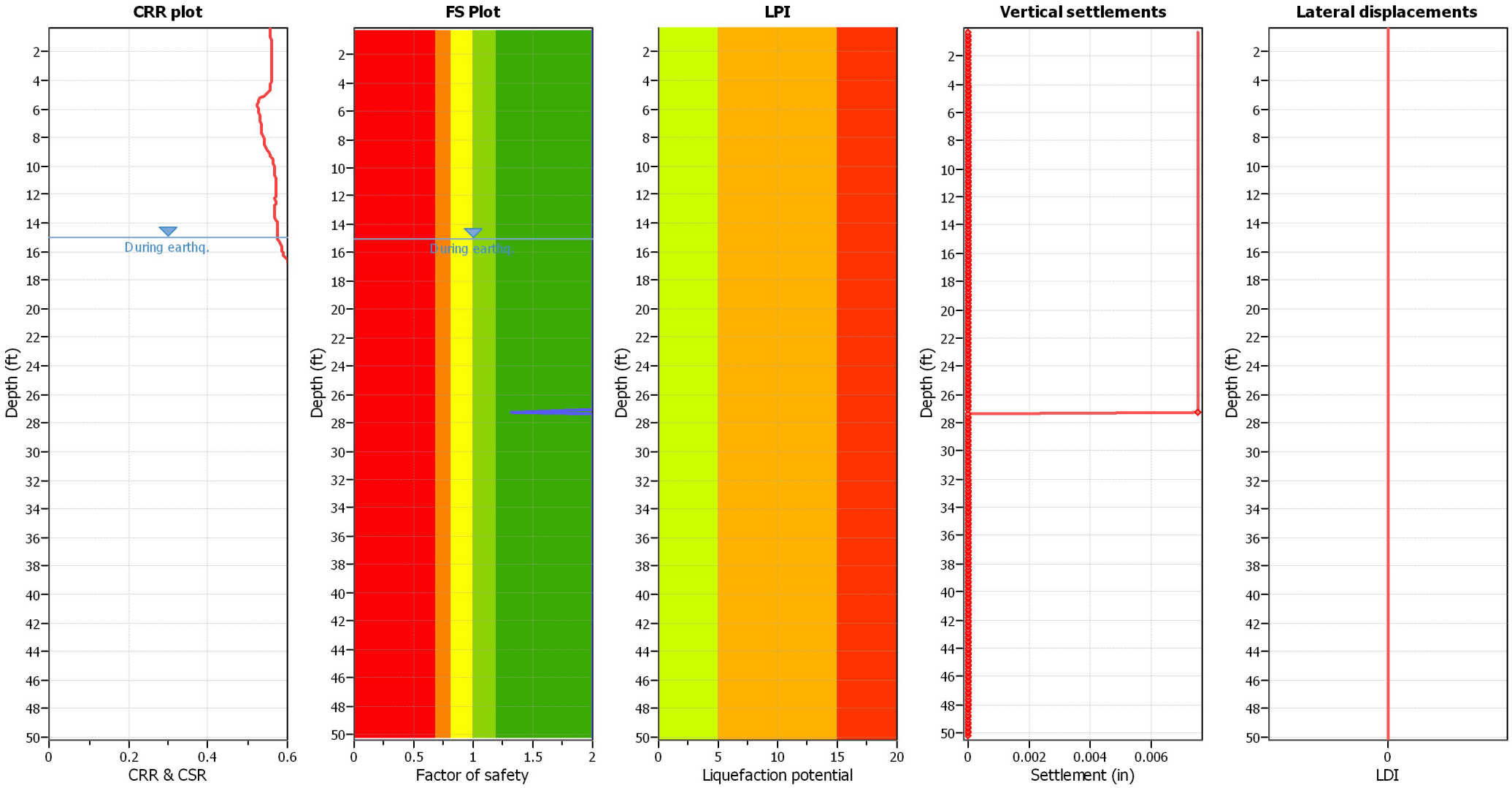
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	15.00 ft	Fill weight:	N/A
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.98	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	18.50 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



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	15.00 ft	Fill weight:	N/A
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:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.98	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	18.50 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

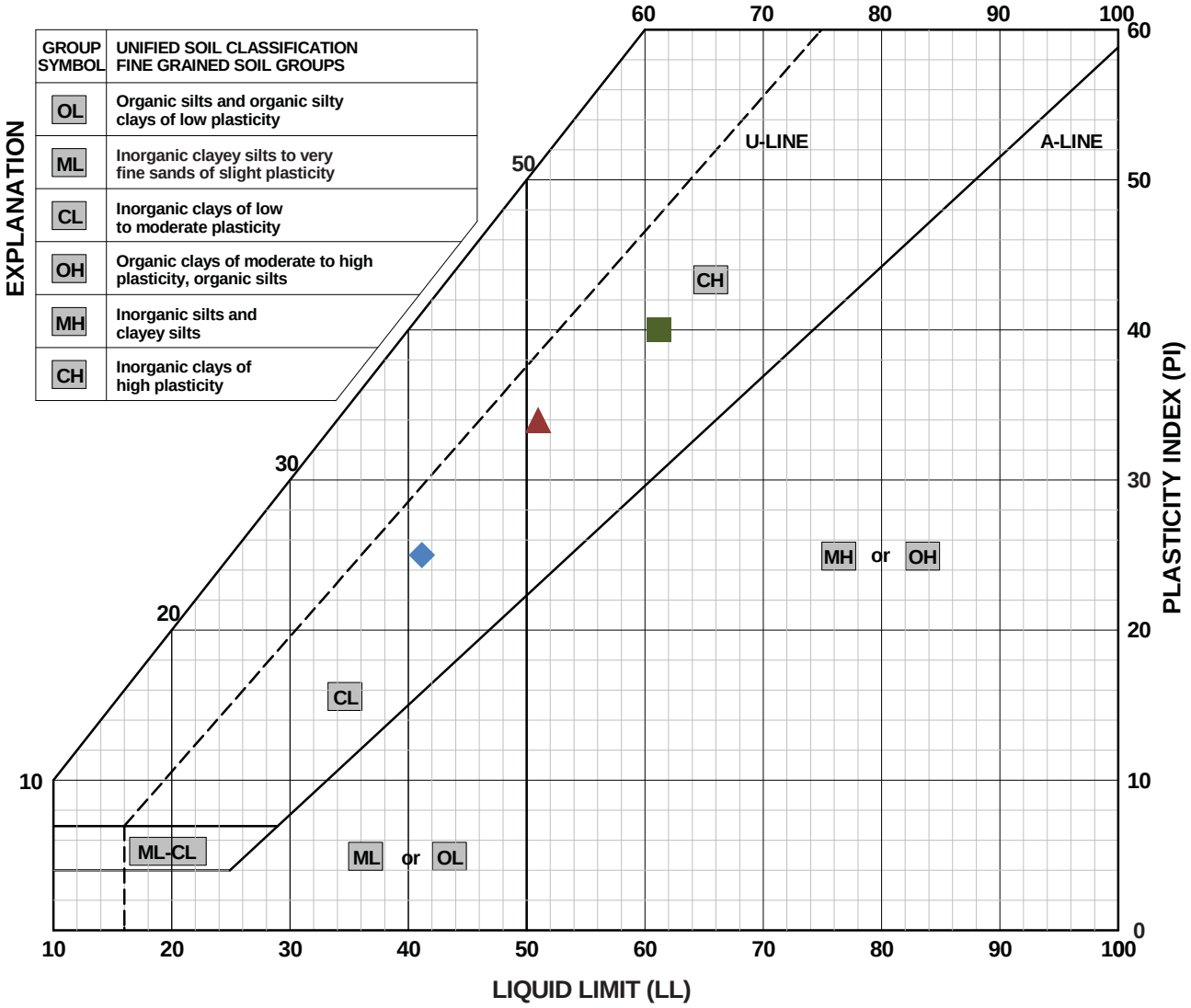
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

APPENDIX C

LABORATORY TEST RESULTS



LEGEND:	SOURCE	DEPTH (ft)	LL	PL	PI	DESCRIPTION
◆	B-2	24.5	41	16	25	Yellowish Brown Sandy Lean Clay (CL)
■	B-3	3.0	61	21	40	Greenish Black Silty Fat Clay (CH)
▲	B-6	3.0	51	17	34	Dark Brown Sandy Fat Clay (CH)

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BSK Associates Engineers & Laboratories

PROJECT NO. G16-054-11L
DRAWN: 4/21/16
DRAWN BY: D. Tower
CHECKED BY: B. Steen
FILE NAME: Lab.indd

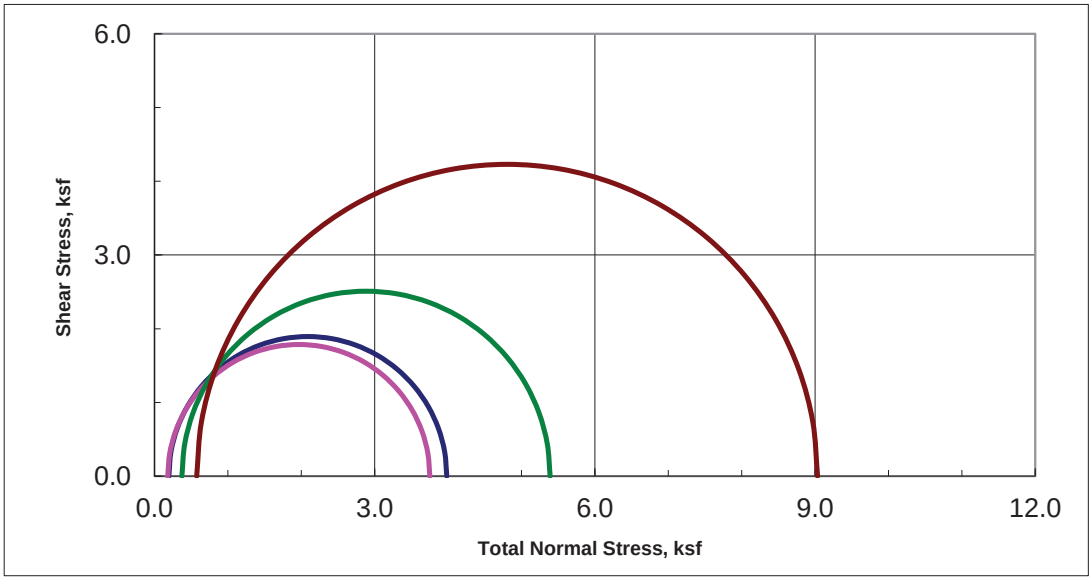
ATTERBERG LIMIT

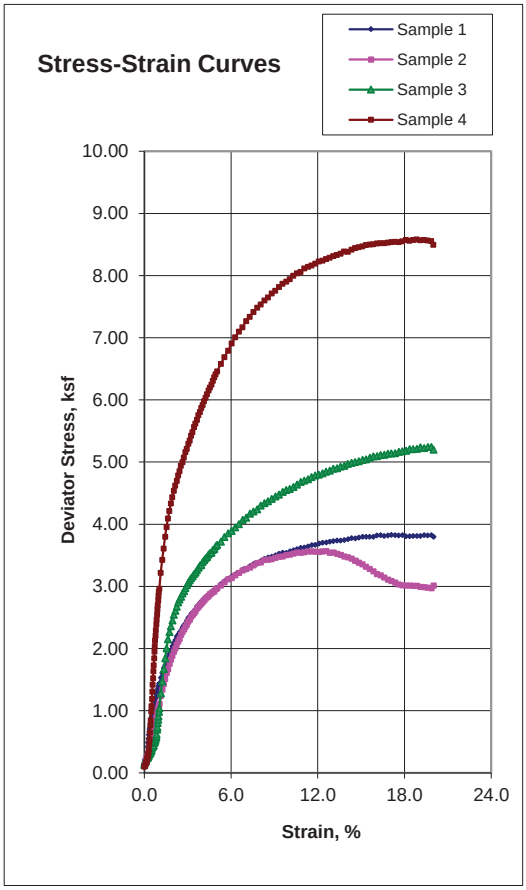
Campus Reconstruction
Harder Elementary School
495 Wyeth Road
Hayward, California

PLATE
C-1

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	25.7	26.0	22.9	19.6
Dry Den,pcf	96.9	95.1	103.9	108.3
Void Ratio	0.740	0.772	0.622	0.556
Saturation %	93.6	90.9	99.5	95.0
Height in	5.04	5.16	5.15	5.18
Diameter in	2.40	2.41	2.40	2.40
Cell psi	1.4	1.3	2.6	4.0
Strain %	15.00	12.58	15.00	15.00
Deviator, ksf	3.787	3.571	5.017	8.459
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.050	0.051	0.051	0.052
Job No.:	664-067			
Client:	BSK Associates			
Project:	G16-054-11L			
Boring:	B-1	B-2	B-4	B-5
Sample:	1c	1c	2c	3c
Depth ft:	3	3	6	9.5

Visual Soil Description	
Sample #	
1	Greenish Black CLAY w/ Sand
2	Dark Brown Silty CLAY w/ Sand
3	Dark Brown Silty CLAY w/ Sand
4	Dark Brown Silty CLAY w/ Sand

Remarks:

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

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BSK Associates Engineers & Laboratories

PROJECT NO.	G16-054-11L
DRAWN:	4/21/16
DRAWN BY:	D. Tower
CHECKED BY:	B. Steen
FILE NAME:	Lab.indd

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION

Campus Reconstruction
Harder Elementary School
495 Wyeth Road
Hayward, California

