Geologic Hazard and Geotechnical Report

Proposed New Agricultural Building
Eureka High School
Eureka, California

Prepared for:

Eureka City Schools

May 2018
016137.100

SHN 812 W. Wabash Ave., Eureka, CA 95501-2138 707-441-8855
Civil Engineering, Environmental Services, Geosciences, Planning & Permitting, Surveying

Reference: 016137.100

May 8, 2018

Mr. Paul Ziegler
Eureka City Schools
2100 J Street
Eureka, CA 95501

Subject: **Geologic Hazard and Geotechnical Report, Proposed New Agricultural Building, Eureka High School, Eureka, California**

Dear Mr. Ziegler:

In accordance with your authorization, SHN has performed a geotechnical investigation and geologic hazard evaluation for the proposed new Agricultural Building located at the Eureka High School campus in Eureka, California as required by the State of California Division of the State Architect. The enclosed report presents the results of our field exploration and laboratory testing, as well as conclusions and recommendations to assist the project design consultants in the design and construction of the new building. The report is intended to comply with criteria presented in *California Geological Survey, Note 48: Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings*, dated October 2013.

Sincerely,

SHN Engineers & Geologists

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JHD:JPB:lms

Enclosure: Report

Reference: 016137.100

Geologic Hazard and Geotechnical Report

Proposed New Agricultural Building
Eureka High School
Eureka, California

Prepared for:
Eureka City Schools

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May 2018

QA/QC:GDS_____

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List of Illustrations

Abbreviations and Acronyms

g	acceleration of gravity
km	kilometers
m	meter
mg/kg	milligrams per kilogram
AAAS	American Association for the Advancement of Science
AAPG	American Association of Petroleum Geologists
ABS	acrylonitrile butadiene styrene (pipe)
Ag	Agriculture
ASCE	American Society of Civil Engineers
ASTM	ASTM-International
B-#	boring-number
BGC	Busch Geotechnical Consultants
BGS	below ground surface
CBC	California Building Code
CDMG	California Division of Mines and Geology
CL	clays
CRR	cyclic resistance ratio
CSR	cyclic stress ratio
CSZ	Cascadia Subduction Zone
H	height
H:V	horizontal to vertical (ratio)
ICBO	International Conference of Building Officials

mm/yr	millimeters per year
pcf	pounds per cubic foot
psf	pounds per square foot
M#	magnitude number
MCE_G	maximum considered earthquake
ML	clayey silts
ML-OL	peaty silt
Mw	maximum earthquake magnitude
NCEER	National Center for Earthquake Engineering Research
NR	no reference
OSHA	Occupational Safety Health Administration
PG&E	Pacific Gas & Electric
PI	Plasticity Index
PVC	polyvinyl chloride
Qh	Hookton Formation
Qmt	Marine Terrace Deposits
S_1	spectral parameter
SC	sands/clayey sands
SDR	standard dimension ratio
SM	silty sands
SPT	Standard Penetration Test
UBC	Uniform Building Code
USGS	United States Geological Survey

1.0 Introduction

This report documents the results of geologic and geotechnical investigations conducted by SHN for the proposed new Agricultural (Ag) Building at the Eureka High School Campus (Figures 1 and 2). This report is intended to provide the district with findings, conclusions, and recommendations related to geologic and geotechnical aspects of project design and construction. The recommendations contained in this report are subject to the limitations presented herein. Attention is directed to “Section 9.0: Additional Services and Limitations” of this report.

The proposed project consists of the construction of a new 10,000-square foot, single-story, wood-framed structure. The proposed project location is within the main portion of the campus currently being used as a parking lot.

Site grading for development of the new building is anticipated to be minimal as it is within a relatively flat portion of the campus (that is, no cuts or fills over a few feet in depth or height would be required).

SHN prepared geologic hazard/geotechnical documents in 2016 and 2004 for a series of proposed projects, including a replacement gym, SHN 2004a, 2004b, and 2004c. We have also been provided previous geotechnical reports by Busch Geotechnical Consultants (BGC) that are applicable to the Eureka High Campus. These are BGC, 1995, 1996a-e, 1998, 1999, 2000, 2003a, 2003b. (See “Section 11.0: References Cited” of this report.)

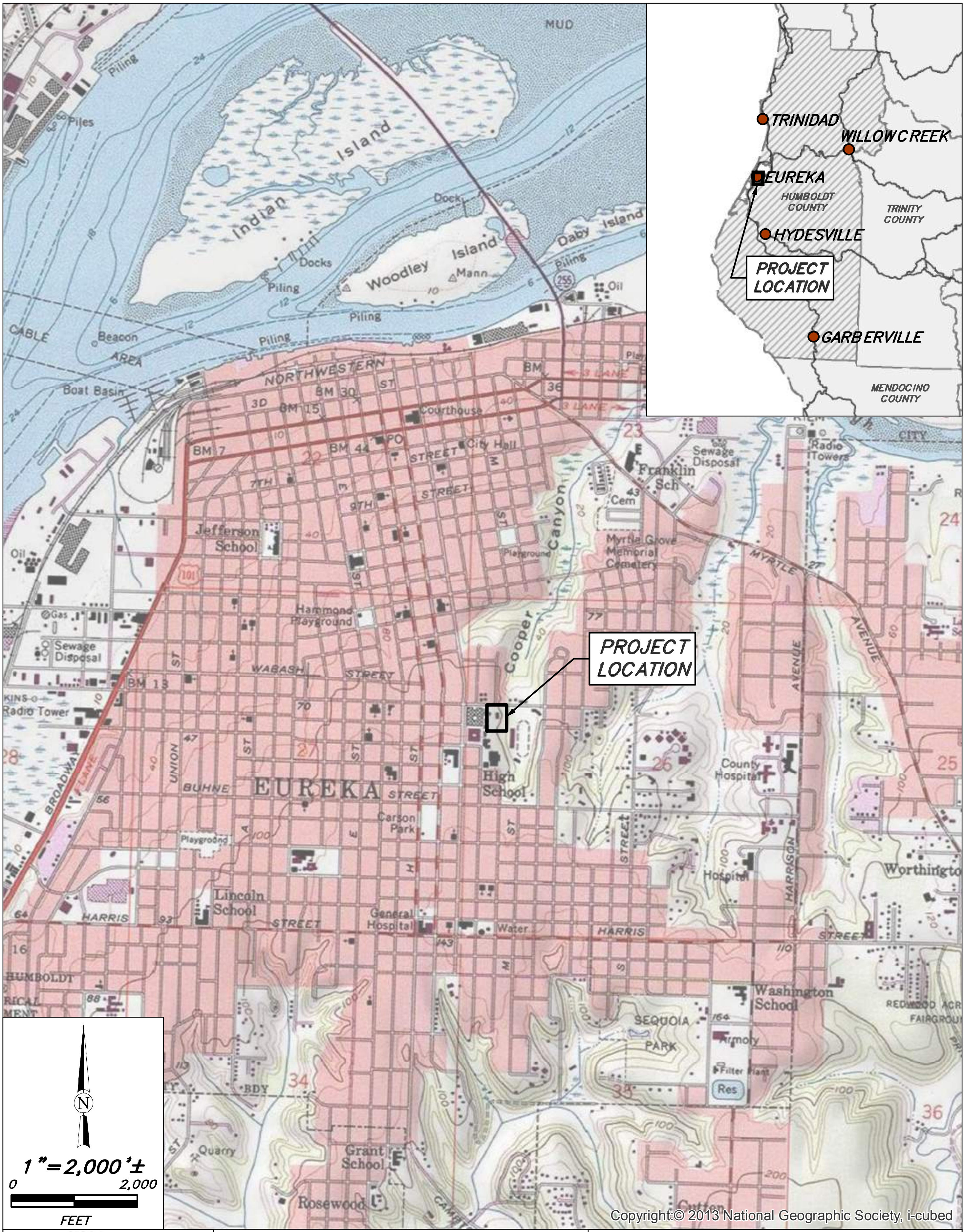
2.0 Geologic Setting

2.1 General

The Eureka/Humboldt Bay region occupies a complex geologic environment characterized by very high rates of active tectonic deformation and seismicity. The area lies just north of the Mendocino Triple Junction (Figure 3), the intersection of three crustal plates (the North American, Pacific, and Gorda plates). North of Cape Mendocino, the Gorda plate is being actively subducted beneath North America, forming what is commonly referred to as the Cascadia Subduction Zone (CSZ). In the Humboldt Bay region, secondary deformation associated with plate convergence is manifested on-land as a series of northwest-trending, southwest-vergent thrust faults, and intervening folds (fold and thrust belt). The geomorphic landscape of the Humboldt Bay region is largely a manifestation of the active tectonic processes and the setting in this dynamic coastal environment.

Basement rock beneath Humboldt Bay and the City of Eureka is the Paleocene-Eocene Yager terrane, a part of the Coastal belt of the Franciscan Complex (Blake et al., 1985; Clarke, 1992). The Franciscan Complex is a regional bedrock unit that consists of a series of "terranes,” which are discrete blocks of highly deformed oceanic crust that have been welded to the western margin of the North American plate over the past 140 million years. The Yager terrane consists of as much as 9,800 feet of well-indurated marine mudstone and thin-bedded siltstone. Yager terrane bedrock is in excess of 1,000 feet below the ground surface (BGS) in the vicinity of Humboldt Bay, based on a deep exploratory well about 4 miles southeast of the City wastewater treatment plant (Woodward-Clyde Consultants, 1980).