PLATE

C-2

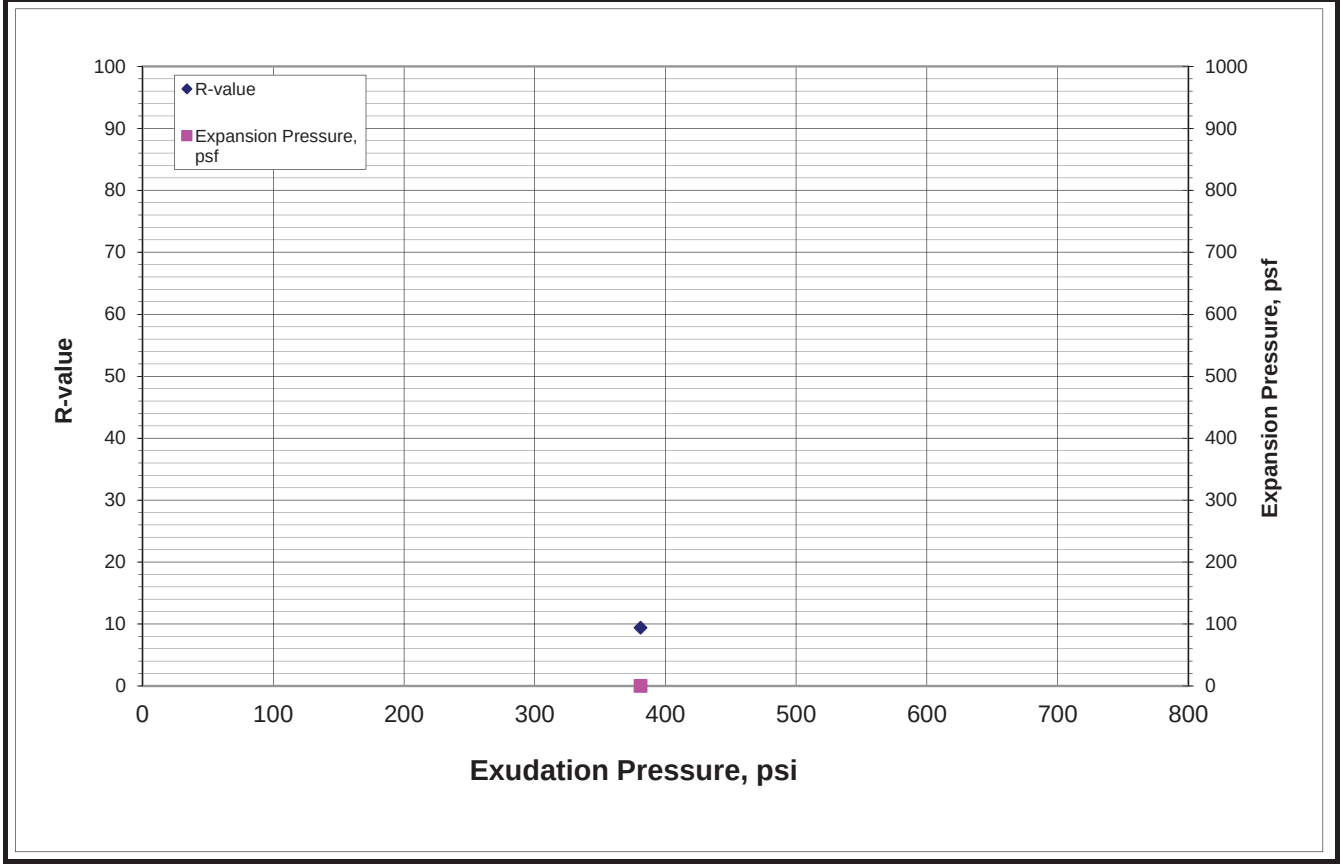
R-value Test Report (Caltrans 301)

Job No.:	664-067	**Date:**	04/13/16	**Initial Moisture,**	16.9%
Client:	BSK Associates	**Tested**	MD	**R-value by Stabilometer**	<5
Project:	G16-054-11L	**Reduced**	RU		
Sample	R-2 @ 1'	**Checked**	DC	**Expansion Pressure**	psf
Soil Type:	Very Dark Grayish Brown CLAY w/ Sand				

Specimen Number	A	B	C	D
Exudation Pressure, psi	381			
Prepared Weight, grams	1200			
Final Water Added, grams/cc	83			
Weight of Soil & Mold, grams	3045			
Weight of Mold, grams	2076			
Height After Compaction, in.	2.56			
Moisture Content, %	25.0			
Dry Density, pcf	91.7			
Expansion Pressure, psf	0.0			
Stabilometer @ 1000				
Stabilometer @ 2000	138			
Turns Displacement	3.95			
R-value	9			

Remarks:

Soil extruded from the mold giving a false exudation pressure. Per Caltrans, the R-Value test was terminated and an R-Value of less than 5 was reported.



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BSK Associates Engineers & Laboratories

PROJECT NO. G16-054-11L	R-VALUE TEST	**PLATE**
DRAWN: 4/27/16		C-3
DRAWN BY: D. Tower		
CHECKED BY: B. Steen	**Campus Reconstruction** **Harder Elementary School** **495 Wyeth Road** **Hayward, California**	
FILE NAME: Lab.indd		

APPENDIX D

GEOLOGIC AND SEISMIC HAZARDS ASSESSMENT

APPENDIX D
GEOLOGIC AND SEISMIC HAZARDS ASSESSMENT REPORT
CAMPUS RECONSTRUCTION
HARDER ELEMENTARY SCHOOL
HAYWARD, CALIFORNIA

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D1. INTRODUCTION

This report presents the geologic and seismic hazards assessment prepared in accordance with the 2013 California Building Code (CBC), CCR Title 24, Chapters 16A and 18A requirements for a Geotechnical/Engineering Geologic Report. The assessment was performed in conformance with California Geological Survey (CGS) Note 48 (2013).

D1.1 Purpose and Scope of Services

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

1. Review of published geologic literature, and current and past investigations at the site;
2. Evaluation of the data collected and preparation of geologic cross sections;
3. Evaluation of potential geologic hazards affecting the site;
4. Performance of a site-specific ground motion hazard analysis; and
5. Determination of Site Class and site-specific seismic design parameters.

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

D1.2 Site Location

As shown on Figure D-1, Harder Elementary School (Site) is located at 495 Wyeth Road in Hayward, Alameda County, California. A site plan of the school campus is shown on the Site Plan, Figure D-2.

The site coordinates are approximately:

Latitude 37.65092 ºN
Longitude 122.07213ºW

The surrounding area is mixed commercial and residential. The adjacent properties to the north, west and south are residential with commercial properties to the east.

D1.3 Site Topography

The project area is generally flat with an elevation of approximately 58 feet Mean Sea Level (msl). The Site and adjacent properties are generally flat lying and slope slightly down to the west/southwest.

D1.4 Groundwater Conditions

The Site is located within the East Bay Plain portion of the Santa Clara Valley groundwater basin. Ward Creek, which has been converted to a concrete lined channel, is located approximately 80 feet west of the Site. During our March and June 2016 field exploration, groundwater was encountered in our soil borings at depths of approximately 18.5 feet to 20 feet below the ground surface (bgs). The historically high depth to groundwater was reported as approximately 20 feet bgs (CGS, 2003).

D2. GEOLOGIC SETTING

The site is located in the Coastal Range geomorphic province that is characterized by north-south trending ridges and valley that are typically highly folded with numerous faults. The Site is located near the transition from the East Bay Plain and the East Bay Hills. The hills east of the site consist of Franciscan Complex and Coast Range Ophiolite along with folded Cretaceous sedimentary deposits and Jurassic and Cretaceous Knoxville Formation.

As shown on Figure D-3 and D-4, the Site is located on alluvial fan deposits that originate from the hills east of the Site. The surrounding hills, located approximately 2,300 feet east of the site, consist of folded units identified by Dibblee 2005 as Jurassic Franciscan Greenstone, gabbro-diabase with Knoxville shale. These geologic formations (Greenstone/gabbro-diabase) may have the potential to contain Naturally Occurring Asbestos (NOA). The potential hazard associated with NOA should be addressed in a Phase I Environmental Site Assessment submitted to the California Department of Toxics Substance Control (DTSC) for review.

Nearby active faults include the Hayward Fault located approximately 2,200 feet east of the Site, the Calaveras Fault located approximately 8 miles east of the site, and the San Andreas Fault located approximately 18 miles west of the Site.

D2.1 Subsurface Conditions

Subsurface conditions are described in the 2016 geotechnical investigation report prepared by BSK Associates (BSK) and to which this geologic and seismic hazards report is appended. The site was the subject of a field investigation of nine soil borings completed up to 30 feet below the ground surface (bgs) and three cone penetration test holes completed to approximately 50 feet bgs. The underlying stratigraphy consists of mostly fine grained silts and clays to a total depth explored of approximately 50 feet bgs. Minor clayey sand layers were encountered in borings B-1, B-2 and B-5 at depths of

approximately 16 feet, 24 feet and 14 feet, respectively. Figure D-5 presents geologic cross sections showing the current site surface topography and the subsurface conditions inferred from the soil borings drilled and CPTs advanced at the site.

D3. GEOLOGIC/SEISMIC HAZARDS

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

D3.1 Fault Rupture Hazard Zones in California

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

The site is not located in a Fault-Rupture Hazard Zone. As show on Figure D-6, the closest Fault-Rupture Hazard Zone is associated with the Hayward Fault located approximately 1,700 feet east of the Site.

D3.2 State of California Seismic Hazard Zones (Liquefaction and Landslides)

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

The site is within the Hayward 7.5 Minute Quadrangle. As shown on Figure D-7, the site is not located in a liquefaction or landslide hazard zone.

D3.3 Slope Stability and Potential for Slope Failure

Figure D-8 presents a distribution of slides and earthflows as mapped by Wentworth et al. (1998). Based on this map, the site is located on surficial deposits, not a landslide prone area.

D3.4 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.

D3.4.1 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 Figure D-9, according to FEMA Flood Hazard Map Layer GIS data (06001C-NFHL), dated 8/03/2009, the Site lies in Zone X outside the 100-year and 500-year floodplains.

D3.4.2 Inundation Hazards - Dams

According to GIS data obtained from California Emergency Management Agency, there are no reservoirs capable of causing inundation due to the breach of a dam in the vicinity of the Site. The Site is located outside the inundation area associated with Ward Creek (Dam Inundation GIS data from California Emergency Management Agency, dated 2013) as shown on Figure D-9.

D3.5 Volcanic Hazards

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

D3.6 Corrosion

Please refer to the section titled “Corrosion Assessment” in the geotechnical report for discussion of the corrosivity of the site soils.

D3.7 Expansive Soils

As discussed in the geotechnical report, the near-surface soils encountered within the current borings throughout the site are clays which exhibit a moderate to high expansion potential. Mitigation of the effects of these soils on the proposed development is discussed in the geotechnical report.

D3.8 Alameda County Safety Element

According to the 2013 Safety Element of the Alameda County General Plan, the site is not currently located in a seismic or geohazard zone.

D4. SEISMIC HAZARD ASSESSMENT

D4.1 Seismic Source Deaggregation

Figures D-10 and D-11 present fault maps showing the major faults that may impact the site in the future. Seismically induced ground motion at a site can be caused by earthquakes on any of the sources surrounding the site. Deaggregation of the seismic hazard was performed by using the USGS Interactive Deaggregation website. The deaggregation determination, at the maximum considered earthquake (MCE) hazard level, results in distance, magnitude and epsilon (ground-motion uncertainty) for each source that contributes to the hazard. Each source has a corresponding epsilon, which is the probabilistic value relative to the mean value of ground motion for that source.

Table D-1 below lists the result of deaggregation based on a probabilistic model developed by the USGS. The most extreme seismic source that contributes to the peak ground acceleration (PGA_M) is a magnitude 7.30 earthquake from a rupture of multiple segments of the Hayward Fault. The modal magnitude of 6.81 is consistent with the general design earthquake ground motion. According to the site-specific analysis for the geometric mean peak ground acceleration (PGA) in Section D4.5.7, the controlling source is a deterministic derived value of magnitude 7.00 from the California gridded source. For liquefaction and seismic settlement, a magnitude of 7.00 should be used.

TABLE D-1
SEISMIC HAZARD DEAGGREGATION
MAXIMUM CONSIDERED EARTHQUAKE

Seismic Source	Percent Contribution	Distance (km)	Magnitude (Mw)	Epsilon (Mean Values)
PGA Deaggregation (USGS 2008)				
Hayward-Rodgers Crk;RC+HN+HS aPr	3.1	1.6	7.30	1.1
Hayward-Rodgers Crk;RC+HN+HS MoB	2.9	1.6	7.29	1.1
Hayward-Rodgers Crk;Unsegmented	8.7	1.7	6.97	1.3
California A-faults	96.9	2.6	6.82	1.4
Hayward-Rodgers Creek;HS aPriori	23.5	2.6	6.68	1.5
Hayward-Rodgers Crk;HN+HS aPrior	19.1	2.6	6.91	1.4
Hayward-Rodgers Creek;HS MoBal	21.4	2.6	6.66	1.5
Hayward-Rodgers Creek;HN+HS MoBa	17.7	2.6	6.86	1.4
CA Compr. crustal gridded	3.1	7.3	6.07	2.0

TABLE D-1 SEISMIC HAZARD DEAGGREGATION MAXIMUM CONSIDERED EARTHQUAKE				
Seismic Source	**Percent Contribution**	**Distance (km)**	**Magnitude (Mw)**	**Epsilon (Mean Values)**
Mean	-	2.7	6.80	1.4
Modal	-	2.6	6.81	1.5

D4.2 Area Faults Characteristic Magnitude

Table D-2 below lists the faults within a radius of 100 kilometers from the site based on the USGS 2008 fault model. The magnitudes presented are deterministic characteristic magnitudes that are determined using magnitude-rupture area relationships developed by Hanks and Ellsworth as presented in Petersen et al. (2008).

TABLE D-2 AREA FAULTS CHARACTERISTIC MAGNITUDE			
Name	**Distance (km)**	**Hanks Magnitude (Mw)**	**Ellsworth Magnitude (Mw)**
Hayward-Rodgers Creek;HN+HS	1.0	6.80	7.00
Hayward-Rodgers Creek;HS	1.0	6.38	6.60
Hayward-Rodgers Creek;RC+HN+HS	1.0	7.25	7.33
Calaveras;CN	13.1	6.80	7.00
Calaveras;CN+CC	13.1	6.84	7.03
Calaveras;CN+CC+CS	13.1	6.84	7.03
Mount Diablo Thrust	20.9	6.50	6.70
Hayward-Rodgers Creek;HN	23.2	6.38	6.60
Hayward-Rodgers Creek;RC+HN	23.2	7.06	7.19
Monte Vista-Shannon	28.5	6.30	6.50
Green Valley Connected	28.6	6.60	6.80
N. San Andreas;SAN+SAP	28.7	7.73	7.70
N. San Andreas;SAN+SAP+SAS	28.7	7.87	7.80
N. San Andreas;SAO+SAN+SAP	28.7	7.95	7.86
N. San Andreas;SAO+SAN+SAP+SAS	28.7	8.05	7.94
N. San Andreas;SAP	28.7	7.11	7.23
N. San Andreas;SAP+SAS	28.7	7.45	7.48
Greenville Connected	31.4	6.80	7.00
Calaveras;CC	32.7	6.17	6.39
Calaveras;CC+CS	32.7	6.28	6.50
San Gregorio Connected	40.6	7.40	7.50

TABLE D-2 AREA FAULTS CHARACTERISTIC MAGNITUDE			
Name	**Distance (km)**	**Hanks Magnitude (Mw)**	**Ellsworth Magnitude (Mw)**
Great Valley 5, Pittsburg Kirby Hills	45.2	6.50	6.70
N. San Andreas;SAN	46.8	7.48	7.51
N. San Andreas;SAO+SAN	46.8	7.80	7.75
Great Valley 7	49.1	6.60	6.90
N. San Andreas;SAS	53.0	6.968	7.123
Hayward-Rodgers Creek;RC	58.0	6.89	7.07
West Napa	59.0	6.50	6.70
Zayante-Vergeles	63.0	6.90	7.00
Great Valley 4b, Gordon Valley	71.4	6.60	6.80
Point Reyes	74.6	6.70	6.90
Monterey Bay-Tularcitos	81.1	7.30	7.20
Ortigalita	81.7	6.90	7.10
Great Valley 8	84.2	6.60	6.80
Hunting Creek-Berryessa	89.7	6.90	7.10
Great Valley 3, Mysterious Ridge	90.8	6.90	7.10
Great Valley 4a, Trout Creek	98.0	6.40	6.60

D4.3 Historical Seismicity

The project site and its vicinity are located in an area traditionally characterized by high seismic activity. A number of large earthquakes have occurred within the site vicinity during historic time (since 1800). Some of the significant regional earthquake events include: the 1906 (M7.8) Great San Francisco earthquake, located 29 miles west of the site and the 1868 (M6.8) earthquake that originated on the Hayward Fault 10 miles west of the site.

In April 2008, scientists and engineers released a new earthquake forecast for the State of California, which were compiled by the U.S. Geological Survey, the Southern California Earthquake Center (SCEC), and CGS with support from the California Earthquake Authority (Field et al., 2008). It updates the earthquake forecast made for the greater San Francisco Bay Area by the 2002 Working Group for California Earthquake Probabilities. According to this recent study, there is a 63 percent probability that one or more magnitude M6.7 or greater earthquakes will occur in the San Francisco Bay Area within the next approximately 30 years (between 2007 and 2036). As has been demonstrated recently by the 1989 (M6.9) Loma Prieta, 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.

Figure D-12 presents historical earthquake magnitudes of significant earthquakes located in the project area based on the Working Group on California Earthquake Probabilities (WGCEP) Historical California Earthquake Catalog (Felzer, 2008). This earthquake catalog is for California and provides a listing for all known M ≥ 5.5 earthquakes that occurred from 1850-1932 and all known M ≥4.0 earthquakes that occurred from 1932-2006. Some pre-1932 earthquakes 4≤ M ≤ 5.5 are also listed.

Table D-3 provides the location, earthquake magnitude, site to earthquake distances, dates and the resulting site peak horizontal acceleration for the period 1800 to 1999. The site has experienced mean plus one sigma peak horizontal acceleration up to 0.78g from the 1868 earthquake on the Hayward Fault.

TABLE D-3
HISTORIC EARTHQUAKES WITHIN 100 MILES OF THE SITE
GROUND MOTION GREATER THAN 0.15G

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	0.78	3.7(6.0)
DMG	37.700	122.500	4/18/1906	0	8.3	0.50	23.6(38.0)
DMG	37.800	122.200	06/10/1836	0	6.8	0.38	12.4(20.0)
DMG	37.700	122.000	03/05/1864	0	5.7	0.37	5.2(8.4)
DMG	37.600	122.100	05/21/1864	0	5.3	0.35	3.8(6.2)
DMG	37.600	122.400	06/01/1838	0	7.0	0.32	18.3(29.4)
T-A	37.670	122.070	12/04/1887	0	4.3	0.27	1.3(2.1)
DMG	37.500	121.900	11/26/1858	0	6.1	0.24	14.0(22.6)
DMG	37.800	122.000	07/04/1861	0	5.6	0.22	11.0(17.7)
DMG	37.600	122.000	07/22/1864	0	4.7	0.22	5.3(8.5)
DMG	37.600	122.000	5/16/1933	0	4.5	0.20	5.3(8.5)
DMG	37.720	122.130	12/17/1954	0	4.5	0.19	5.7(9.2)
DMG	37.720	122.120	12/29/1942	0	4.3	0.17	5.4(8.7)
GSB	37.727	122.130	3/27/1984	7	4.4	0.17	6.1(9.9)
MGI	37.800	122.500	06/21/1808	0	6.3	0.17	25.5(41.1)
DMG	37.300	121.900	10/08/1865	0	6.3	0.17	26.0(41.8)
DMG	37.600	121.800	6/11/1903	0	5.5	0.16	15.3(24.6)
BRK	37.830	121.810	1/24/1980	0	5.8	0.16	18.9(30.4)
DMG	37.250	121.750	7/1/1911	0	6.6	0.16	32.8(52.8)
DMG	37.800	122.200	07/31/1889	0	5.2	0.16	12.4(20.0)
GSB	37.036	121.883	10/18/1989	18.5	7.0	0.16	43.7(70.3)
DMG	37.500	122.300	02/15/1856	0	5.5	0.16	16.2(26.1)
DMG	37.600	122.000	6/5/1907	0	4.0	0.15	5.3(8.5)

D4.4 Earthquake Ground Motion, 2013 California Building Code

D4.4.1 Site Class

Based on Section 1613.3.2 of the 2013 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 7-10. Based on the estimated average value Undrained Shear Strength (Su) of the soil, as per Table 20.3-1 of ASCE 7-10, the Site is Class D (1,000 psf < Su < 2,000 psf).

D4.4.2 Seismic Design Criteria

The 2013 California Building Code (CBC) utilizes ground motion based on the Risk-Targeted Maximum Considered Earthquake (MCE_R) that is define in the 2013 CBC as the most severe earthquake effects considered by this code, determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk. Ground motion parameters in the 2013 CBC are based on ASCE 7-10, Chapter 11.

The United States Geologic Survey (USGS) has prepared maps presenting the Risk-Targeted MCE spectral acceleration (5% damping) for periods of 0.2 seconds (S_S) and 1.0 seconds (S_1). The values of S_S and S_1 can be obtained from the USGS Ground Motion Parameter Application available at: http://earthquake.usgs.gov/designmaps/us/application.php

Table D-4 below presents the spectral acceleration parameters produced for Site Class D by the USGS Ground Motion Parameter Application and Chapter 16 of the 2013 CBC based on ASCE 7-10.

TABLE D-4
SPECTRAL ACCELERATION PARAMETERS
RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE

Criteria	Value		Reference
MCE Mapped Spectral Acceleration (g)	S_S = 2.392	S_1 = 0.995	USGS Mapped Value
Site Coefficients (Site Class D)	F_a = 1.000	F_v = 1.500	ASCE Table 11.4
Site Adjusted MCE Spectral Acceleration (g)	S_{MS} = 2.392	S_{M1} = 1.493	ASCE Equations 11.4.1-2
Design Spectral Acceleration (g)	S_{DS} = 1.595	S_{D1} = 0.995	ASCE Equations 11.4.3-4

D4.4.3 Seismic Design Category

The long period spectral response acceleration coefficient, S_1, is greater than than 0.75, therefore the site lies in Seismic Design Category E, based on Risk Category III. When S_1 is greater than or equal to 0.75g, the Seismic Design Category is E for buildings in Risk Categories I, II, and III, and F for those in Risk Category IV.

D4.4.4 Geometric Mean Peak Ground Acceleration

As per Section 1803A.5.12 of the CBC, peak ground acceleration (PGA) utilized for dynamic lateral earth pressures and liquefaction, shall be based on a site specific study (ASCE 7-10, Section 21.5) or ASCE 7-10, Section 11.8.3. The USGS Ground Motion Parameter Application based on ASCE 7-10, Section 11.8.3 produced the values shown in Table D-5 based on Site Class D.

TABLE D-5
GEOMETRIC MEAN PEAK GROUND ACCELERATION
MAXIMUM CONSIDERED EARTHQUAKE

Criteria	Value	Reference
Mapped Peak Ground Acceleration (g)	PGA = 0.922	USGS Mapped Value
Site Coefficients (Site Class D)	F_{PGA} = 1.000	ASCE Table 11.8-1
Geometric Mean PGA (g)	PGA_M = 0.922	ASCE Equations 11.8-1

D4.5 Site-Specific Ground Motion Analysis

Section 1616A.1.3 of the 2013 CBC specifies that for buildings assigned to Seismic Design Category E or F, or when required by the building official, a ground motion hazard analysis shall be performed in accordance with ASCE 7 Chapter 21 as modified by Section 1803A.6 of this code. A site-specific seismic hazard analysis in general conformance with ASCE 7-10 Chapter 21 Section 21.2 was performed. Our ground motion analysis includes:

1. Determination of risk-targeted maximum considered earthquake (MCE_R) ground motion, deterministic MCE_R ground motion and probabilistic MCE_R ground motion.
2. Determination of site-specific maximum considered earthquake geometric mean (MCE_G) peak ground acceleration.
3. The analysis was performed according to requirements of the ASCE 7-10, sections 21.2 through 21.5.

D4.5.1 Deterministic MCER Ground Motion

Estimates of the MCE deterministic ground motion were determined using the software program EZ-Frisk 7.65 developed by Risk Engineering. The Frisk analysis indicates that at periods less than one second, the California Gridded source would produce the highest ground motion at the site from a deterministic standpoint. At periods above one second, the Hayward Fault dominates the ground motion.

Site-specific ground motions can be influenced by the types of faulting, magnitudes of the earthquakes, and local soil conditions. Ground Motion Prediction Equations (GMPE) account for these effects and are

used to make estimates of ground motion at a site resulting from a scenario earthquake. Many GMPEs have been developed to estimate the variation of spectral acceleration with earthquake magnitude and distance from the site to the source of an earthquake. Next Generation Attenuation of Ground Motion relationships were developed by the Pacific Earthquake Engineering Research (PEER) Center that presented GMPEs for shallow crustal earthquakes in Western North America.

The 84th percentile of the maximum rotated component ground motion values were determined using four different Next Generation Attenuation relationships (NGAs). Distant Cascadia sources did not significantly increase acceleration values. The Site ground motion is dominated by numerous local faults, therefore Cascadia sources were not included in the analysis. The acceleration values from each of three attenuation relationships were averaged using equal weight. The following attenuation relationships were used in the analysis:

- Boore-Atkinson (2008) NGA Maximum Rotated Horizontal Component
- Campbell-Bozorgnia (2008) NGA Maximum Rotated Horizontal Component
- Chiou-Youngs (2008) NGA Maximum Rotated Horizontal Component
- Abrahamson-Silva (2008) NGA Maximum Rotated Horizontal Component

The analysis included seismic sources, based on the 2008 USGS fault model, within 200 kilometers of the site.

Soil amplification was accounted for in the analysis using the shear wave velocity (Vs) of 1,082 fps (330 m/s) estimated from the averaged Undrained Shear Strength values from CPT-1, CPT-2 and CPT-3 (Wair, 2012). In addition, some of the GMPEs require input for Z1.0 (defined as the depth in meters to a layer with Vs = 1,000 m/s) and Z2.5 (depth in km to a layer with Vs= 2,500 m/s). These two parameters intend to capture the basin effect on site response. The Z1.0 parameter is estimated to be 450 meters based on relationships established for Quaternary alluvium. The project site is located in the East Bay Trough with an estimated depth to 2,500 m/s of 3 kilometers for older Cenozoic sedimentary rocks (Brocher, 2005).

As specified in ASCE 7-10, Section 21.2.2, the deterministic spectral acceleration values representing the MCE_R are taken as the 84th percentile of the maximum rotated component 5% damped spectral accelerations. The deterministic response spectra are plotted on Figure D-13.

D4.5.2 Deterministic Lower Limit

ASCE 7-10 Section 21.2.2 specifies that the ordinates of the deterministic MCE_R ground motion response spectrum shall not be taken lower than the deterministic lower limits where:

SaM = 1.5Fa and SaM = 0.6(Fv/T), Ss = 1.5 and S1 = 0.6

As per Tables 11.4-1 and 11.4-2 of ASCE 7-10, Site Class D, Fa = 1.00 and Fv = 1.50

The MCE_R deterministic lower limits using the above parameters and the 84th percentile deterministic site specific response spectrum adjusted using the deterministic lower limits are shown on Figure D-13.

D4.5.3 Probabilistic MCER Ground Motion

The probabilistic MCE_R ground motion was determined using the method in ASCE 7-10 Section 21.2.1.1. Estimates of the MCE_R probabilistic ground motion were determined using the software program EZ-Frisk 7.65 Build 4 developed by Risk Engineering. The analysis included active faults within 200 km of the Site. Mean maximum rotated component acceleration values were determined using the same attenuation relationships and soil amplification as specified in the deterministic analysis above. The acceleration values from each of attenuation relationships were averaged using equal weight. The probabilistic MCE spectral acceleration values based upon our analysis are plotted on Figure D-14.

As specified in ASCE 7-10 Section 21.2.1.1, the MCE_R ground motion was determined by the adjusting the spectral acceleration values using the risk coefficients C_{RS} (0.985) and C_{R1} (0.961) obtained from the USGS Ground Motion Parameter Application. The risk targeted MCE_R probabilistic spectrum based upon our analysis is plotted on Figure D-15.

D4.5.4 Design Response Spectrum (5% Damping)

As shown on Figure D-16, the MCE_R deterministic spectrum is less than the probabilistic spectrum at all periods. According to ASCE 7-10 21.2.3 the lesser spectral values were used to construct the design spectrum. The site-specific design response spectrum is taken as 2/3 of the MCE_R spectral values. As shown on Figure D-17, the site specific design spectrum was adjusted such that values are greater than 80% of the general design spectrum and should be utilized for design (5% Damping).

D4.5.5 Site Specific MCE Geometric Mean (MCEG) Peak Ground Acceleration

Per ASCE 7-10, Section 21.5 the site-specific MCE_G peak ground acceleration, PGA_M, was taken as the lesser of the probabilistic geometric mean peak ground acceleration and the deterministic geometric mean peak ground acceleration. The site-specific MCE_G peak ground acceleration should be greater than 80 percent of the general PGA_M.

The geometric mean values can be determined by dividing the MCE deterministic and probabilistic PGA value by 1.1 as recommended by NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, FEMA P-750 / 2009. The probabilistic value used is before application of the risk coefficients.

D4.5.6 Probabilistic MCEG Peak Ground Acceleration

The probabilistic geometric mean peak ground acceleration would be 1.450g (PGA from Figure D-14) divided by 1.1 or 1.318g.

D4.5.7 Deterministic MCEG Peak Ground Acceleration

The deterministic geometric mean peak ground acceleration would be 1.075g (PGA from Figure D-13) divided by 1.1 or 0.977g. This value is greater than $0.5*F_{PGA}$, where F_{PGA}=1.0 for a PGA=0.50g as stipulated in ASCE 7-10 Section 21.5.2. The controlling seismic source for the PGA is the California Gridded source (Mw=7.00).

D4.5.8 Site-Specific MCEG Peak Ground Acceleration

The lesser value of the geometric mean probabilistic and deterministic peak ground accelerations is the deterministic value, which is 0.977g. This value is greater than 80 percent of the PGA_M determined from ASCE 7-10 Section 11.8-1 (see Table D-5), therefore 0.977g should be used as the Site PGA value.

D4.6 Seismically Induced Ground Failure

D4.6.1 Liquefaction

Liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase, resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil is not confined. Soils most susceptible to liquefaction are loose, clean, uniformly graded, silt and fine sand, as well as some lean clay deposits. Based on the subsurface exploration performed for the investigation, the site is underlain by interbedded alluvial soils consisting primarily of soft to firm clays and silts, and medium dense clayey sands..