Basement rock in the Humboldt Bay region is unconformably overlain by a late Miocene to middle Pleistocene age sequence of marine and terrestrial deposits referred to as the Wildcat Group (Ogle, 1953). The marine portion of the Wildcat Group includes some 6,000 to 8,000 feet of mudstone and lesser amounts of sandstone that were deposited in a deep coastal basin (that is, an earlier version of the Eel River



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Project Location

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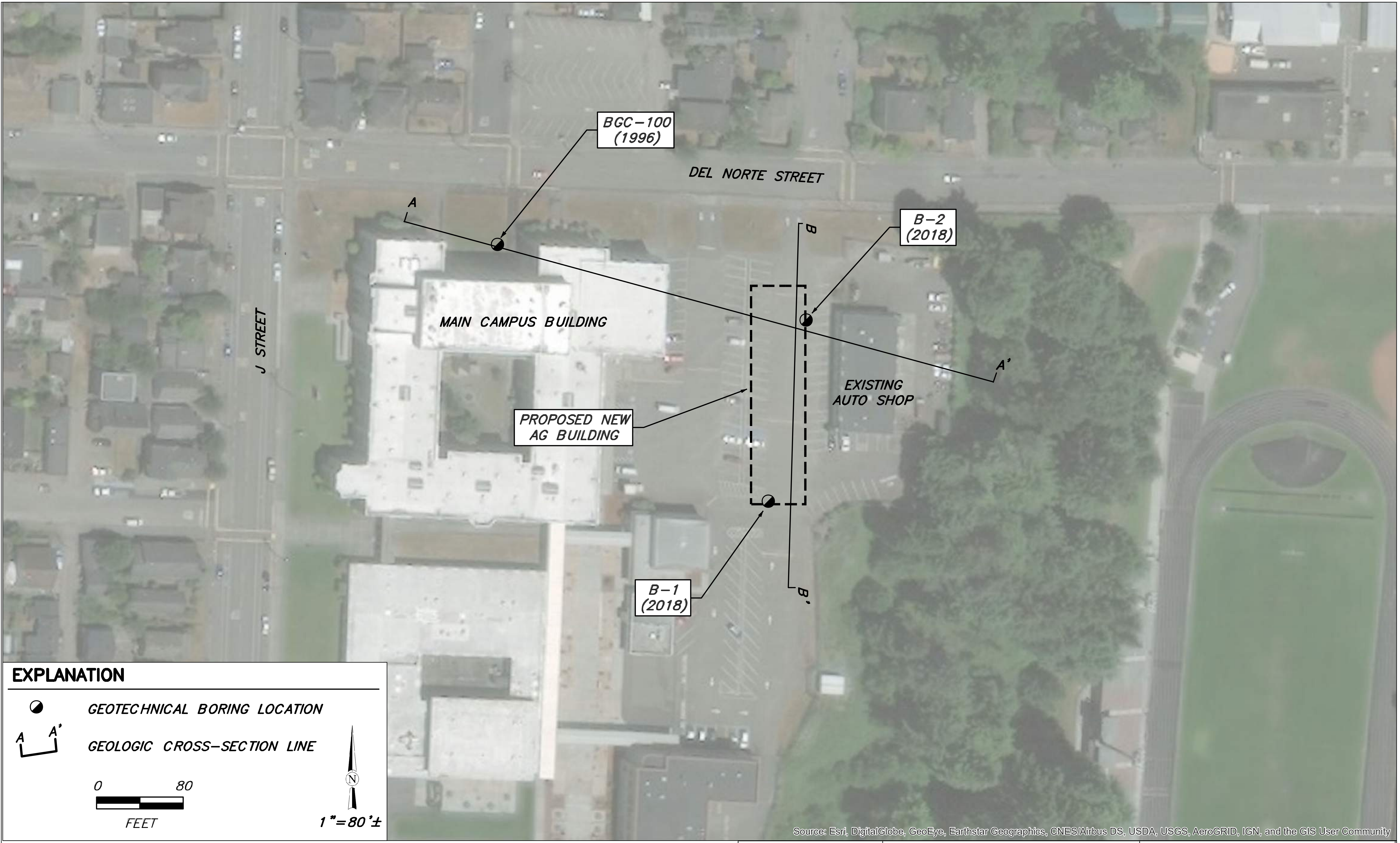
Figure1_ProjectLocationMap

Figure 1



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

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Site Map Showing
Boring Locations
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Figure2_SiteMap

Figure 2

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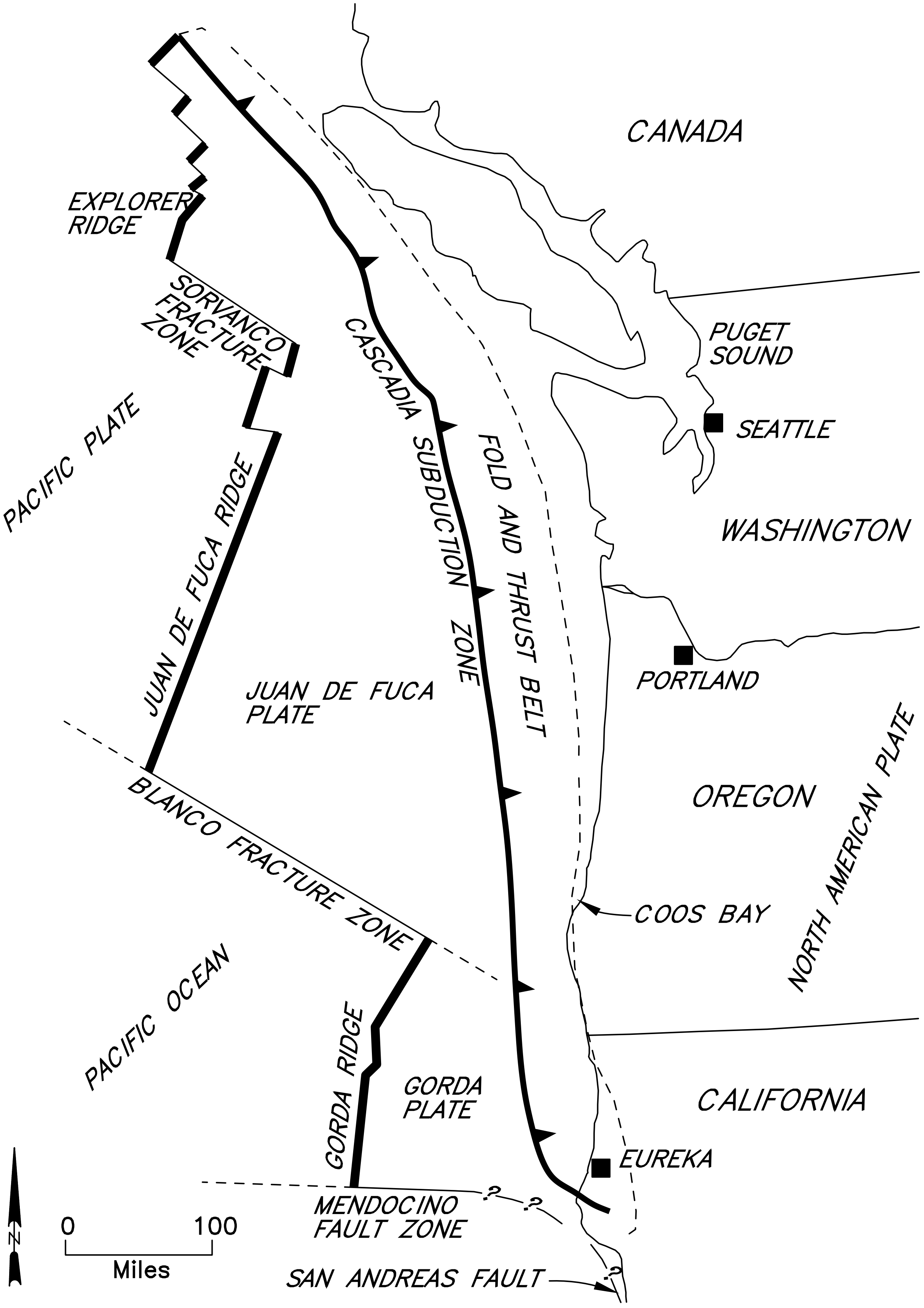

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Regional Tectonic Setting

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Figure3_RegionalTectonicSetting.pdf

Figure 3

basin). Gradationally overlying the marine portion of the Wildcat Group are 2,500 to 3,250 feet of nonmarine sandstone and conglomerate, which represent the uppermost part of the Wildcat depositional sequence. The Wildcat Group is truncated at its top by an unconformity of middle Pleistocene age, and is overlain by coastal plain and fluvial deposits of middle to late Pleistocene age. In the Eureka area, these middle and late Pleistocene age deposits are referred to as the Hookton Formation (Ogle, 1953). Hookton Formation sediments are described as gravel, sand, silt, and clay, which have a characteristically yellow-orange color.

Along the coast of northern California between Cape Mendocino on the south and Big Lagoon, about 60 miles (100 kilometers [km]) to the north of the Eureka High School campus, a sequence of uplifted late Pleistocene age marine terraces is preserved The terraces are preserved as erosional remnants of raised shore platforms and associated cover sediments. Sea level has fluctuated throughout the late Pleistocene in response to the advance and retreat of large continental ice sheets. Marine terraces preserved along the coast represent surfaces eroded during the highest levels of these sea level fluctuations, superimposed on a coastline being uplifted by regional tectonics. Marine terraces in the region range in age from about 64,000 years old, to as much as 240,000 years old.

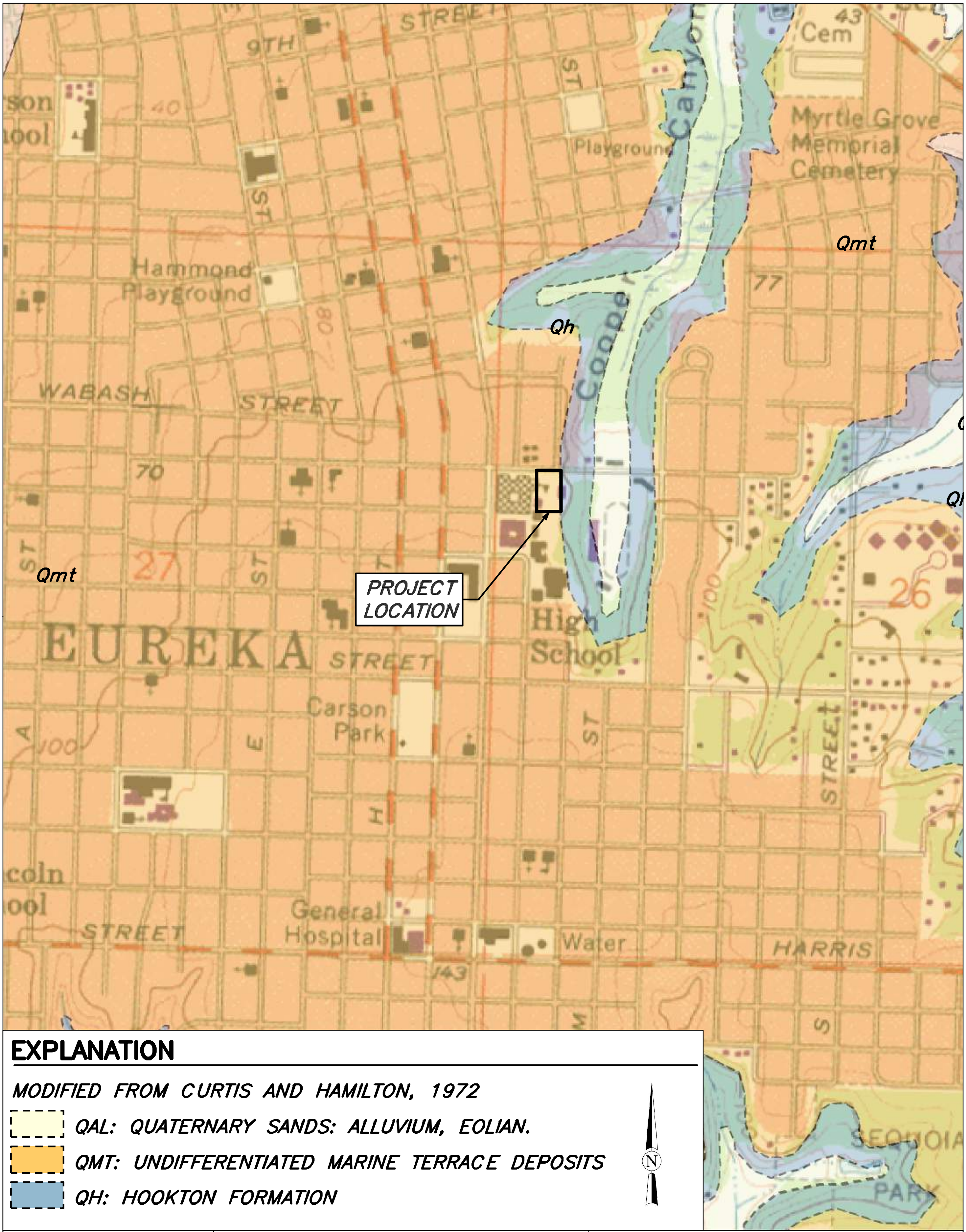
The City of Eureka occupies a series of northward-dipping marine terrace surfaces eroded onto the Hookton Formation (Figure 4). The ages of the individual terrace surfaces in the Eureka area are poorly constrained, and individual surfaces have not, to date, been accurately mapped. Marine terraces in the study area are associated with 20 to 30+ feet of predominantly silty sand covering the abrasion platform (for example, “marine terrace deposits” in this report).