In order for liquefaction 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
- Ground shaking is of sufficient intensity to act as a triggering mechanism

In addition, after soil liquefies, dissipation of the excess pore pressures can produce volume changes within the liquefied soil layer, which can result in ground surface settlement. Liquefaction-induced settlement calculations based on the three CPTs advanced to 50 feet at the site resulted in less than ½

inch of settlement as discussed in the geotechnical report. The depths to the liquefiable layers ranged from 20 to 30 bgs.

D4.6.2 Lateral Spread

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material. These phenomena typically occur adjacent to free faces such as slopes and creek channels. Because the design water table is at 15 feet bgs and the lack of nearby incised channels to that depth,, the potential for lateral spreading to take place at the site is low.

D4.6.3 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 in the borings drilled for this project; therefore we consider the potential for seismic settlement to occur at this site is negligible.

D5. REFERENCES

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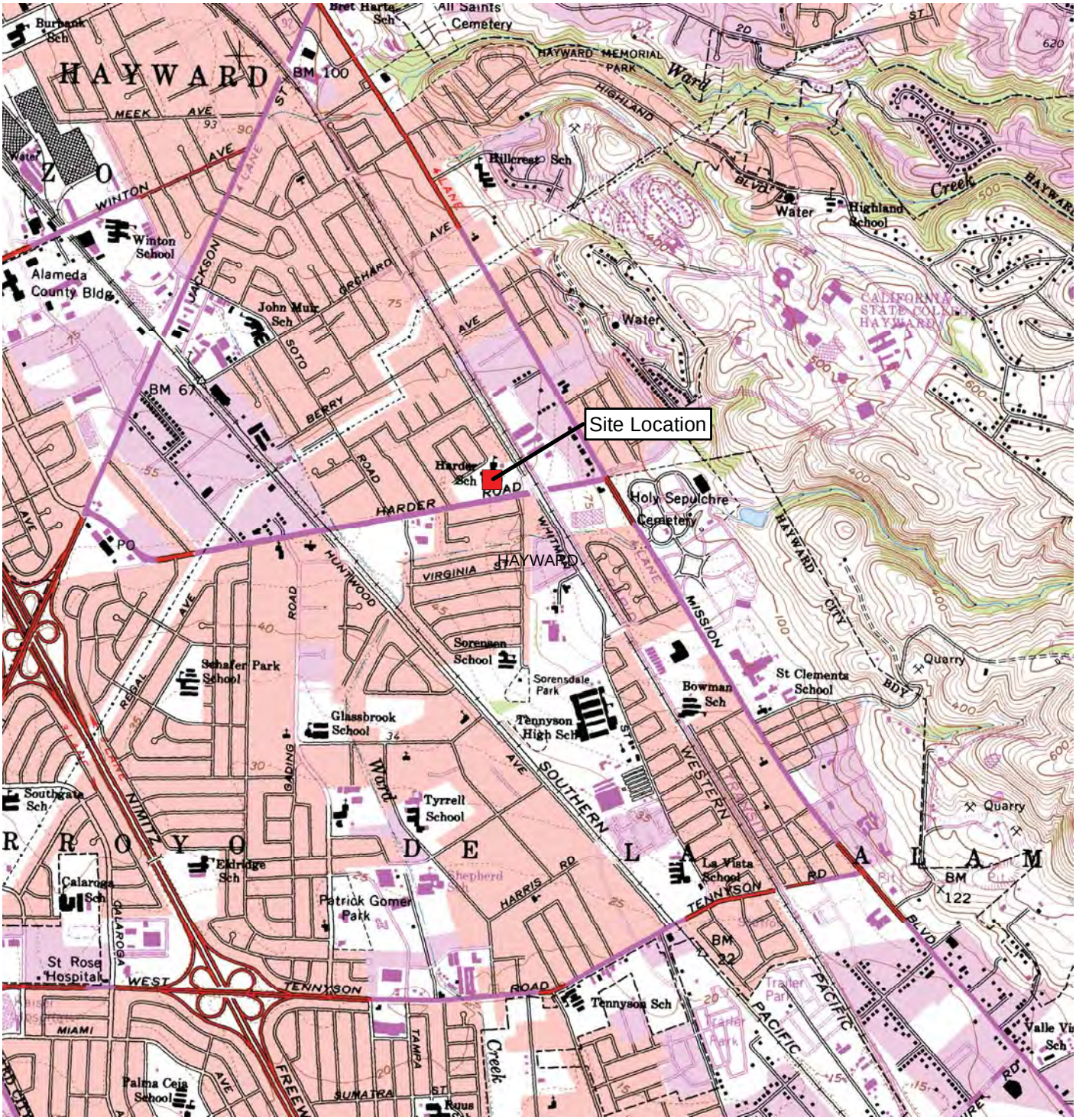

Reference: http://www.atlas.ca.gov/download.html#/casil/imageryBaseMaps LandCover/baseMaps/drg/7.5_minute_series_albers_nad83_trimmed

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PROJECT NO. G1605411L	**AREA TOPOGRAPHIC MAP**	Figure
Drawn: 4/12/2016	**Geologic/Seismic Hazards Assessment**	
DRAWN BY: MBC		**D-1**
CHECKED BY: BS	Campus Reconstruction Harder Elementary School Hayward, California	
FILE NAME: Figure D-1 Topomap		

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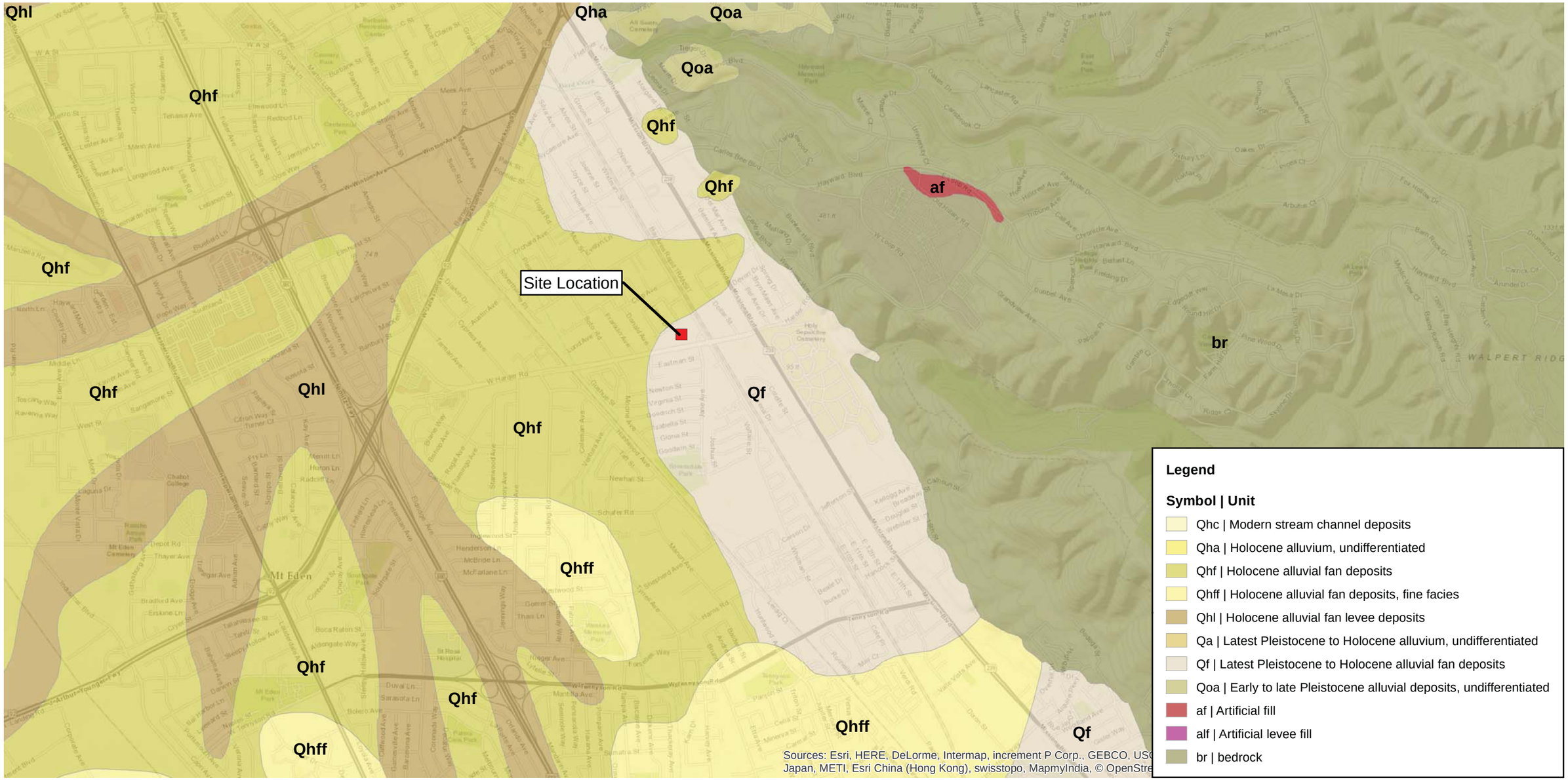
PROJECT NO. G1605411L	**SITE PLAN** **Geologic/Seismic Hazards Assessment**	Figure
Drawn: 4/13/2016		**D-2**
DRAWN BY: MBC		
CHECKED BY: BS	Campus Reconstruction Harder Elementary School Hayward, California	
FILE NAME: Figure D-2 Site map		



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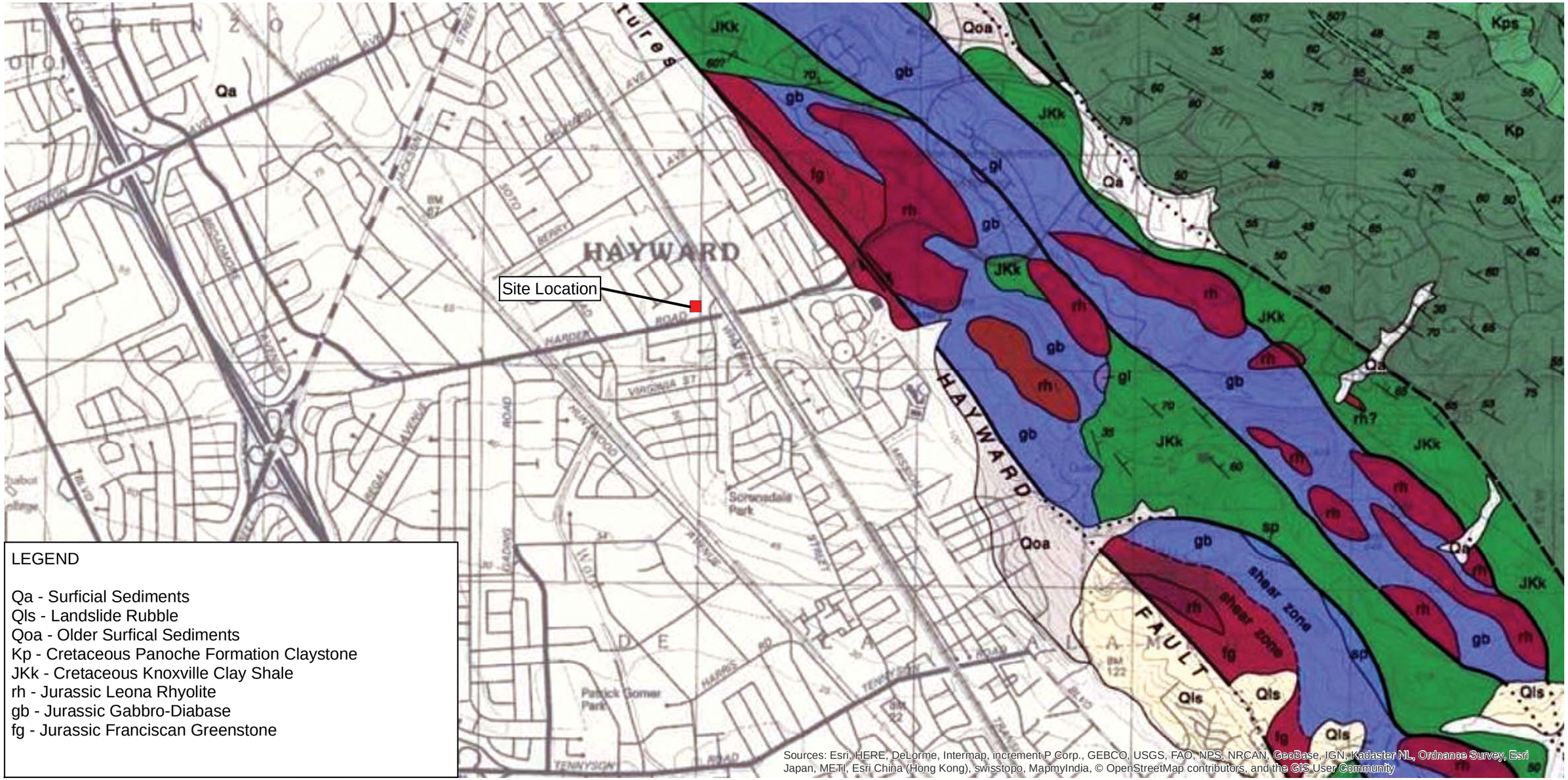
PROJECT NO. G1605411L	
Drawn: 4/12/2016	
DRAWN BY: MBC	
CHECKED BY: BS	
FILE NAME: Figure D-3 Geologic_map	

QUATERNARY GEOLOGIC MAP
Geologic/Seismic Hazards Assessment

Campus Reconstruction
Harder Elementary School
Hayward, California

Figure
D-3



LEGEND

Qa - Surficial Sediments
Qls - Landslide Rubble
Qoa - Older Surfical Sediments
Kp - Cretaceous Panoche Formation Claystone
JKk - Cretaceous Knoxville Clay Shale
rh - Jurassic Leona Rhyolite
gb - Jurassic Gabbro-Diabase
fg - Jurassic Franciscan Greenstone

References: Dibblee, T.W., and Minch, J.A., 2005, Geologic map of the Hayward quadrangle, Contra Costa and Alameda Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-163

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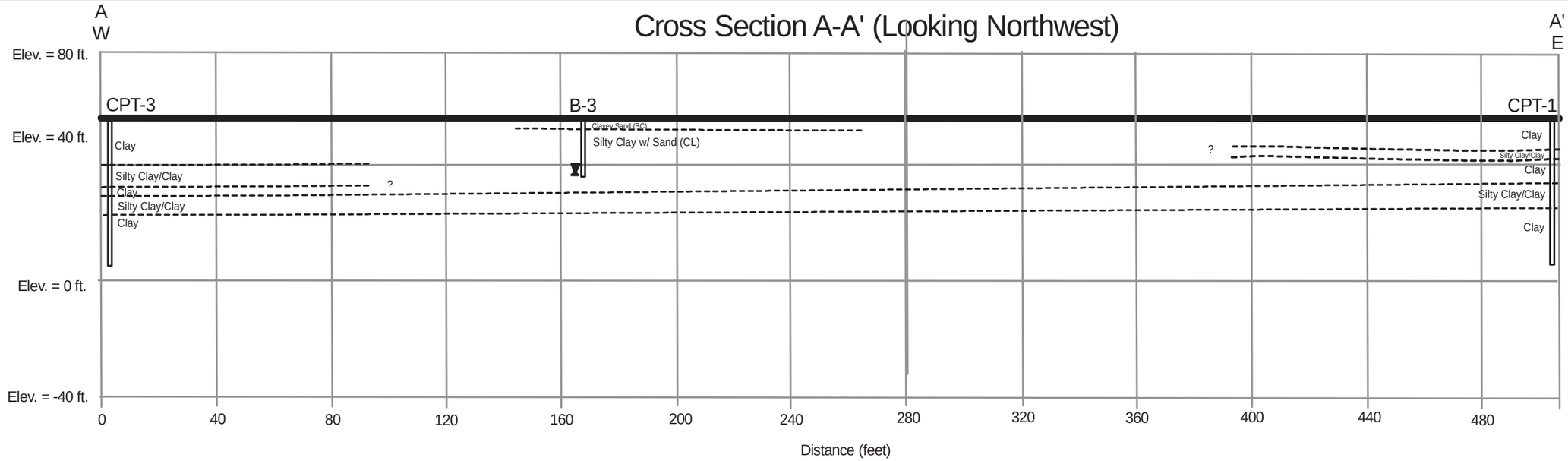
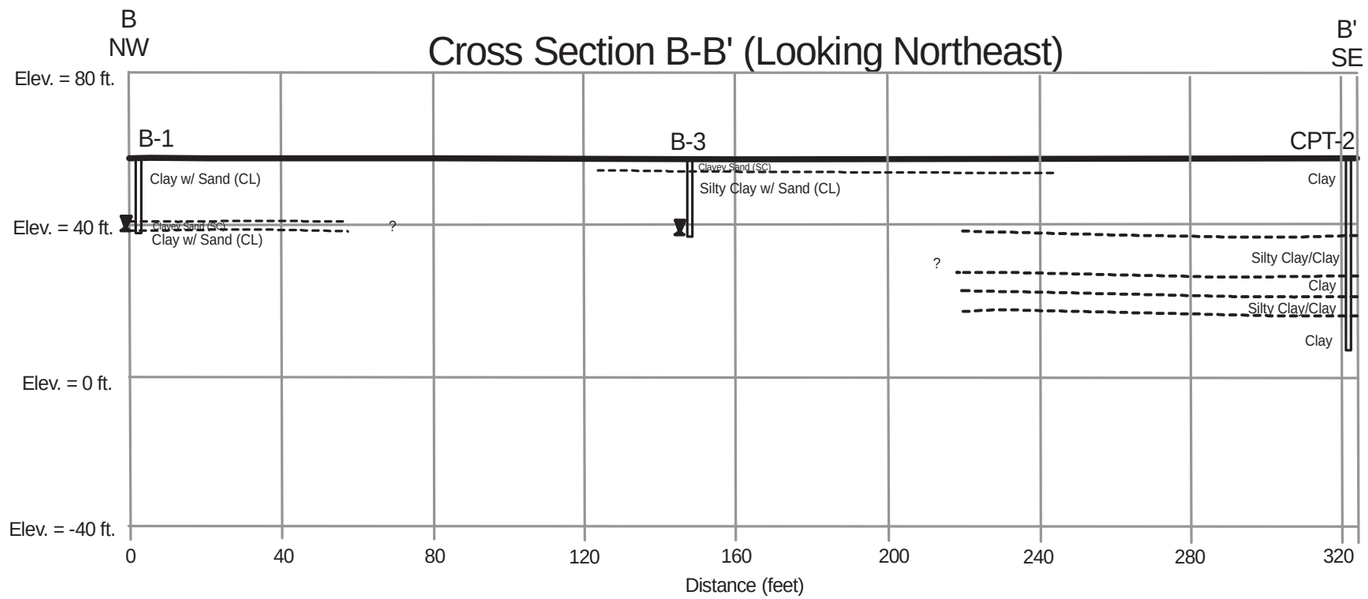
PROJECT NO. G1605411L	**GEOLOGIC MAP**	Figure
Drawn: 4/12/2016	**Geologic/Seismic Hazards Assessment**	
DRAWN BY: MBC		**D-4**
CHECKED BY: BS	Campus Reconstruction	
FILE NAME: Figure D-4 Geologic_map2	Harder Elementary School	
	Hayward, California	





NOTE: The cross sections shown here are for illustrative purposes only and area based on extrapolation and interpolation between and beyond borings drilled at the site. As such, the cross sections should be considered approximate and should not be used for construction. Actual subsurface conditions may vary and will need to be confirmed during grading by the Engineering Geologist or Geotechnical Engineer.

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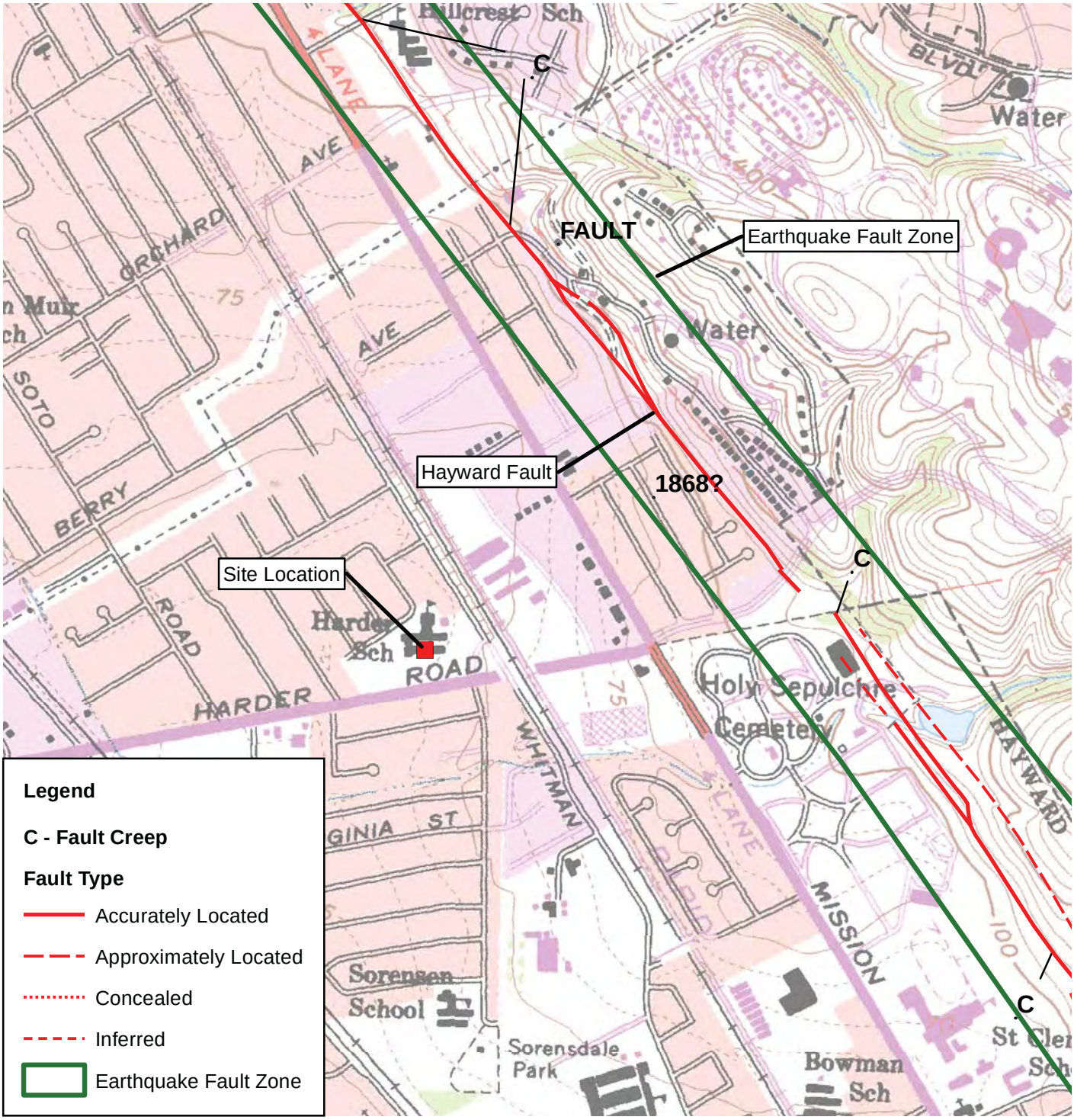
0 40 80
Approximate Scale in Feet
Horizontal=Vertical

LEGEND
Unit Contact
Soil Boring Location
Groundwater Level

NOTES:
1) Locations are Approximate.

PROJECT NO. G1605411L	GEOLOGIC CROSS SECTIONS	Figure
Drawn: 2/4/2016	Geologic/Seismic Hazards Assessment	D-5
DRAWN BY: MBC		
CHECKED BY: BS	Campus Reconstruction Harder Elementary School Hayward, California	
FILE NAME: Figure D-5 Cross-section3		



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

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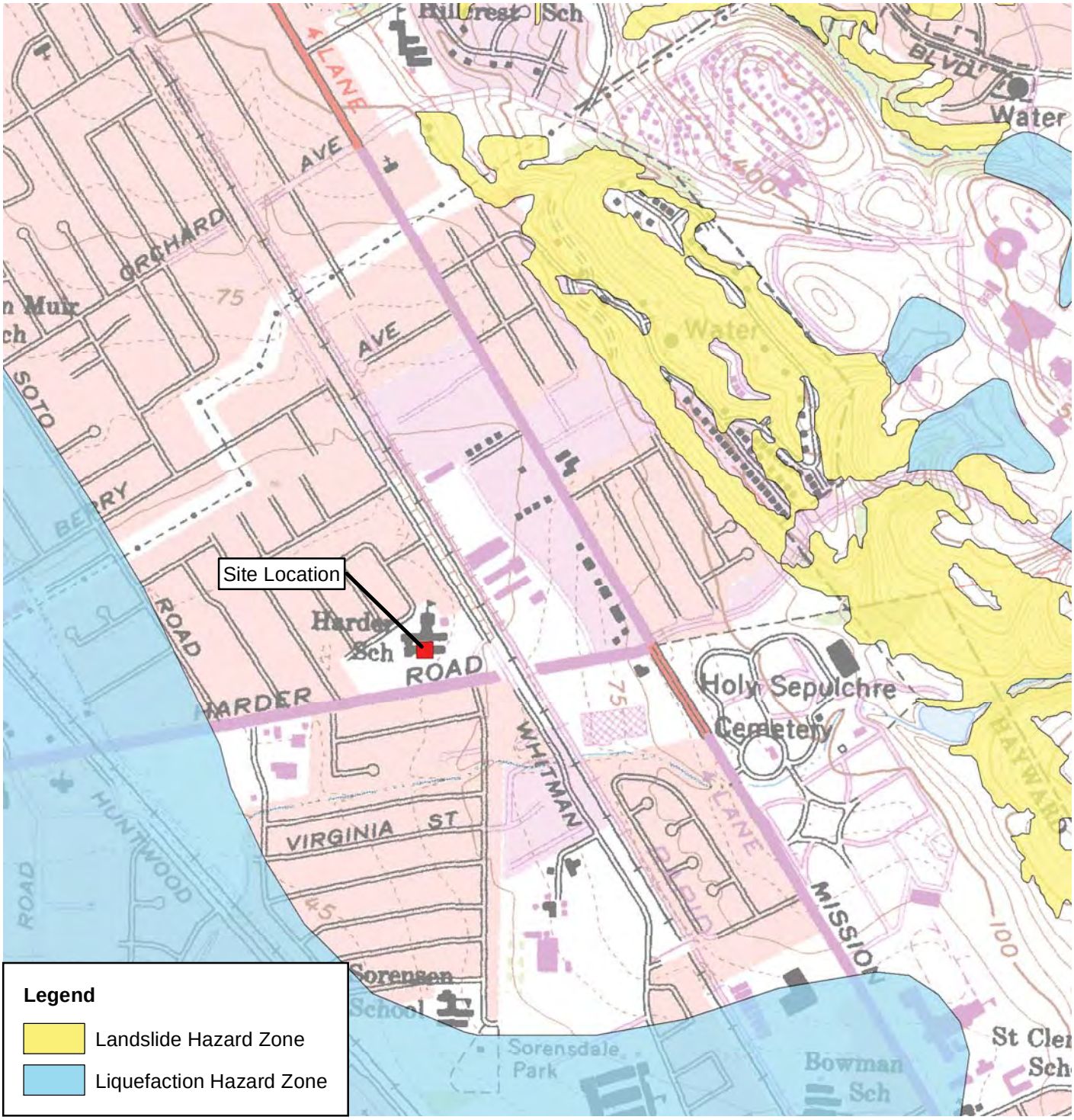
PROJECT NO. G1605411L	**Alquist-Priolo Earthquake Fault Zone**	Figure
Drawn: 4/12/2016	**Geologic/Seismic Hazards Assessment**	
DRAWN BY: MBC		**D-6**
CHECKED BY: BS	Campus Reconstruction Harder Elementary School Hayward, California	
FILE NAME: Figure D-6 AP_Fault_map		