The footprint of the proposed Ag Building is situated on a marine terrace surface loosely correlated with the McKinleyville terrace, which is reported by Carver and Burke (1992) to be associated with marine isotope stage 5b with an approximate age of 100,000 years. The site is adjacent to an erosional stream valley or gulch, within which Albee Stadium is located (Figure 4). The gulch is the head of Cooper Canyon, which extends to the north toward the Eureka slough at the north edge of Eureka (Figure 1). The gulch penetrates beneath the depth of the marine terrace sediment veneer, into the Hookton Formation.

2.2 Seismic Setting

The project site is located in a region of high seismicity. More than 60 earthquakes have produced discernible damage in the region since the mid-1800s. Historical seismicity and paleoseismic studies in the area suggest there are six distinct sources of damaging earthquakes in the Eureka region (Figures 3 and 5): 1) the Gorda Plate, 2) the Mendocino fault, 3) the Mendocino Triple Junction, 4) the northern end of the San Andreas fault, 5) faults within the North American Plate (including the Mad River fault zone), and 6) the Cascadia Subduction Zone (Dengler et al., 1992).

Earthquakes originating within the Gorda Plate account for the majority of historic seismicity. These earthquakes occur primarily offshore along left-lateral faults; they are generated by the internal deformation within the plate as it moves toward the CSZ. Significant historic Gorda Plate earthquakes have ranged from magnitude (M)5 to M7.5. The November 8, 1980, earthquake (M7.2) was generated 30 miles (48 km) off the coast of Trinidad on a left-lateral fault within the Gorda Plate.



EXPLANATION

MODIFIED FROM CURTIS AND HAMILTON, 1972

🟨 *QAL: QUATERNARY SANDS: ALLUVIUM, EOLIAN.*

🟧 *QMT: UNDIFFERENTIATED MARINE TERRACE DEPOSITS*

🟦 *QH: HOOKTON FORMATION*

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Geologic Map
(Curtis and Hamilton, 1972)
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Figure4_GeologyMap

Figure 4

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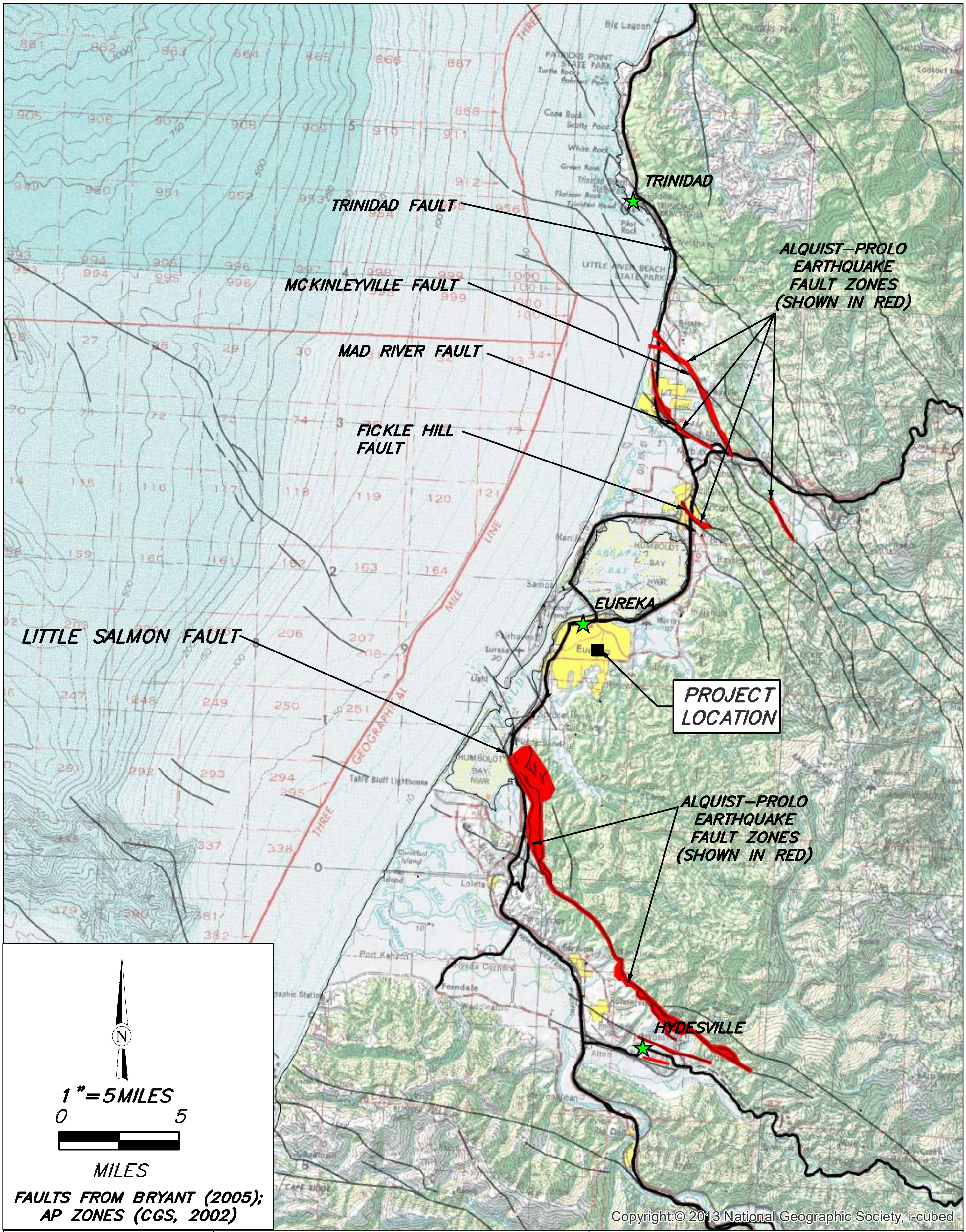

FAULTS FROM BRYANT (2005); AP ZONES (CGS, 2002)

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May 2018	Figure5_RegionalFaultMap	Figure 5

The Mendocino fault is the second most frequent source of earthquakes in the region. The fault represents the plate boundary between the Gorda and Pacific plates, and typically generates right lateral strike-slip displacement. Significant historic Mendocino fault earthquakes have ranged from M5 to M7.5. The September 1, 1994, M7.2 event originating west of Petrolia was generated along the Mendocino fault. The Mendocino triple junction was identified as a separate seismic source only after the M6.0 August 17, 1991, earthquake. Significant seismic events associated with the triple junction are shallow onshore earthquakes that appear to range from M5 to M6. Raised Holocene age marine terraces near Cape Mendocino suggest larger events are possible in this region.

Earthquakes originating on the northern San Andreas fault are extremely rare, but can be very large. The northern San Andreas fault is a right lateral strike-slip fault that represents the plate boundary between the Pacific and North American plates. The fault extends through the Point Delgada region and terminates at the Mendocino triple junction. The 1906 San Francisco earthquake (M8.3) caused the most significant damage in the north coast region, with the possible exception of the April 1992 Petrolia earthquake (Dengler et al., 1992).

Earthquakes originating within the North American plate can be anticipated from a number of intraplate sources, including the Mad River fault zone and Little Salmon fault (Figure 5). There have been no large magnitude earthquakes associated with faults within the North American plate, although the December 21, 1954, M6.5 event may have occurred in the Mad River fault zone. Damaging North American plate earthquakes are expected to range from M6.5 to M8. The Little Salmon fault appears to be the most active fault in the Humboldt Bay region, and is capable of generating very large earthquakes.

2.3 Regional Faults

As noted above, the project area is located in a region that has numerous onshore and offshore faults; however, no known fault projects through the City of Eureka (Jennings, 1994; Hart & Bryant, 1997). Figure 5 shows the location of the regional faults relative to the City and Eureka High campus. Table 1 presents fault location and information data collected from the United States Quaternary Faults and Fold Database (USGS, 2008).

Table 1. Summary of Nearby Active Faults
Proposed New Agricultural Building,
Eureka High School, Eureka, California

Fault Name	Approximate Distance to Rupture Plane (kilometers)	Maximum Earthquake Magnitude (Mw)
Little Salmon	5.1	7.0
Mad River	13.0	7.1
Fickle Hill	10.1	6.9
McKinleyville	13.7	7.0
Table Bluff	9.9	7.0
Trinidad	20.3	7.3
Big Lagoon/Bald Mtn. Fault Zone	38.5	7.3
Cascadia Subduction Zone	19.5	8.3
Garberville/Briceland	58.0	6.9
Mendocino Fault Zone	63.5	7.4

Table 1. Summary of Nearby Active Faults
Proposed New Agricultural Building,
Eureka High School, Eureka, California

Fault Name	Approximate Distance to Rupture Plane (kilometers)	Maximum Earthquake Magnitude (Mw)
San Andreas	63.1	7.6
Lake Mountain	80.3	6.7
Maacama	94.3	7.1

Little Salmon Fault. The Little Salmon fault is the closest known active fault to the project area (Wills, 1990). The Little Salmon fault is a northwest-trending, southwest-vergent reverse fault (the northeast side of the fault slides up and over the southwest side of the fault along a northeast-dipping fault plane). The Bay Entrance and Buhne Point faults near King Salmon (identified during detailed studies for the Humboldt Bay Power Plant) may be secondary strands of the Little Salmon fault (Woodward-Clyde Consultants, 1980). Humboldt Hill, directly northeast of the fault, appears to be a fold in the hanging wall directly above the Little Salmon fault.