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

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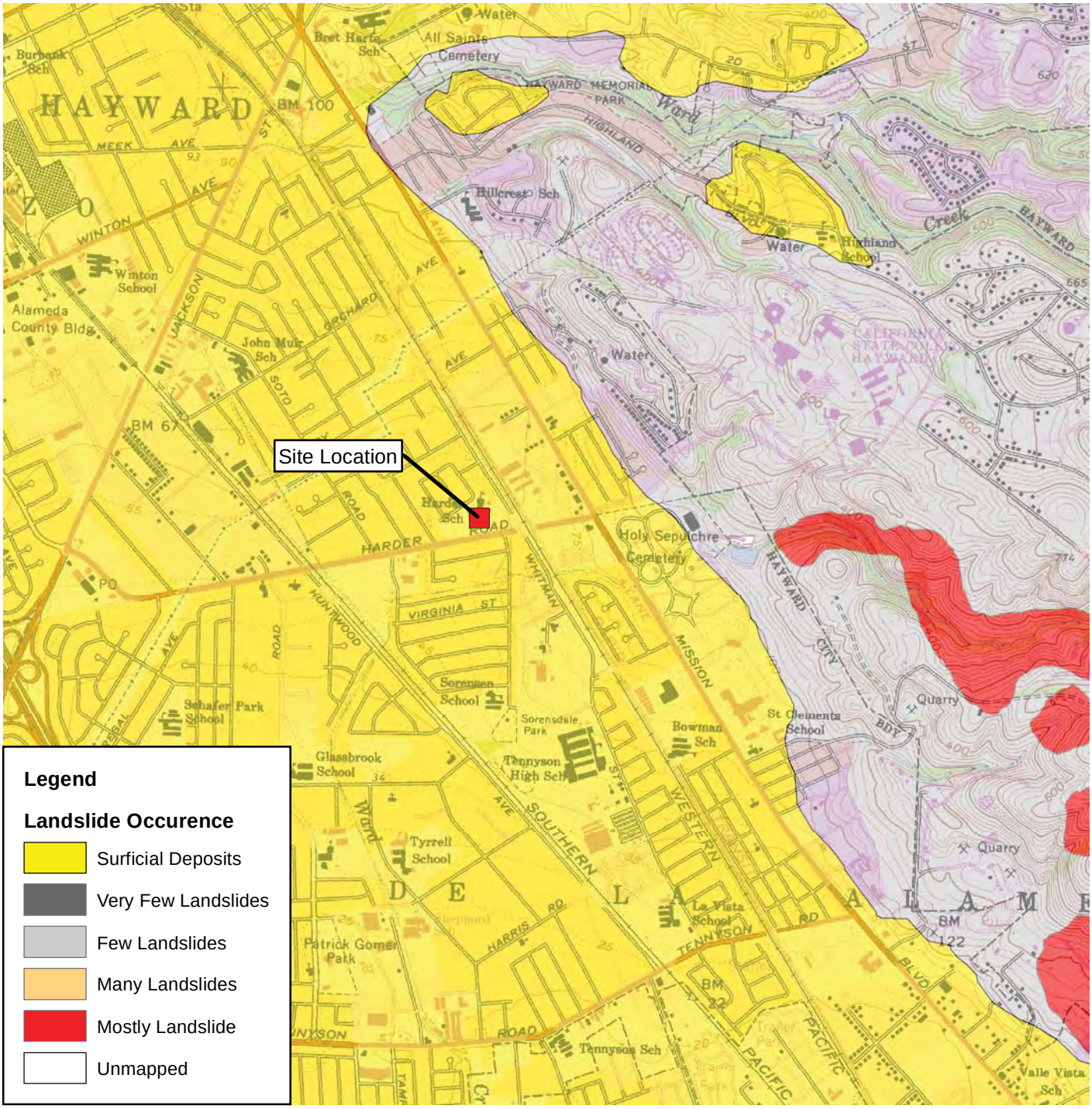
PROJECT NO. G1605411L	**State Seismic Hazard Zones**	Figure
Drawn: 4/12/2016	**Geologic/Seismic Hazards Assessment**	
DRAWN BY: MBC		**D-7**
CHECKED BY: BS	Campus Reconstruction Harder Elementary School Hayward, California	
FILE NAME: Figure D-7 Seismic Hazard Zone		



Reference: Adapted from USGS Open-File Report 97-745C,
By: C.M. Wentworth et al, 1998

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PROJECT NO. G1605411L	**LANDSLIDE HAZARD MAP**	Figure
Drawn: 4/12/2016	**Geologic/Seismic Hazards Assessment**	
DRAWN BY: MBC		**D-8**
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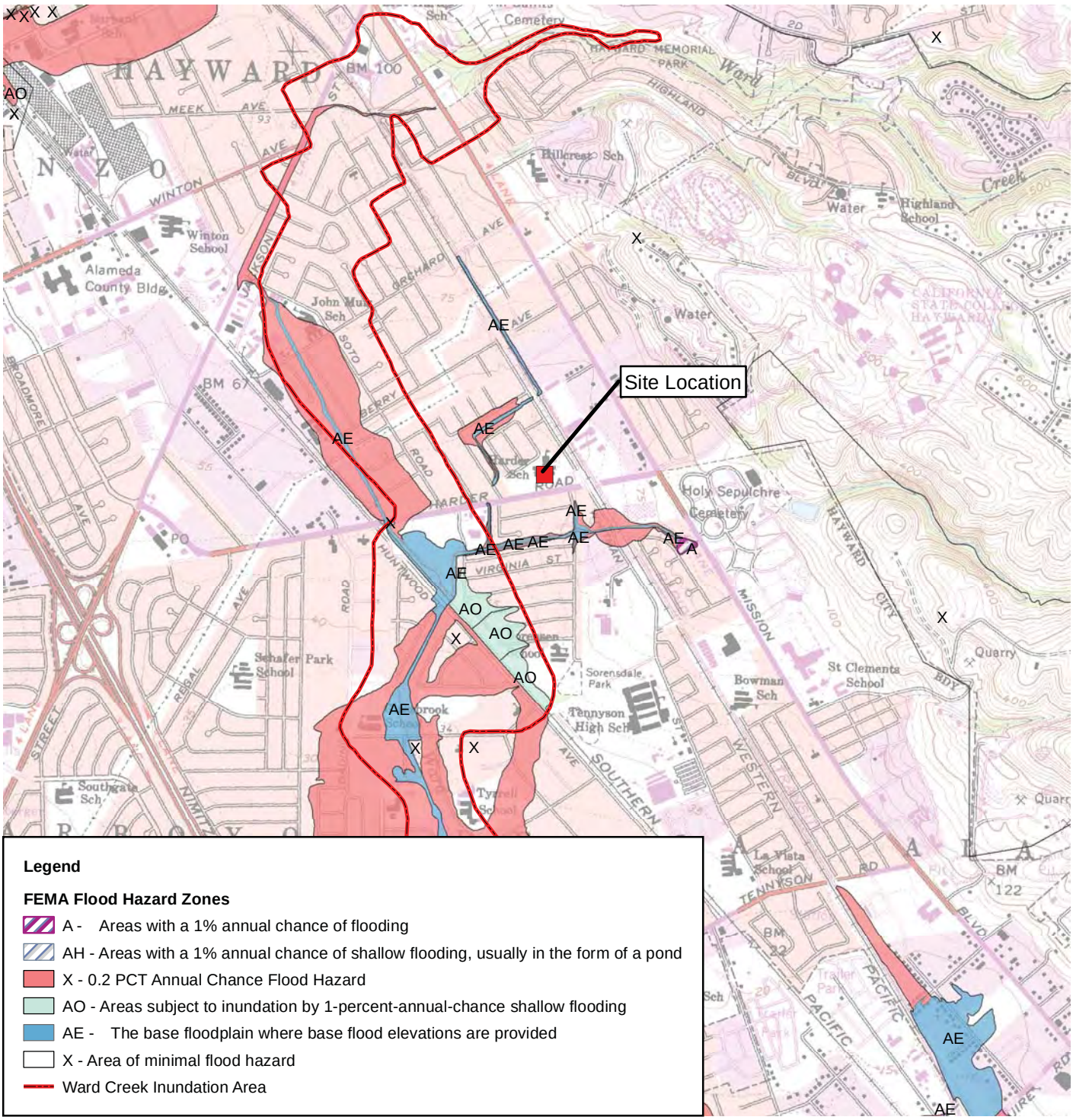
FILE NAME: Figure D-7 landslides		



Legend

FEMA Flood Hazard Zones

- A - Areas with a 1% annual chance of flooding
- AH - Areas with a 1% annual chance of shallow flooding, usually in the form of a pond
- X - 0.2 PCT Annual Chance Flood Hazard
- AO - Areas subject to inundation by 1-percent-annual-chance shallow flooding
- AE - The base floodplain where base flood elevations are provided
- X - Area of minimal flood hazard
- Ward Creek Inundation Area

Reference: FEMA Flood Hazard Layer, 06001C-NFHL, 8/03/2009
State of California OES Dam Inundation DVD

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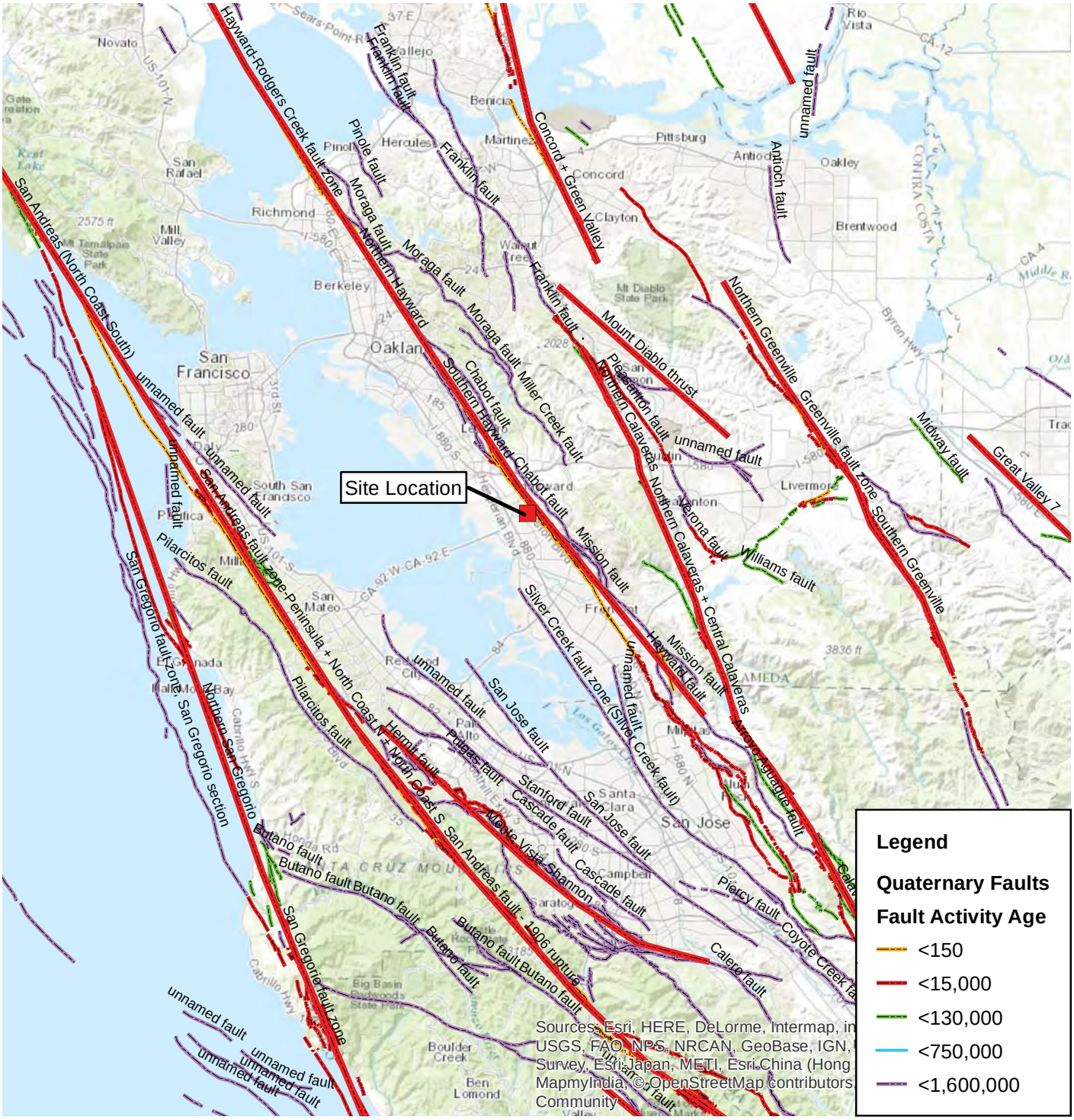
PROJECT NO. G1605411L	**FEMA FLOOD ZONES**	Figure
Drawn: 4/13/2016	**Geologic/Seismic Hazards Assessment**	
DRAWN BY: MBC		**D-9**
CHECKED BY: BS	Campus Reconstruction Harder Elementary School Hayward, California	
FILE NAME: Figure D-7 FEMA map		



Reference: USGS Quaternary Fault Database ftp://hazards.cr.usgs.gov/maps/qfault/
November 3rd, 2010

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PROJECT NO. G1605411L	**Regional Fault Map**	Figure
Drawn: 4/13/2016	**Geologic/Seismic Hazards Assessment**	
DRAWN BY: MBC		**D-10**
CHECKED BY: BS	Campus Reconstruction Harder Elementary School Hayward, California	
FILE NAME: Figure D-8 Regional Fault map		

Reference: USGS Quaternary Fault Database ftp://hazards.cr.usgs.gov/maps/qfault/
November 3rd, 2010

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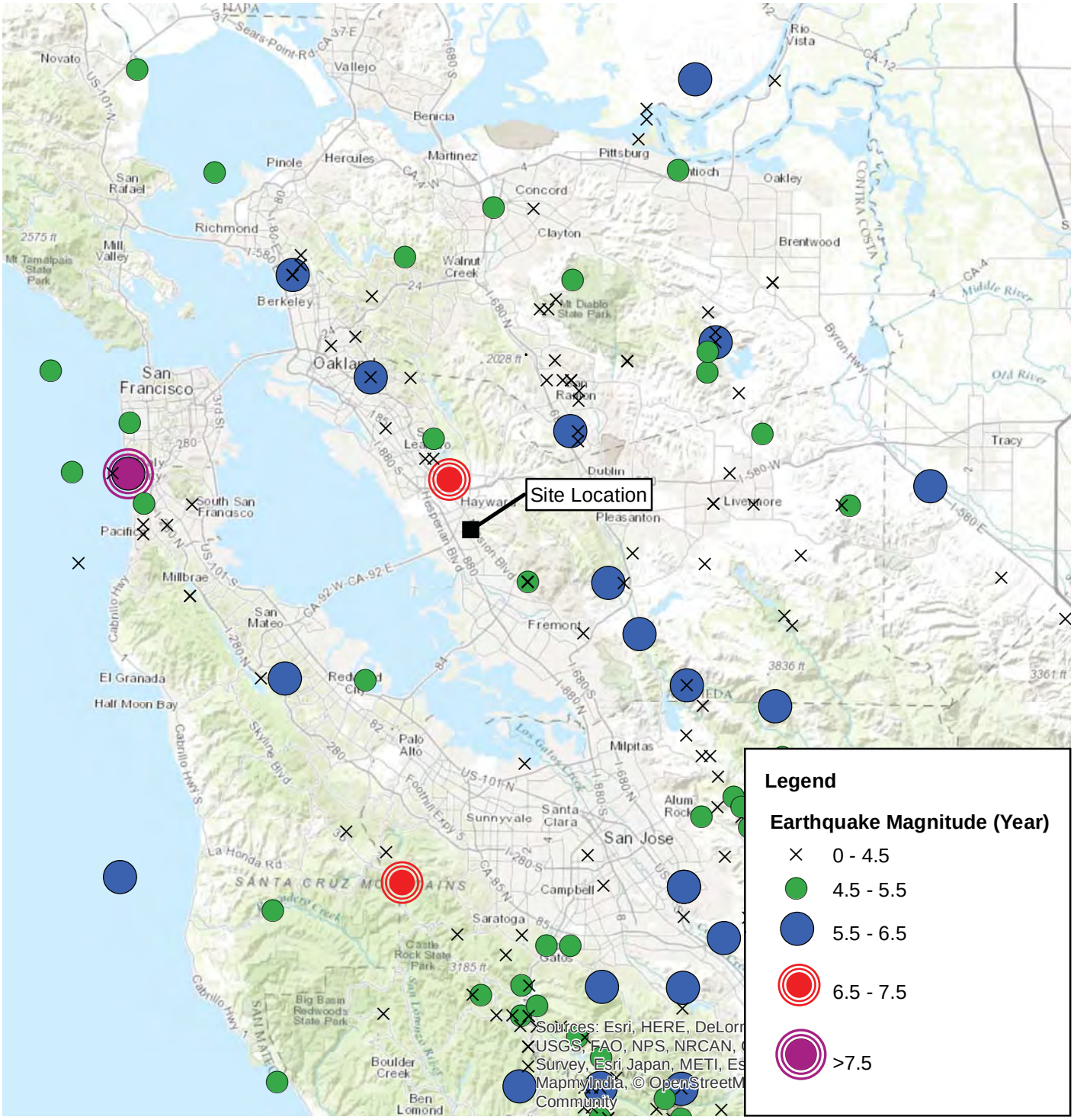
PROJECT NO. G1605411L	**Local Fault Map**	Figure
Drawn: 4/13/2016	**Geologic/Seismic Hazards Assessment**	**D-11**
DRAWN BY: MBC		
CHECKED BY: BS	Campus Reconstruction Harder Elementary School Hayward, California	
FILE NAME: Figure D-11 Local Fault map		