Offset relations within the upper Wildcat Group suggest vertical separation exceeds 5,900 feet (1,800 meters), representing about 4.4 miles (7 km) of dip-slip motion on the Little Salmon fault since the Quaternary (in the past 700,000 to 1 million years) (Woodward-Clyde Consultants, 1980). Paleoseismic studies of the Little Salmon fault indicate that the fault deforms late Holocene sediments at the southern end of Humboldt Bay (Carver and Clarke, 1992). Estimates of the amount of fault slip for individual earthquakes along the fault range from 15 to 23 feet (4.5 to 7 meters [m]). Radiocarbon dating suggests that earthquakes have occurred on the Little Salmon fault about 300, 800, and 1,600 years ago. Average slip rate for the Little Salmon fault for the past 6,000 years is between 6 and 10 millimeters per year (mm/yr). Based on currently available fault parameters, the maximum magnitude earthquake (Mw) for the Little Salmon fault is thought to be between 7.0 (CDMG/USGS, 1996) and 7.3 (Geomatrix Consultants, 1994).

Cascadia Subduction Zone. The Cascadia Subduction Zone (CSZ) represents the most significant potential earthquake source in the north coast region. The CSZ is the location where the oceanic crust of the Gorda and Juan de Fuca plates are being subducted beneath continental crust of the North American Plate. A great subduction event may rupture along 200 km or more of the coast from Cape Mendocino to British Columbia, may be up to M9.5, and could result in extensive tsunami inundation in low-lying coastal areas. The April 25, 1992, Petrolia earthquake (M7.1) appears to be the only recorded historic earthquake involving slip along the subduction zone, but this event was confined to the southernmost portion of the fault. It is estimated that there have been six significant subduction zone events along the CSZ in the last 3,000 years. Paleoseismic studies along the subduction zone suggest that great earthquakes are generated along the zone every 300 to 500 years. Historical records from Japan describing a tsunami thought to have originated along the CSZ suggest the most recent great subduction event occurred on January 27, 1700. A great subduction earthquake would generate long duration, very strong ground shaking throughout the north coast region.

The CSZ is located offshore, west of the north coast region. Available mapping indicates that the surface expression of the subduction zone is located some 35 to 40 miles west of the project site (Clarke, 1992; McLaughlin et al., 2000). Seismic profiles suggest that the subduction interface dips landward at an angle of about 11 degrees (McPherson, 1992), which would place it at a depth of 7 to 8 miles beneath the project area.

North Spit Fault. The North Spit fault was identified in seismic profiles offshore of the North Spit, west of Humboldt Bay (Earth Sciences Associates, 1975); it may be a part of the Little Salmon fault system. However, the fault's existence or extent is uncertain, because it was not imaged in seismic profiles farther offshore (McCulloch and others, 1977), and it has never been identified on land. Despite its uncertainty, the fault is relevant to this project, because its mapped projection is relatively close to the project area (about 2.7 miles to the southwest). The fault is not recognized or zoned by the State of California as an active or potentially active fault.

2.4 Historical Seismicity

A search of historical earthquake records was performed using the U.S. Geologic Survey (USGS) Preliminary Determinations of Epicenters Catalog on the USGS web site. Our search included historical data from 1918 to the present.

A total of 52 earthquake records were identified with a magnitude greater than M5.0 within a 100-km radius around the site. The largest earthquake events included a M7.2 in 1980 offshore of Trinidad, approximately 40 km to the north of the Eureka High School campus; and a M7.1 in 1992 near Petrolia, approximately 40 km to the south of Eureka High School. The closest earthquake greater than a M5.0 was a M6.4 in 1932 approximately 8 km to the southwest. A map showing regional historical seismicity from 1918 to present is included as Figure 6.

2.5 Stratigraphy/Earth Materials

2.5.1 General

The following are descriptions of earth materials present at the Eureka High campus. The distribution of geologic materials is shown on the geologic map (Figure 4). Geologic cross-sections of the site are shown in Figures 7 and 8. Deeper machine borings on the upper marine terrace surface are inferred to have penetrated through the late Pleistocene age marine terrace deposits, into underlying Hookton Formation sediments.

2.5.2 Hookton Formation (Qh)

Middle to Late Pleistocene age Hookton Formation sediments are present at depth beneath the entire Eureka High campus. In upland areas of the campus, along J Street, the Hookton Formation sediments are buried beneath a veneer of younger marine terrace sediments. In the gulch to the east within which Albee Stadium is located, erosion has penetrated beneath the terrace sediments and upper Hookton Formation, exposing Hookton Formation sediments from lower in the stratigraphic section. The Hookton Formation was initially defined by Ogle (1953), who considered all sediments above the Wildcat Group as Hookton Formation. The unit is characterized by its extremely variable lithology, consisting of gravels, sand, and clay. Most of the Hookton Formation is non-marine, although some sediment in the western part of its extent is shallow marine. Hookton Formation sediments were encountered in all machine borings at the site. In our subsurface explorations, we encountered a wide range of Hookton Formation sediments, including clayey silts (ML) and sands (SC), as well as silty sands (SM), clays (CL), and peaty silt (ML-OL). These deposits also

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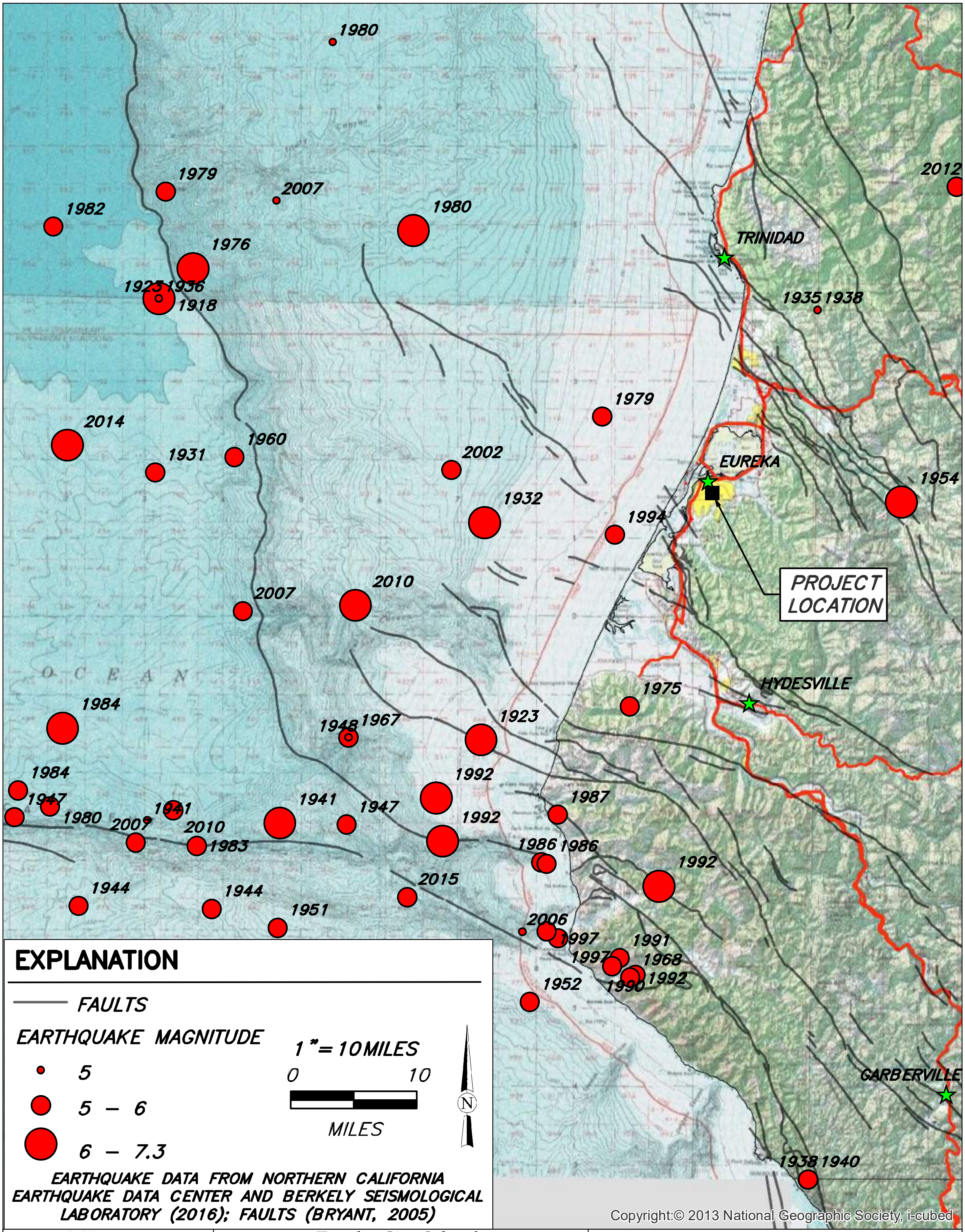

EARTHQUAKE DATA FROM NORTHERN CALIFORNIA EARTHQUAKE DATA CENTER AND BERKELY SEISMOLOGICAL LABORATORY (2016); FAULTS (BRYANT, 2005)