Reference:USGS Open File Report 2007-1437H, USGS WGCEP Historical California Earthquake Catalog, 2008

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BSK Associates Engineers & Laboratories

PROJECT NO. G1605411L	
Drawn: 4/13/2016	
DRAWN BY: MBC	
CHECKED BY: BS	
FILE NAME: Figure D-12 Historic Earthquakes map	

Historical Earthquakes

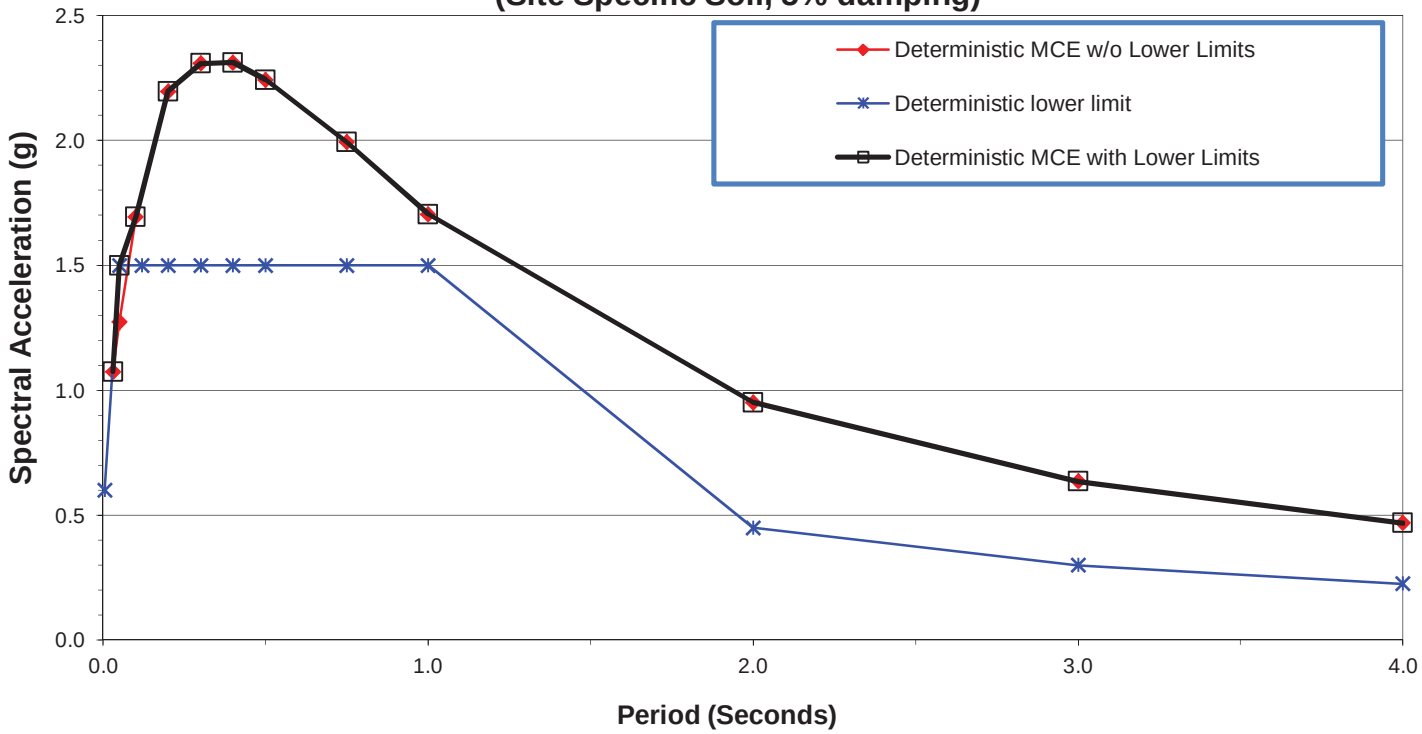
Geologic/Seismic Hazards Assessment

Campus Reconstruction
Harder Elementary School
Hayward, California

Figure

D-12

Deterministic Response Spectra
Maximum Considered Earthquake
Maximum Rotated Horizontal Component
(Site Specific Soil, 5% damping)



				Attenuation Relationship			
Period (Second)	Deterministic MCE with Lower Limits (g)	Deterministic MCE w/o Lower Limits (g)	Sa (Median) (g)	Sa (BA08) (g)	Sa (CB08) (g)	Sa (CY08) (g)	Sa (AS08) (g)
Calif. Gridded, Hayward Fault Beyond 0.4 Second							
PGA	1.075	1.075	1.075	1.176	0.891	1.175	1.064
0.05	1.500	1.273	1.273	1.483	1.054	1.415	1.154
0.1	1.694	1.694	1.694	2.038	1.371	1.868	1.516
0.2	2.196	2.196	2.196	2.565	1.764	2.351	2.104
0.3	2.308	2.308	2.308	2.743	1.830	2.369	2.290
0.4	2.312	2.312	2.312	2.810	1.891	2.267	2.280
0.5	2.242	2.242	2.242	2.483	1.963	2.422	2.102
0.75	1.992	1.992	1.992	2.174	1.868	2.114	1.811
1.0	1.704	1.704	1.704	1.725	1.664	1.879	1.547
2.0	0.951	0.951	0.951	0.978	0.996	0.992	0.838
3.0	0.636	0.636	0.636	0.678	0.675	0.646	0.544
4.0	0.469	0.469	0.469	0.499	0.520	0.463	0.395

Notes: BA08 Boore-Atkinson (2008) NGA Maximum Rotated Horizontal Component
CB08 Campbell-Bozorgnia (2008) NGA Maximum Rotated Horizontal Component
CY08 Chiou-Youngs (2008) NGA Maximum Rotated Horizontal Component
AS08 Abrahamson-Silva (2008) NGA MRC

BSK Associates Engineers & Laboratories

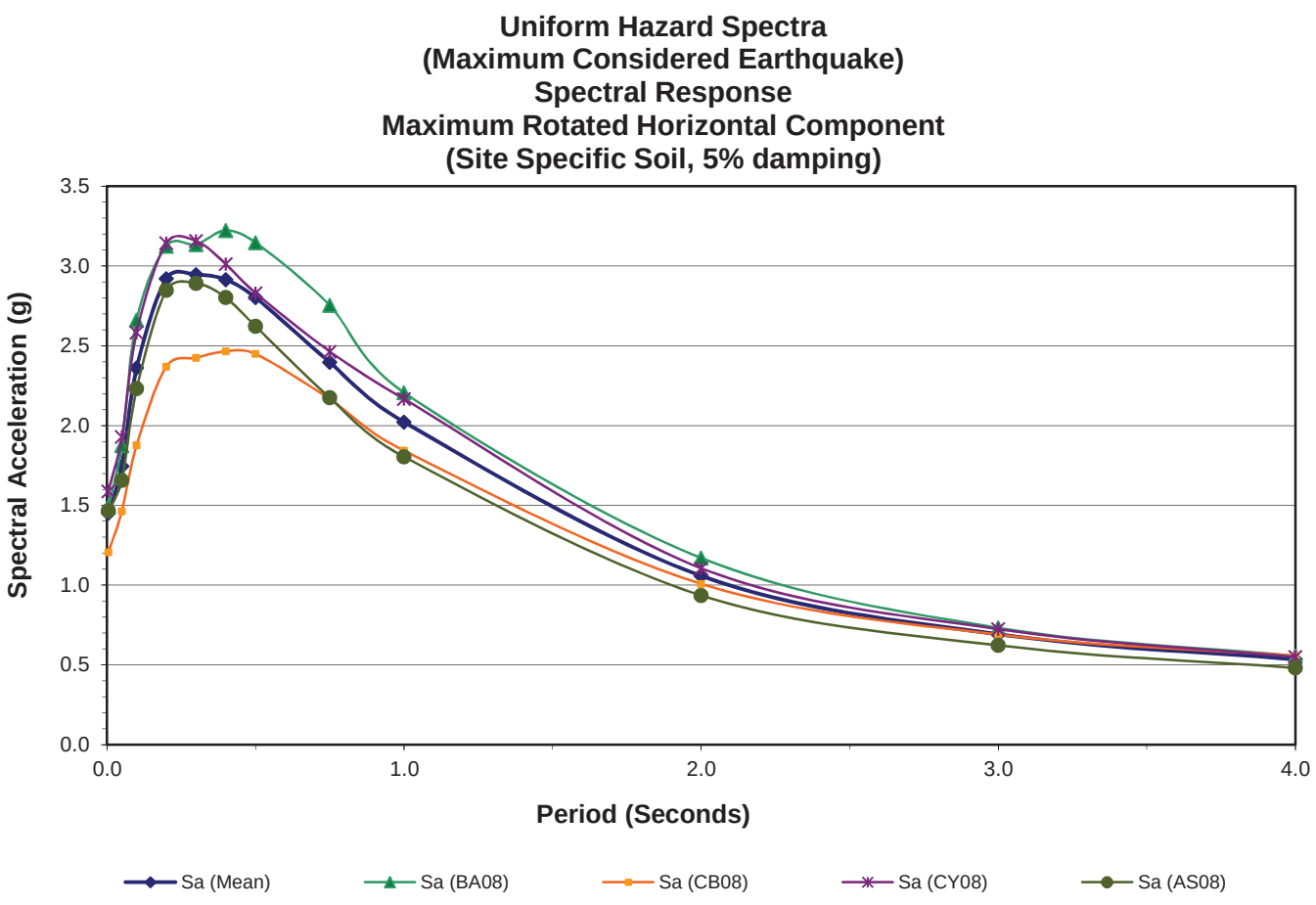
PROJECT NO. G1605411L
Date: 4/18/2016
BY: MBC
CHECKED BY: BS
FILE NAME: resp-spectra

Deterministic Response Spectra

Geologic/Seismic Hazards Assessment

Campus Reconstruction
Harder Elementary School
Hayward, California

Figure
D-13

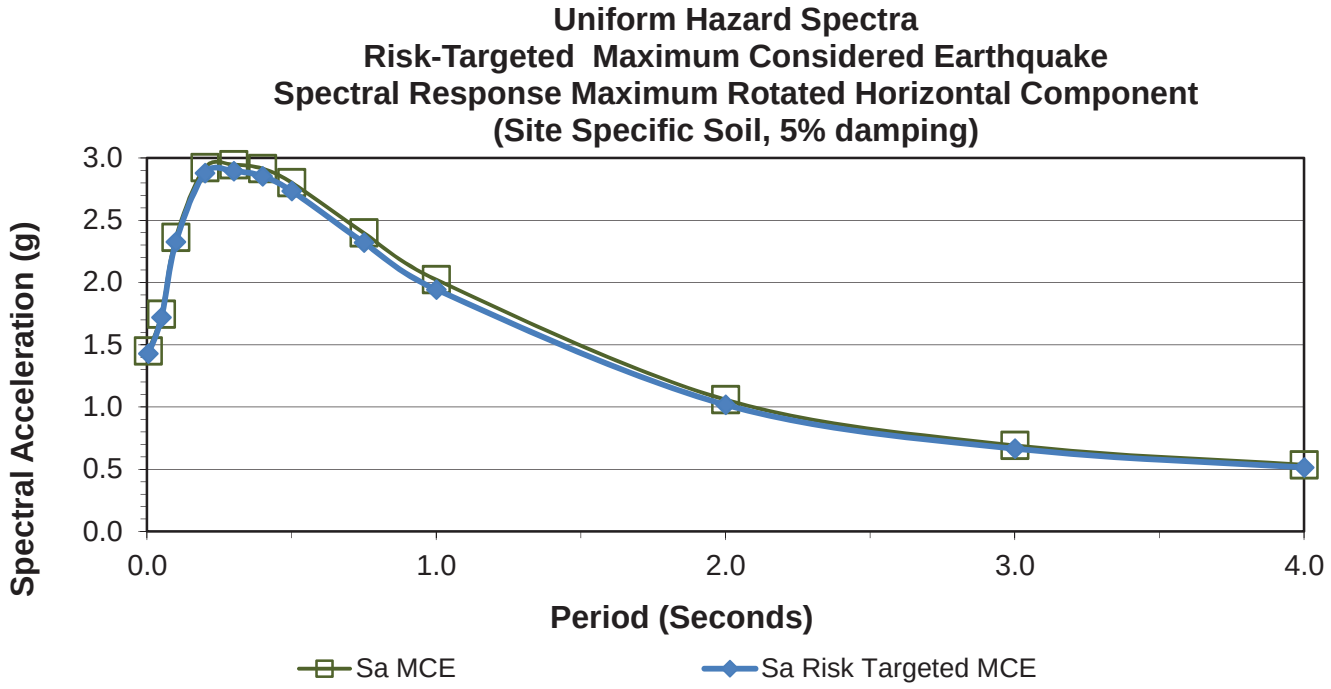


		Attenuation Relationship			
Period (Second)	Sa (Mean) (g)	(BA08) (g)	(CB08) (g)	(CY08) (g)	Sa (AS08) (g)
PGA	1.450	1.494	1.205	1.589	1.466
0.05	1.745	1.874	1.463	1.929	1.656
0.1	2.361	2.661	1.877	2.584	2.231
0.2	2.921	3.125	2.370	3.144	2.847
0.3	2.946	3.135	2.425	3.157	2.892
0.4	2.914	3.223	2.465	3.012	2.802
0.5	2.802	3.147	2.449	2.831	2.623
0.75	2.397	2.754	2.167	2.464	2.175
1.0	2.022	2.207	1.843	2.167	1.804
2.0	1.058	1.170	1.008	1.106	0.935
3.0	0.692	0.732	0.690	0.724	0.622
4.0	0.535	0.556	0.557	0.549	0.482