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Regional Seismicity Map
(NCEDC & BSL, 2016)
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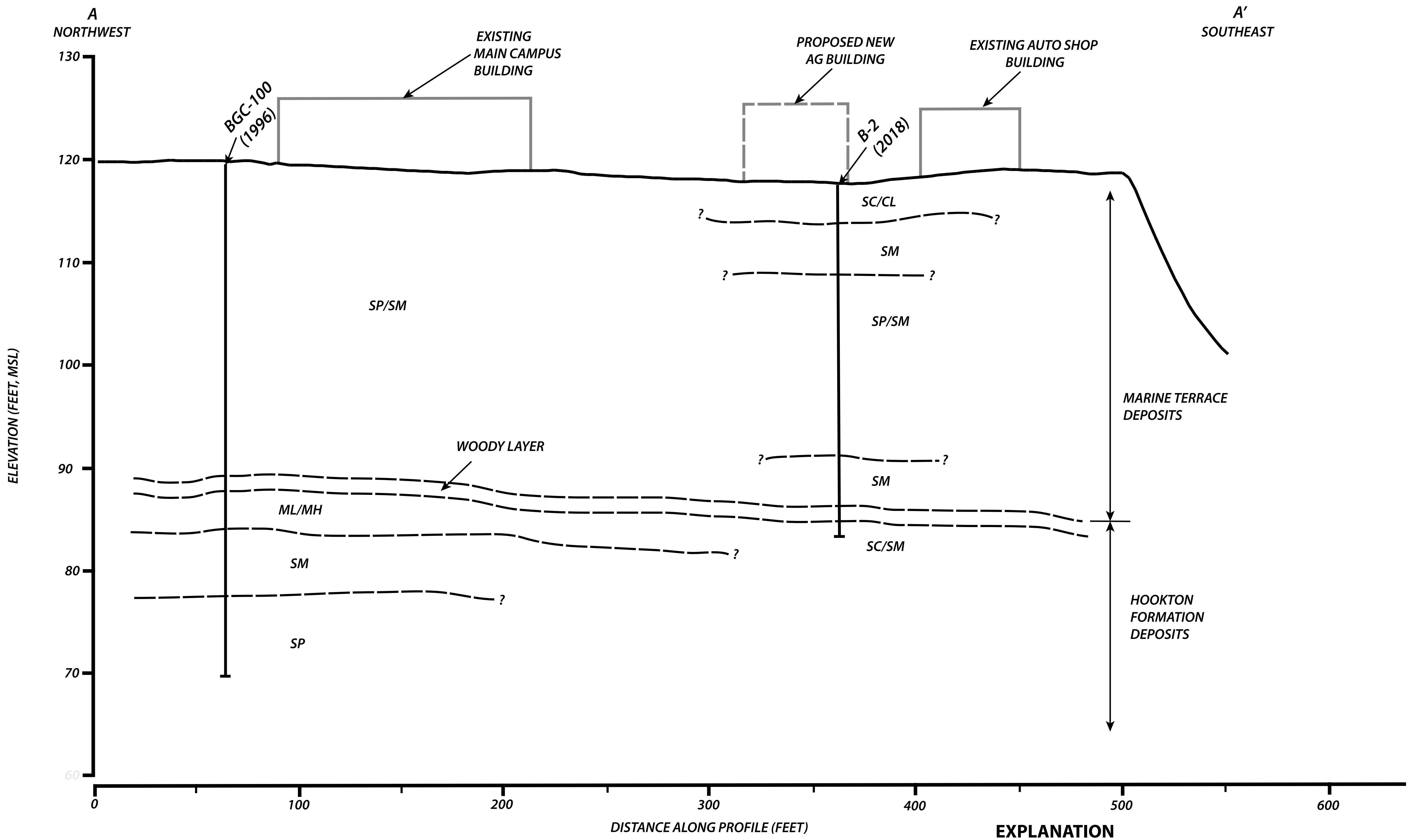
Figure6_RegionalSeismicityMap

Figure 6



EXPLANATION

SM	SILTY SAND
SC	CLAYEY SAND
SP	POORLY GRADED SAND
ML/MH	SILT AND SILT WITH SAND
CL	CLAY

SCALE:
HORIZONTAL: 1"=50'
VERTICAL: 1"=10'
V.E.=5x

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Geologic Cross-Section
A-A'
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Figure7_CrossSectionAA.pdf
Figure 7

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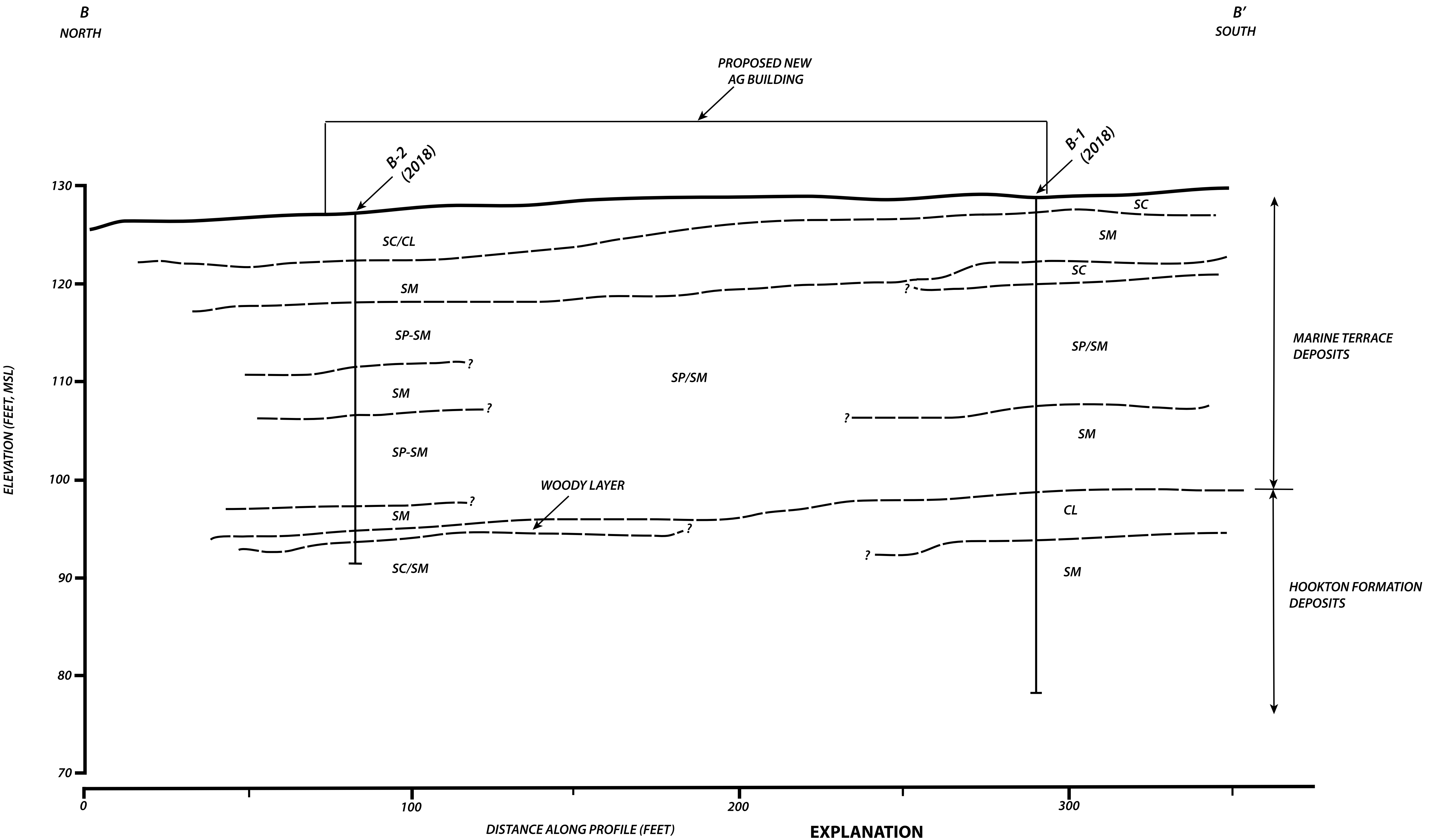

EXPLANATION

SM	*SILTY SAND*
SC	*CLAYEY SAND*
SP	*POORLY GRADED SAND*
SP-SM	*POORLY GRADED SAND WITH SILT*
CL	*CLAY*

SCALE:
HORIZONTAL: 1″=30′
VERTICAL: 1″=10′
V.E.=3x

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Geologic Cross-Section
B-B′
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Figure8_CrossSectionBB.pdf
Figure 8

Path: \\eureka\projects\2016\016137-EHS-Geotech\GIS\FIGURES\Figure7_CrossSectionBB.pdf exhibit a wide range of consistencies. Non-normalized standard penetration test (SPT) blow counts in Hookton sediments ranged from 8 to 44 blows per foot. Hookton Formation sediments overlie the upper part of the Wildcat Group, but that contact was not observed during this study.

2.5.3 Marine Terrace Deposits (Qmt)

Late Pleistocene age marine terrace deposits are present as an approximately 30- to 35-foot thick veneer overlying Hookton Formation sediments at the site. As described above, these sediments were deposited on a marine abrasion platform, in this case eroded onto Hookton Formation sediments. Original geologic maps of the area did not distinguish the marine terraces from the underlying Hookton Formation (Ogle, 1953), but that distinction has become increasingly common.

Because of the marine origin of both the terrace deposits and the underlying Hookton Formation sediments, it is often difficult to distinguish the two during subsurface exploration activities. A horizon of soft, black organic silt or clay with organics that was encountered in many of the deeper machine borings on the terrace surface, at depths between 31 and 36½ feet, appears to mark the contact between the marine terrace deposits and the underlying Hookton Formation (Figure 7). The organic layer, which contains abundant wood debris, appears to represent a paleo-ground surface prior to the deposition of terrace deposits. The woody horizon was also noted at similar depths in previous geotechnical reports (BGC, 2003a). This interval was identified at a depth of 32 feet in boring B-2 installed as part of the current study. The contact between the marine terrace deposits and the Hookton formation within B-1 was primarily mapped based on the transition into soft clay at a depth of 30 feet and an increase in material density below the clay.

At the Eureka High campus, the terrace deposits primarily consist of silty sands (SM) and clayey sands (SC) with medium dense consistency. A black, low density, fine sandy silt is present on marine terrace deposits throughout the region. This material is interpreted as a wind-blown deposit (loess) laid down on top of the terrace during the most recent low sea level stand (coincident with the last glacial maximum), which has developed into an organic rich topsoil. The topsoil has been removed from most of the terrace surface beneath the developed portions of the campus during site grading; however, several areas of intact or reworked topsoil were apparent during some of our previous subsurface investigations and those by previous consultants.

2.5.4 Artificial Fill (af)

Artificial fill of any significant thickness was not encountered in our borings for the Ag Building. Fill thicknesses in previous borings on the terrace surface have generally been less than 2 feet, usually consisting of loose to medium dense silty or clayey sands, or reworked topsoil.

Artificial fills are known to occur elsewhere on the Eureka High campus, particularly along the sidewalls of Cooper Canyon (BGC, 1996b), above Albee Stadium. However, no improvement is proposed in those areas, so we have not assessed the nature of fills that may be present.

3.0 Field Investigations and Laboratory Testing

SHN conducted geotechnical field investigations to provide foundation design and site development criteria for the proposed new Ag Building. Our field investigations were limited to reconnaissance of the project site, as well as supervising the drilling and sampling of two new exploratory borings (B-1 and B-2) within the footprint of the proposed Ag Building (Figure 2). These borings supplement the extensive subsurface investigations associated with previous studies on campus.

SHN's current exploratory borings were advanced to maximum depths of 51.5 feet and 36.5 feet BGS, for B-1 and B-2, respectively. The borings were logged in general accordance with the Unified Soil Classification System (see Figure 2 for boring locations and Appendix 1 for boring logs). The borings were advanced with a truck-mounted drill rig. The drill rig generally used solid-flight continuous augers above the water table, and mud rotary drilling methods below the groundwater surface.