Notes:
BA08 Boore-Atkinson (2008) NGA MRC
CB08 Campbell-Bozorgnia (2008) NGA MRC
CY08 Chiou-Youngs (2008) NGA MRC
AS08 Abrahamson-Silva (2008) NGA MRC

BSK Associates Engineers & Laboratories

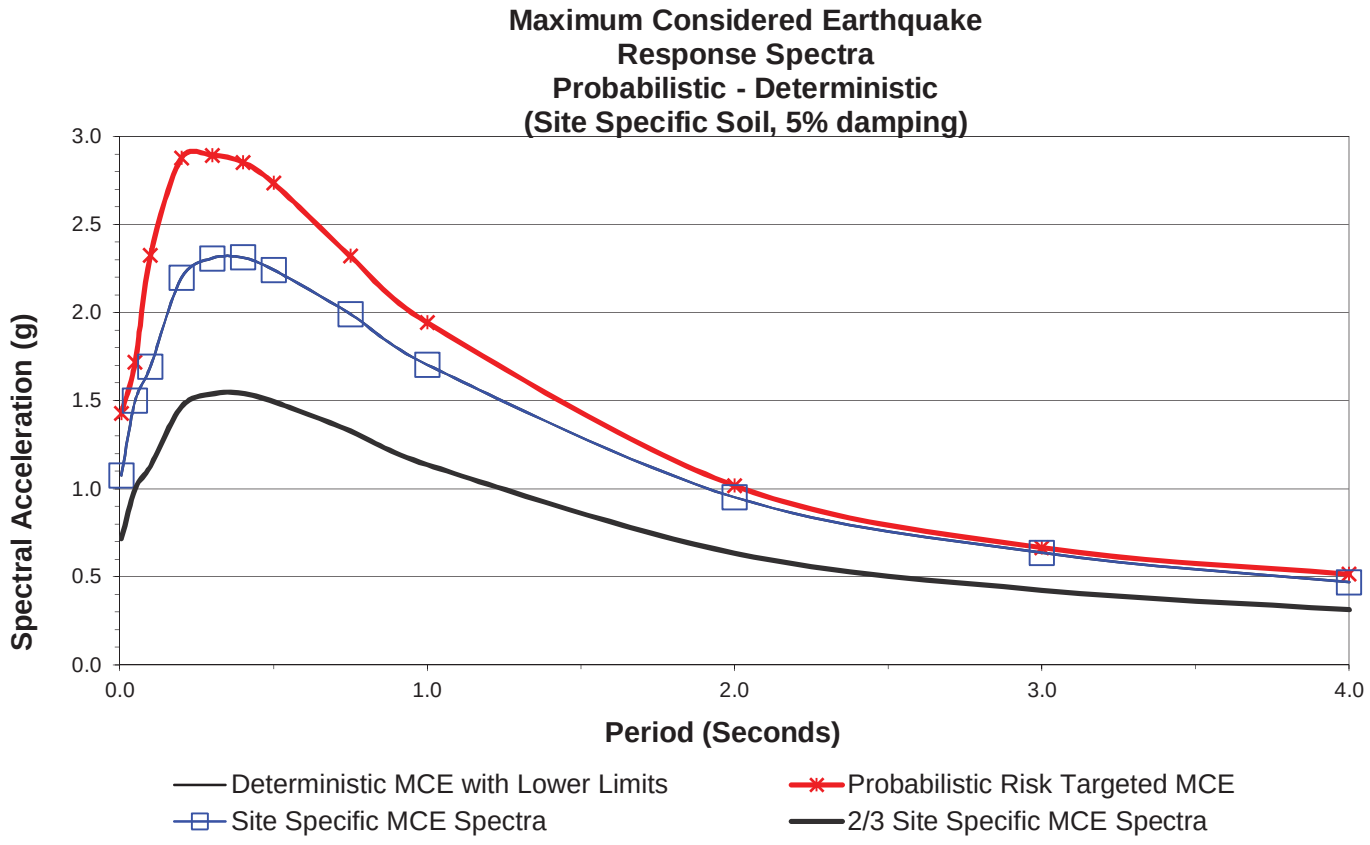
PROJECT NO. G1605411L	Uniform Hazard Spectra	Figure
Date: 4/18/2016		
BY: MBC	Geologic/Seismic Hazards Assessment	D-14
CHECKED BY: BS		
FILE NAME: resp-spectra	Campus Reconstruction Harder Elementary School Hayward, California	



Period (Second)	Sa MCE (g)	Risk Coefficent C_R	Sa Risk Targeted MCE (g)
PGA	1.450	0.985	1.428
0.05	1.745	0.985	1.719
0.1	2.361	0.985	2.326
0.2	2.921	0.985	2.877
0.3	2.946	0.982	2.893
0.4	2.914	0.979	2.853
0.5	2.802	0.976	2.735
0.75	2.397	0.968	2.320
1	2.022	0.961	1.943
2	1.058	0.961	1.017
3	0.692	0.961	0.665
4	0.535	0.961	0.514

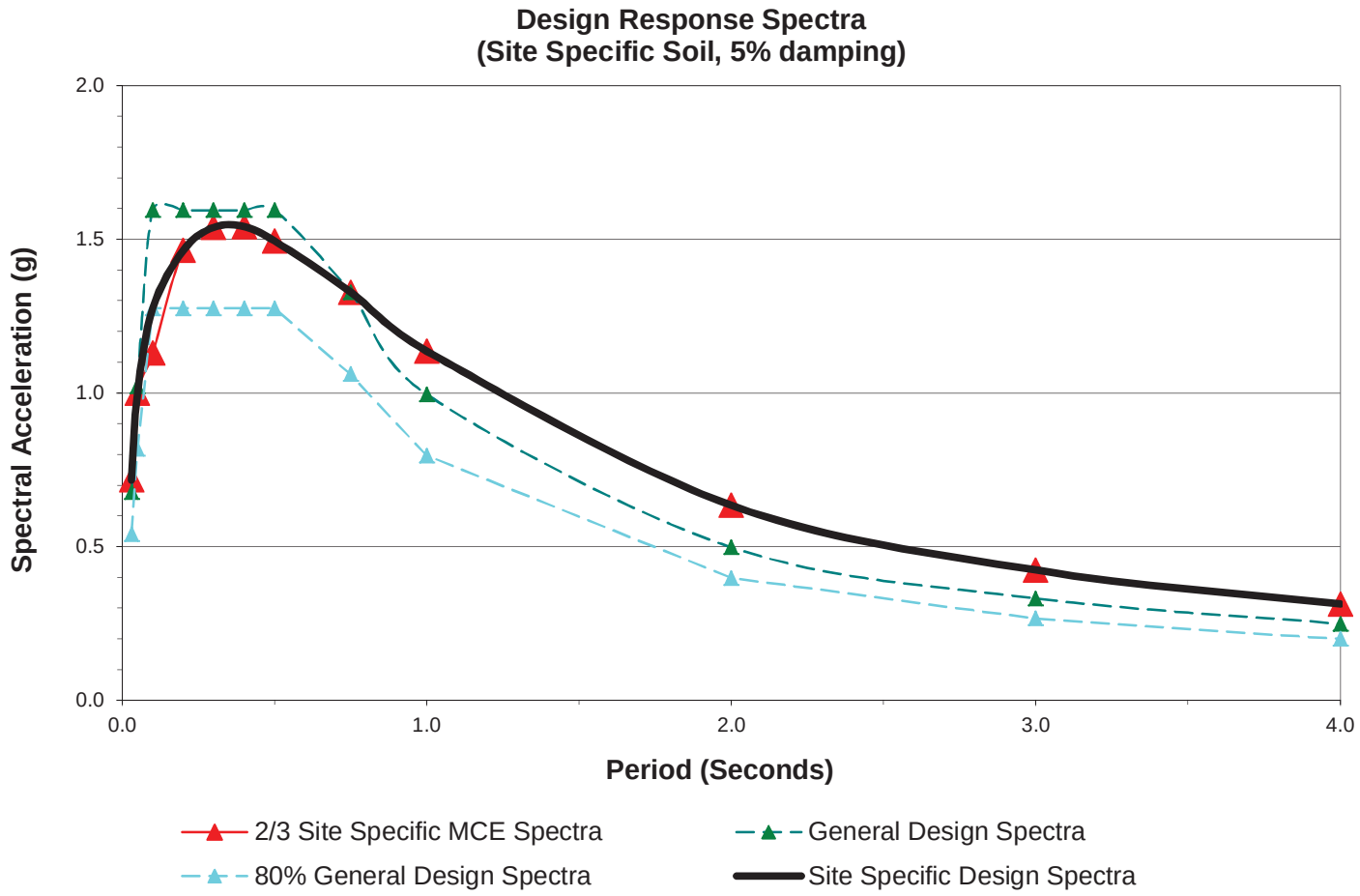
Notes: C_R From USGS Web Application

BSK Associates Engineers & Laboratories	PROJECT NO. G1605411L Date: 4/18/2016	**Uniform Hazard Spectra**	Figure
	BY: MBC CHECKED BY: BS	Geologic/Seismic Hazards Assessment	D-15
	FILE NAME: resp-spectra	Campus Reconstruction Harder Elementary School Hayward, California	



Period (Second)	Deterministic MCE with Lower Limits Sa (g)	Probabilistic Risk Targeted MCE Sa (g)	Site Specific MCE Spectra Sa (g)	2/3 Site Specific MCE Spectra Sa (g)
PGA	1.075	1.428	1.075	0.717
0.05	1.500	1.719	1.500	1.000
0.1	1.694	2.326	1.694	1.129
0.2	2.196	2.877	2.196	1.464
0.3	2.308	2.893	2.308	1.539
0.4	2.312	2.853	2.312	1.541
0.5	2.242	2.735	2.242	1.495
0.75	1.992	2.320	1.992	1.328
1.0	1.704	1.943	1.704	1.136
2.0	0.951	1.017	0.951	0.634
3.0	0.636	0.665	0.636	0.424
4.0	0.469	0.514	0.469	0.313

BSK Associates Engineers & Laboratories	PROJECT NO. G1605411L Date: 4/18/2016	**MCE Response Spectra**	Figure
	BY: MBC CHECKED BY: BS	Geologic/Seismic Hazards Assessment	D-16
	FILE NAME: resp-spectra	Campus Reconstruction Harder Elementary School Hayward, California	



Period (Second)	2/3 Site Specific MCE Spectra Sa (g)	General Design Spectra Sa (g)	80% General Design Spectra Sa (g)	Site Specific Design Spectra Sa (g)
PGA	0.717	0.676	0.541	0.717
0.05	1.000	1.021	0.817	1.000
0.1	1.129	1.595	1.276	1.276
0.2	1.464	1.595	1.276	1.464
0.3	1.539	1.595	1.276	1.539
0.4	1.541	1.595	1.276	1.541
0.5	1.495	1.595	1.276	1.495
0.8	1.328	1.327	1.061	1.328
1.0	1.136	0.995	0.796	1.136
2.0	0.634	0.498	0.398	0.634
3.0	0.424	0.332	0.265	0.424
4.0	0.313	0.249	0.199	0.313

Notes: General=General Response Spectrum based on 2013 CBC

BSK Associates Engineers & Laboratories	PROJECT NO. G1605411L Date: 4/18/2016	**Design Response Spectra**	Figure
	BY: MBC CHECKED BY: BS	Geologic/Seismic Hazards Assessment	D-17
	FILE NAME: resp-spectra	Campus Reconstruction Harder Elementary School Hayward, California	

APPENDIX E

CORROSIVITY TEST RESULTS BY CERCO ANALYTICAL

California State Certified Laboratory No. 3

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

12 April, 2016

Job No. 1604059
Cust. No.12667

Ms. Carrie Foulk
BSK Associates Engineers & Laboratories
324 Earhart Way
Livermore, CA 94551

Subject: Project No.: G16-054-11L
Project Name: Harder Elementary School
Corrosivity Analysis – ASTM Test Methods

Dear Ms. Foulk:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on April 7, 2016. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, the sample is 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 concentration is none detected to 15 mg/kg.

The sulfate ion concentration is none detected to 15 mg/kg.

The pH of the soil is 7.90 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 370-mV which is 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.

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

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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.: G16-054-11L
Client's Project Name: Harder Elem. School, Hayward
Date Sampled: 31-Mar-16
Date Received: 7-Apr-16
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 12-Apr-2016

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)*
1604059-001	B4 Bulk @ 1'-2'	370	7.90	-	1,600	-	N.D.	N.D.
Method:		ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:		-	-	10	-	50	15	15
		11-Apr-2016	11-Apr-2016	-	12-Apr-2016	-	11-Apr-2016	11-Apr-2016

Cheryl McMillen
Laboratory Director

* 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 F

EXHIBIT 1 – SUMMARY OF COMPACTION RECOMMENDATIONS

EXHIBIT 1
SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendations (See Notes 1, 2, 3, 4, 6)
Subgrade Preparation and Placement of General Engineered Fill, Including Imported Fill	Compact upper 12 inches of subgrade and entire fill to a minimum of 90 percent compaction at 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.
Lime-Treated Soil	Compact lime-treated on-site soils to a minimum of 90 percent compaction and at least 3 percent over optimum moisture content.
Trenches[5]	Compact trench backfill 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. Where trenches will be under flatwork or paving, the upper 12 inches should be compacted as recommended below for flatwork and pavement. Proper granular bedding and shading should be used beneath 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 baserock to a minimum of 90 percent compaction at near optimum moisture content. Where exterior flatwork is exposed to vehicular traffic, compact upper 12 inches of subgrade to a minimum of 92 percent compaction and baserock to a minimum of 95 percent compaction.
AC & Concrete 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 baserock 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 D 1557.

(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 <u>only</u>, the percent compaction in trenches may be reduced to 85 percent.

(6) Where fills are greater than 7 feet in depth below finish grade, the portion below a depth of 7 feet should be compacted to a minimum of 95 percent compaction.

References: 1. http://earth.google.com, 2016
2. Option A- Site Plan, Harder Elementary 8-29-18 by Lionakis.

Note: Locations are approximate

Legend

B-1 Approximate Soil Boring Location (BSK, 2016)

CPT-1 Approximate CPT Location (BSK, 2016)

Approximate footprint & layout of planned campus reconstruction

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BSK ASSOCIATES

PROJECT NO. G16-054-11L	SITE PLAN	FIGURE
DRAWN: 11/30/18		1
DRAWN BY: D. Tower		
CHECKED BY: C. Foulk	Campus Reconstruction Harder Elementary School 495 Wyeth Road Hayward, California	
FILE NAME: Lab.indd		