Penetration resistance tests were conducted as the drill rig borings were advanced. We installed the borings for our 2004 study using a 140-pound, 30-inch drop, down-hole hammer, with rope and cathead release; the borings for our 2016 and current study were installed using an auto-hammer. Two samplers were used, a modified California split spoon sampler, with a nominal inside diameter of 2.5 inches (with liners), and a 2-inch outside diameter SPT sampler (without liners). Sampler types are noted on the logs (Appendix 1).

Selected undisturbed samples were collected, and laboratory tests were conducted. Laboratory testing for index properties included in-place moisture content, dry density, percent fines, unconfined compressive strength, and Atterberg Limits (plasticity). Direct shear testing was conducted on samples collected from B-1, and a composite sample of shallow soils from B-1 and B-2 was sent to Cooper Testing Labs for analysis of corrosivity. Laboratory results are presented on the logs in Appendix 1 and laboratory worksheets in Appendix 2.

See the attached boring logs (Appendix 1) for detailed soil descriptions, the penetration resistance test results, and laboratory index test results.

4.0 Evaluation of Potential Geologic Hazards

4.1 Surface Fault Rupture

No known active fault crosses the Eureka High School campus. We found no evidence in our investigation that a previously unrecognized active fault may be present. The Eureka marine terrace, in general, is a low relief topographic surface that would be anticipated to express fault morphology clearly, if active faults were present. The age of the undeformed marine terrace surface in the project vicinity, as described above, is sufficient to preclude Holocene fault activity. The nearest known active fault is the Little Salmon fault, which is mapped approximately 5 miles to the southwest (Figure 5) of the Eureka High School campus. The North Spit fault, a fault inferred to cross the North Spit of Humboldt Bay, based on offshore seismic profiles, may be within about 3 miles of the site. The North Spit fault has never been verified onshore. The nearest fault within the Mad River fault zone, the Fickle Hill fault, is nearly 7 miles to the north of the Eureka High campus. The risk of surface fault rupture at the Eureka High campus is negligible.

4.2 Seismic Ground Shaking

The project is located within a seismically active area. For Risk Categories I, II, and III, the California Building Code (CBC) specifies Seismic Design Category E for seismic hazard conditions where the mapped spectral parameter S_1 exceeds 0.75g (where g is the acceleration of gravity) (CBC, 2013). For the Ag Building site, the mapped S_1 value is 1.223g, necessitating site-specific procedures defined in American Society of Civil Engineers (ASCE) 7-10.

In 2016, SHN subcontracted with Pacific Engineering to perform site-specific seismic evaluation for the Jay Willard Gymnasium located approximately 500 feet south of the current project. Their analysis and report is considered applicable to the proposed Ag Building site and their full report is presented in Appendix 3. The procedures used in development of the seismic design criteria are also in substantial conformance with the 2016 CBC, which went into effect January 1, 2017.

4.3 Liquefaction

Liquefaction is described as the sudden loss of soil shear strength due to a rapid increase of soil pore water pressures caused by cyclic loading from a seismic event. In simple terms, it means that a liquefied soil acts more like a fluid than a solid when shaken during an earthquake. In order for liquefaction to occur, the following are needed:

- granular soils (sand, silty sand, sandy silt, and some gravels),
- a high groundwater table, and
- a low density of the granular soils (usually associated with young geologic age).

Liquefaction occurs during or closely following dynamic loading of moderately consolidated, geologically recent, essentially non-cohesive soil materials beneath the groundwater level. Relatively strong, prolonged earthquakes are the common source of dynamic loading causing liquefaction. During shaking, the soil structure tends to collapse, while pore groundwater pressure builds up in the soils until shear strength is lost, and the soil/groundwater mixture temporarily acts as a liquid until excess pore pressures dissipate. Liquefied soil (a soil/water slurry) can be ejected to the ground surface in sand boils, “sand volcanoes,” or through ground cracks. Block gliding, lateral spreading, or chaotic movement of upper, non-liquefied soils can occur because of underground liquefied layers. Foundation support can be compromised. Soil layers can become softened and weakened, without fully liquefying, and cyclic deformations under earthquake pulses are possible.

Liquefaction has been documented on numerous occasions in the greater Eureka area following historical moderate to large magnitude earthquakes. Specific accounts of historical ground failures are presented in an excellent compilation prepared by Youd and Hoose (1978). Careful interpretation of the historical accounts, however, indicates that liquefaction events in the area are entirely confined to recent alluvial sediments in the Eel River Valley and late Holocene age bay margin sediments surrounding Humboldt Bay. There is no historical account of liquefaction on the Eureka marine terraces, for which the existing and potentially proposed new gymnasium is sited, or on nearby marine terraces.

4.4 Slope Stability

As described above, most of the Eureka High campus is located on a low-gradient terrace surface, which by nature has a negligible potential for slope failure. There is, however, mass wasting potential along the sidewalls of Cooper Canyon, on the slopes to the east of the project site (Figure 2). The closest approach of the canyon sidewalls to the proposed development is about 150 feet. Slope steepness on the canyon sidewalls range from about 45% to as much as 65%. Slope gradient in many places may have been decreased by the placement of fill; however, native slopes may be locally steeper (BGC, 1996b). The slope stability hazard associated with the proposed new Ag Building location is considered low to negligible.

5.0 Geotechnical Site Conditions

The site of the proposed new Ag Building is located on a relatively level, marine terrace surface. Specific descriptions of the soils encountered in the borings are included on the boring logs in Appendix 1. General descriptions are provided below.

In the project area, earth materials are, in general, comprised of stratified, predominantly sandy soils, containing variable silt and clay content, with significant proportions comprised of fine sands containing moderate to relatively low percentages of silt and clay. In the current borings, field sampler penetration blow counts indicate medium dense to dense consistencies, with occasional loose or very dense classifications.

Based on the subsurface conditions encountered at our current and previous exploration locations, laboratory test results, and our interpretation of the stratigraphic conditions within the upper 100 feet of the ground surface, we classify the site as a Site Class D consisting of a “stiff soil profile” in accordance with Chapter 20 of ASCE 7-10.

Groundwater was encountered at an approximate depth of 21 feet in the borings installed for this study. The borings were installed in late March when groundwater can be expected to be near its highest elevations. Groundwater has been encountered in other portions of the campus at depths ranging from 6 to 24 feet BGS in borings. These groundwater observations represent unstabilized water levels observed during mud rotary drilling operations that occurred under a variety of seasonal conditions.

Water levels at other times can be expected to fluctuate in response to seasons, storm events, and other factors. For the purposes of our liquefaction assessment, we assume a seasonal high groundwater table to be encountered at 5 feet BGS, though we consider this a conservative assumption.

Expansive soils are not common in the greater Eureka area. In previous investigations, a composite sample of the site’s siltier, clayey upper soils was tested for Expansion Index (Uniform Building Code [UBC] Test Standard 18-1; ICBO, 2009). The test result was –2.7, indicating very low expansion potential per UBC Table 18-1-A. In addition, the highest Plasticity Index (PI) from 13 Atterberg Limits tests was 17, and the 5 Atterberg Limits tests in the upper 16 feet indicated a maximum PI of 11, in previous investigations (SHN, 2004a). These test values indicate low expansion potential. Soils encountered in the current investigation are similar in constituency and geologic deposition, and a low risk of significantly expansive soils is concluded.

6.0 Geotechnical Conclusions and Discussion

6.1 General

Based on the results of our field and laboratory investigations, it is our opinion that the project can be developed as proposed, if our recommendations are followed, and that noted conditions and risks are acknowledged.

At the proposed new Ag Building location, the primary geotechnical site consideration is suitable foundation bearing materials, and a potential for liquefaction in rare, great earthquakes. Consequently, the recommendations presented below include provisions for foundation embedment, and for using relatively strong, well connected foundation systems for building elements. As is true in all of Humboldt County, the site is subject to strong ground motion from a variety of seismic sources.

6.2 Liquefaction, Co-Seismic Settlement, Lateral Spreading

A description of the liquefaction process is presented in Section 4.3 above; additional descriptions follow.

Most of the Eureka High campus (the higher-in-elevation, relatively level area supporting the main campus building cluster) is located on a portion of a Pleistocene-age marine terrace. The presence of moderately consolidated, saturated, clean sand deposits within the terrace stratigraphy at the site suggests a potential for liquefaction.

Based on the published results of geotechnical testing and post-earthquake studies, the susceptibility of sediments to liquefaction can be directly correlated to the type, origin, and age of the deposits. Geologic materials most susceptible to liquefaction are geologically recent (that is, late Holocene age) sand- and silt-rich deposits, located adjacent to streams, rivers, bays, or ocean shorelines. It should be noted that these “most susceptible” conditions do not exist in the marine terrace deposits at the proposed Ag Building site. Liquefaction occurs only when susceptible materials are saturated. Susceptibility to liquefaction decreases with increasing geologic age (Youd and Perkins, 1978). For example, Table 2 in the Youd and Perkins paper presents estimated liquefaction susceptibility of Holocene marine terraces as low, and Pleistocene marine terraces as very low. The upper portions of the Eureka High School campus are Pleistocene age marine terrace materials.

At the project site, which is underlain by marine terrace materials, liquefaction is considered a low risk under all but rare, major seismic events, due to the geologic age (mid- to late- Pleistocene) of the marine terrace deposits.

The liquefaction potential was evaluated quantitatively using the data collected from borings B-1, and B-2. We conservatively assumed a depth to groundwater level of 5 feet. Liquefaction potential is indicated by the analysis method assuming an M_W 7.5 earthquake with a peak horizontal ground surface acceleration (maximum considered earthquake [MCE_G]; Table 5 in Appendix 3) of 1.25g. We conducted a quantitative liquefaction analysis using the software program LiqIT (version 4.7.7.1) by GeoLogismiki, Inc. The calculation method used is in accordance with the procedures that were developed by National Center for Earthquake Engineering Research (NCEER). The potential for liquefaction is assessed by a calculation of the estimated cyclic stress ratio (CSR) induced by the upper-bound earthquake, compared with the capacity of the soil to resist liquefaction, expressed in terms of the cyclic resistance ratio (CRR). The risk of liquefaction is considered significant where the ratio of CRR to CSR, or factor of safety, approaches a value of about 1.3 or less.

The factor of safety for liquefaction was calculated at less than 1.0 for several stratigraphic intervals in the soil profiles (Appendix 4). As a result, the computer model provides an estimate of the magnitude of potential co-seismic settlement in each of the two boring locations. The total estimated co-seismic settlement is calculated to range from 6.88 inches in B-1 to 2.51 inches in B-2.

It should be noted that this empirical analysis method is derived from a database collected "mostly from sites on level to gently sloping terrain underlain by Holocene alluvial or fluvial sediment at shallow depths (less than 15 m)" (NCEER, 1997). Geologic age is not taken into account. The marine terrace deposits supporting most of the Eureka High campus are geologically older than the sites comprising the database on which the analysis method is based, and the method is not strictly applicable. We applied the method to the marine terrace deposits at the campus, because it is a currently accepted state of the art method of analyzing liquefaction potential, but due to the geologic age of the site's marine terrace and Hookton deposits, it can be assumed that they are less likely to liquefy than the analysis method would indicate.

For example, SHN is not aware of any liquefaction occurring in the marine terrace deposits on which the campus is located during the April 1992, Petrolia earthquakes (maximum moment magnitude 7.2), or the 1906 San Francisco earthquake, both of which caused reported liquefaction effects in the more susceptible Holocene age Eel River delta area. Additionally, no liquefaction effects "except to a very limited extent" were inferred for a similar geologic terrace on which the City of Fort Bragg is located in the 1906 San Andreas fault earthquake (Lawson and others, 1908; Youd and Hoose, 1978). The Fort Bragg terrace deposits are approximately equal in age and composition to the Eureka terrace deposits, and were strongly shaken by this great earthquake.

Based on our geologic review and subsurface investigation, we conclude that **the risk to the proposed developments associated with seismically induced liquefaction is low**. Based on the low potential for liquefaction, we also conclude that the risk of coseismic compaction and lateral spreading during rare, great earthquakes in the underlying deposits at this site is correspondingly low. A low potential for liquefaction should not be confused with *no* potential. Though we consider the settlement estimates derived from our quantitative analysis (2.5 inches at B-1 and 6.88 inches at B-2) to be exaggerated, it would be prudent to incorporate design and construction considerations that allow for liquefaction-induced and/or consolidation settlement on the order of 2 inches during a relatively rare, very strong, upper bound seismic event. Some damage may result from these settlements.

Building code criteria include provisions for some structural damage in major seismic events, but not to the point of building collapse. For example, recent building codes have been based on the following criteria: structures should,

> *…be able to 1) resist a minor level of earthquake motion without damage; 2) resist a moderate level of earthquake ground motions without structural damage, but possibly experience some nonstructural damage; 3) resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage* (Kramer, 1996).

As another example, the 2016 California Building Standards Administrative Code, Chapter 4, Section 4-201, states,

> *Essential services buildings constructed pursuant to these rules and regulations shall be designed and constructed to... resist, insofar as practical, the forces generated by winds and major earthquakes of the intensity and severity of the strongest anticipated at the building site without catastrophic collapse, but may experience some repairable*

> *architectural or structural damage. An essential services building as designed and constructed shall be capable of providing essential service to the public after a disaster.*

In liquefaction events, a lesser degree of differential foundation settlement, and less damage to buildings, has been observed to be associated with continuous foundation systems or mat foundations, where individual foundations are structurally tied or restrained from settling markedly on their own (Liu and Dobry, 1997). Isolated, structurally non-integrated, column footings are more susceptible to differential settlements. Consequently, as a precaution, recommendations below include provisions for using relatively strong, well-connected foundation systems for building elements to reduce risk of abrupt differential settlement.

The foundation and slab on grade recommendations presented below assume the acceptance of some degree of risk of adverse effects resulting from relatively rare, very strong, upper bound seismic events, as discussed above. No very strong earthquake (for example $M_W \geq 7.5$) has occurred in the last 150 years. How these terrace deposits will behave in a great quake has not been observed since the advent of European settlement of this portion of the continent. The recurrence interval for very strong earthquake events originating on the CSZ is 300 to 500 years. As discussed in Section 2.3 above, under "Cascadia Subduction Zone," evidence suggests the last major subduction zone quake occurred on January 27, 1700.

6.3 Settlement Under Static Conditions

In our opinion, under normal static conditions, the risk of significant post-construction foundation settlement will be mitigated to a low level if the recommended site preparations are completed, and if the structures are supported on the recommended shallow foundation system. Recommendations for the foundation system include provisions for structural integration of foundation and floor slab systems, and for relatively low bearing values. Due to the variability of soils deposits and the inherent limitations of current engineering and construction practices, some post-construction vertical settlement may occur. We estimate that with the project constructed in accordance with the following recommendations, total post-construction settlement of the new building is not likely to exceed ¾ inch, and post-construction differential settlement is not likely to exceed ½ inch, with a differential settlement gradient estimated not to exceed ¼ inch in 10 feet.

6.4 Expansive Soils

No high plasticity, potentially expansive soils were observed or are anticipated. Test results discussed under Section 5.0 above did not indicate significant potential for expansive soils behavior. No high plasticity clayey soils stratum was encountered, or is generally anticipated in the geologic formation comprising the site; risk of adverse consequences to the structures from expansive soils is considered low. Recommendations are provided for geotechnical engineering review of the foundation excavations prior to pouring the foundations, at which time the anticipated absence of high-plasticity, potentially expansive soils can be confirmed.

6.5 Soil Corrosivity

In order to assess the potential for soil chemistry at the site to result in excessive corrosion of concrete and steel structural elements, we submitted a composite sample from B-1 and B-2 at the depth intervals of 2 to

4 feet BGS to Cooper Testing Laboratory. The test indicates a resistivity of 10,180 ohm-centimeters, 10 milligrams per kilogram (mg/kg) for chloride, 44 mg/kg sulfate, a pH of 4.8, and an oxidation reduction potential of 518 millivolts (Appendix 2).

The resistivity measurements indicate the soils to be “mildly corrosive.” The pH value of less than 6.0 also suggests a corrosive environment. Corrosion prevention measures should be considered in the design.

7.0 Recommendations

Below we provide general recommendations for the design/construction of the new gymnasium. We recommend that SHN be consulted during the design and planning phases where these recommendations are being applied to ensure that they are appropriate for the specific application.

7.1 Site Preparation and Grading Recommendations

In the following recommendations, "compact" and "compacted" refer to obtaining a minimum of 90% of the maximum dry density as referenced to the ASTM-International (ASTM) D1557 test method. We recommend the following:

- As appropriate, notify Underground Service Alert (1-800-642-2444) prior to commencing site work, and use this location service and other methods to avoid injury or risk to life from underground and overhead utilities, and to avoid damaging them.
- Strip all existing improvements, cultural debris, vegetation, root-systems, dark-colored organic-rich topsoil, and any especially soft or loose soils from areas to receive structural fill or improvements, and a minimum of 5 feet outside perimeter footings and 3 feet beyond exterior slabs. Any existing fill soils that may be encountered should be excavated and removed within the building site.
- With the exception of vertical sides or steps, subgrade surfaces to receive structural fill should be cut-graded to slope no steeper than 10 percent.
- Conduct a geotechnical engineering review of exposed subgrade surfaces. The Geotechnical Engineer will recommend that any remaining unsuitable soils (such as, overly weak, compressible, or disturbed soils) be additionally stripped. This evaluation may include in-place soil density testing, as well as proofrolling as described in the following paragraph.
- Scarify and compact the upper 6 inches of exposed subgrade soils that are to receive structural fills. Alternatively, the subgrade surface may be proofrolled using a 10-wheel, 10-cubic yard dump truck loaded with gravel, or equivalent. The proofrolling should be accomplished under the observation of the Geotechnical Engineer, or qualified representative, with the soil damp or moist (not wet or dry), and a firm, non-yielding surface should be evident during the proofrolling. If a yielding surface is observed (pumping, weaving under wheel loads), additionally excavate the yielding area, and replace the overexcavated material with Caltrans specification Class 2 baserock, in a manner that will result in a stable subgrade surface under the proofrolling, following the overexcavation and replacement.
- Structural fill material should consist of relatively non-plastic (Liquid Limit less than 35, PI less than 12) material containing no organic material or debris, and no individual particles over 4 inches across. We suggest the use of granular soils (sand, gravel) for fill, because these soils are relatively easy to moisture condition and compact.

- Structural fill should be placed to design grades and compacted to a minimum of 90% of the maximum dry density as determined by the ASTM D1557 test method. Fill material should be placed in loose 8-inch lifts, moisture conditioned, and compacted. If fills to support structures are to be more than 2 feet in depth, the grading plan should be reviewed by the Geotechnical Engineer in advance for conditions that could result in excess settlement. Structural fill should extend horizontally beyond the exterior footing perimeters a minimum of 5 feet. Ignore surficial landscaping fill in determining foundation embedment depth.
- Cut and fill slopes up to 4 feet in height should be placed no steeper than 1½H:1V (horizontal to vertical) and 2H:1V, respectively. Higher or steeper slopes should be reviewed by this office for stability.

7.2 Foundation Recommendations

The following foundation recommendations are for general design of typical moderately loaded portions of the foundation system of the new Ag Building.

Seismic design criteria for the structures are presented in Appendix 3.

Following site preparation as recommended, foundations may be constructed. Foundations should be sized, embedded, and reinforced to at least the minimums presented in the current edition of the CBC. Such foundations may be designed so they do not exceed an allowable bearing capacity of 2,500 pounds per square foot (psf) for dead plus long-term live loads. These values may be increased by one-third to account for the short-term effects of wind and/or seismic loading.

Foundation embedment depth should be determined starting at the surface of competent, undisturbed, native soils, or the surface of structural fill placed as recommended above. The provided bearing values are applicable to both competent, undisturbed, native soils, and structural fill placed as recommended.
A friction coefficient of 0.35 may be used for the footing/soil contact. The ultimate friction coefficient may be as low as 0.15 if waterproofing is used. Frictional resistance may be calculated in conjunction with an allowable lateral passive pressure represented by an equivalent fluid weighing 300 pounds per cubic foot (pcf) for short-term loadings (such as, lateral foundation resistance in response to wind or earthquake loadings). Lateral passive pressure can be calculated where footings bear laterally against competent undisturbed native subsoils, or structural fill. The passive resistance within 12 inches of the ground surface should be neglected unless these materials are protected and confined by a slab-on-grade or pavement.

The ground surface around the structure perimeter should be sloped away, or other design measures, implemented to provide positive surface water drainage away from perimeter foundation areas.

In general, we recommend interconnecting foundation elements, and making them resistant to cracking and deformation under bending stresses. For example, continuous foundation elements should be designed to act as grade beams; isolated shallow foundations should be interconnected with grade beams, and/or by a strong floor slab system, so that individual footings are not free to settle alone. Well-reinforced floor slabs should be designed to be resistant to bending, to reduce differential settlement risk. Structural integration of the foundation system, and designing it to be resistant to deformation under bending stresses, should reduce the risk of distress from differential settlement that could conceivably occur in a rare, great earthquake.

7.3 Slab-on-Grade Recommendations

Following site preparation and grading as recommended, slabs-on-grade may be constructed. Where no new structural fill is to be placed beneath slab areas, the exposed soil subgrade surfaces should first be prepared to receive the slab and subslab materials by compaction or proofrolling as recommended above for subgrade surfaces to receive fill. Soil subgrade should not be allowed to dry out during the construction process. Soil subgrade should be maintained in a damp or moist condition until the slab is placed.

Where slab surface moisture would be a significant concern (such as, for interior floors), we recommend that the slabs be underlain by a vapor retarder consisting of a highly durable membrane not less than 15 mils thick (such as, Stego Wrap Vapor Barrier by Stego Industries, LLC or equivalent), underlain by a capillary break consisting of 4 inches of ½- to ¾-inch crushed rock. Please note that these recommendations do not comprise a specification for “waterproofing.” For greater protection against concrete dampness, we recommend that a waterproofing consultant be retained.

Based on our understanding, two opposing schools of thought currently prevail concerning protection of the vapor retarder during construction. Some believe that 2 inches of sand should be placed above the vapor retarder to protect it from damage during construction and also to provide a small reservoir of moisture (when slightly wetted just prior to concrete placement) to benefit the concrete curing process. Still others believe that protection of the vapor retarder and/or curing of concrete are not as critical design considerations when compared to the possibility of entrapment of moisture in the sand above the vapor retarder and below the slab. The presence of moisture in the sand could lead to post-construction absorption of the trapped moisture through the slab and result in mold or mildew forming at the upper surface of the slab.

We understand that recent trends are to use a highly durable vapor retarder membrane (at least 15 mils thick) without the protective sand covering for interior slabs surfaced with floor coverings, including but not limited to, carpet, wood, glued tiles, and linoleum. However, it is also noted that several special considerations are required to reduce the potential for concrete edge curling if sand will not be used, including slightly higher placement of reinforcement steel and a water-cement ratio not exceeding 0.5 (Holland and Walker, 1998). We recommend that you consult with other members of your design team (such as, your structural engineer, architect, and waterproofing consultant) for further guidance on this matter.

We recommend floor slabs be designed to be well reinforced, to reduce risk of cracking from bending or differential settlement. Care should be taken to ensure that floor slab reinforcing bars remain in correct position during concrete placement.

If floor slabs are to be constructed below adjacent, average, finished exterior ground elevations, the capillary break material beneath the slab should be thickened to 8 inches in minimum thickness, and this layer should be compacted with a walk-behind vibratory plate compactor. The 8-inch minimum subdrainage/capillary break layer should be drained by 3-inch diameter perforated drainpipe. The drainpipe may be placed level in the bottom of the layer of drainage material or in trenches below the drainage layer, with trench backfill comprised of permeable material extending up to and hydraulically connected with the subdrainage layer above. A layer of woven filter fabric should separate the underlying soil subgrade from the capillary/permeable material. The filter fabric should be 6-ounce per square yard minimum weight, with an apparent opening size less or equal to 0.25 millimeters. The perforated drainpipes should be placed at a spacing not exceeding 6 feet on centers if the drain piping is placed in the drainage layer, or 10 feet on centers if placed in shallow trenches just below the drainage layer. The drainage piping should be gravity drained to daylight through collector piping. Outside the building perimeter, non-perforated (tightline) drainpipe should be used to drain any collected water to daylight, which should slope at least ¼ inch per foot.

Where below-grade floor slabs are bordered by retaining walls, the retaining walls should be backdrained and waterproofed as recommended in the following section.

7.4 Retaining Wall Recommendations

Retaining walls should be designed to resist static earth pressures, seismic earth pressures, and surcharge pressures. Retaining wall backfill should be placed and compacted according to the recommendations above in Section 7.1 and drainage should be provided behind walls according to the recommendations that follow. Retaining wall foundations should be designed according to the recommendations above in Section 7.2.

Active earth pressures may be used for design of unrestrained retaining walls where the top of the wall is free to translate or rotate. To develop active earth pressures, the walls should be capable of deflecting by at least 0.004H (where H is the height of the wall). At-rest earth pressures should be used for design of retaining walls where the wall top is restrained such that the deflections required to develop active soil pressures cannot occur or are undesirable. Cantilever walls retaining firm native soil or engineered fill may be designed for active or at-rest lateral earth pressures for various backfill slopes using the equivalent fluid unit weights presented in Table 2.

Table 2. Equivalent Fluid Unit Weight
New Ag Building, Eureka High School, Eureka, California
(pcf)[1]

Backfill Slope	At-Rest Conditions	Active Conditions
Level	62	36
3H:1V[2]	81	46
2H:1V	89	55

1. pcf: pounds per cubic foot
2. H:V: horizontal to vertical

Lateral earth pressures for backfill slopes other than those given above can be estimated by interpolation. The lateral earth pressures should be applied to a plane extending vertically upward from the base of the heel of the retaining wall to the ground surface.

The lateral earth pressures given above apply where the wall backfill is fully drained, is not subject to traffic or other surcharge loads, and is not subject to heavy compaction equipment within a distance of one-third the height of the backfill. Lateral surcharge pressures are discussed later in this section.

If retaining wall backfill will be subject to passenger vehicle or light truck traffic loading within a distance of H/2 from the top of the wall (where H is the wall height), the wall should be designed to resist an additional uniform lateral pressure of 72 psf applied to the back of yielding walls (active conditions), or 124 psf applied to the back of non-yielding walls (at-rest conditions). Surcharge loads imposed by greater loads or unusual loads within a distance of H of the back of the wall should be considered on a case-by-case basis.

In addition to the active or at-rest lateral soil pressures, retaining walls should be designed to resist additional dynamic earth pressures during earthquake loading. The additional dynamic pressure increment may be calculated using an additional equivalent fluid pressure of 10 pcf for back slopes up to 3H:1V. The dynamic pressure increment should be applied to the wall as a triangular distribution so the resultant force acts at a distance of 0.33H above the base of the wall (where H is the height of the wall). Under the combined effects of static and dynamic loading, a safety factor of 1.1 against sliding or overturning is acceptable. The dynamic component of the lateral earth pressure was calculated using the Mononabe-Okabe equation and, therefore, assumes that sufficient deformation of the wall will occur during seismic loading to develop active soil conditions.

A drainage system should be constructed on the backside of all retaining walls. The drainage system for backfilled walls should consist of a 4-inch diameter perforated pipe surrounded by Class 2 Permeable Material complying with Section 68 of the Caltrans Standard Specifications, latest edition. Alternatively, the perforated pipe may be surrounded by clean coarse gravel or drain rock, provided the gravel or rock is completely separated from the surrounding soil by an engineering filter fabric, such as, Mirafi 140N or similar fabric. The section of permeable material should be at least 12 inches wide and should extend up the back of the wall to within about 18 inches of finished grade. The drainage material should be capped with compacted fine-grained soil, soil-cement, or other relatively impermeable material or barrier. The pipe should be polyvinyl chloride (PVC) Schedule 40 or acrylonitrile butadiene styrene (ABS) pipe with a standard dimension ratio (SDR) of 35 or better. Perforations in the drainpipe should be ¼ inch in diameter. The perforated pipe should be placed holes-down near the bottom of the section of permeable material and should discharge by gravity to a suitable outlet. Accessible subdrain cleanouts should be provided and maintained on a regular basis.

8.0 Construction Considerations

The following construction considerations are presented to aid in project planning. These considerations are not intended to be comprehensive; other issues may arise that will require coordination between the owner, the engineer, and the contractor's construction methods and capabilities.

It is important to note that even small quantities of persistent seepage may substantially complicate construction operations if the proposed excavation extends near or below areas of saturated soil.

OSHA (Occupational Safety Health Administration) trench and excavation safety regulations should be acknowledged and followed. Test results indicate soil cohesion varies in the site’s upper soils. In general, OSHA Type B or C soils are indicated, requiring excavation sideslopes of 1H:1V or 1.5H:1V, respectively, for excavations up to 20 feet in depth. Given that soil cohesive strength is anticipated to be variable, evaluations of soils exposed in specific locations should be made by a competent person. Compliance with safety regulations is the responsibility of the contractor. Excavations may require shoring to allow construction workers to enter.

Due to the weak nature of some of the site's sandy soils, some trenches are considered subject to sidewall instability (sloughing, running, or sudden collapse of the trench sidewalls). In general, cohesive to moderately cohesive upper site soils are anticipated, and sidewall instability is not considered likely in trenches of moderate depth.

9.0 Additional Services

9.1 Project Bid Documents

Prior to bidding, prospective contractors for the project often contact us regarding the information contained in our report. These informal contacts could result in incomplete or misinterpreted information being provided to the contractor. Consequently, we recommend a pre-bid meeting to answer such questions prior to bid submittal. Alternatively, such questions should be addressed to the owner or designated representative, who, after consulting with SHN, can appropriately respond to all prospective contractors with clarifications or additional information.

9.2 Plan and Specification Review

During the design phase, it is important that communications between the design team and SHN be maintained to optimize compatibility between the design and soil and groundwater conditions.

We have assumed, in preparing our recommendations, that we will be retained to review those portions of the plans and specifications that pertain to earthwork and foundations. The purpose of this review is to confirm that our earthwork and foundation recommendations have been properly interpreted and implemented during design. If we are not provided this opportunity for review of the plans and specifications, our recommendations could be misinterpreted.

9.3 Construction Phase Monitoring

To assess construction conformance with the intent of our recommendations, it is important that a representative of our firm:

- monitor adequate site stripping, including removal of vegetation, root-filled soils, upper dark-colored organic topsoils, and uncontrolled existing fill soils where recommended;
- determine methods for and monitor adequate subgrade preparation;
- monitor placement of structural fill;
- monitor foundation excavations;
- monitor deep foundation installations; and
- monitor backdrains, underdrains, capillary break layers, and waterproofing.

This construction phase monitoring is important because it provides the owner and SHN the opportunity to verify anticipated site conditions, and recommend appropriate changes in design or construction procedures if site conditions encountered during construction vary from those described in this report. It also allows SHN to recommend appropriate changes in design or construction procedures if construction methods adversely affect the competence of onsite soils to support the structural improvements.

10.0 Limitations

The analyses, conclusions, and recommendations contained in this report are based on site conditions that we observed at the time of our investigation, data from our subsurface explorations and laboratory tests, our current understanding of proposed project elements, and on our experience with similar projects in similar geotechnical environments. We have assumed that the information obtained from our limited subsurface explorations is representative of subsurface conditions throughout the site. In order to confirm this assumption, a representative of our firm must observe and evaluate actual soil conditions encountered during project construction operations.

Subsurface conditions may differ from those disclosed by our limited investigations. If differing conditions are encountered during construction, our firm should be notified immediately so that we can reevaluate the applicability of our conclusions and recommendations. Such an evaluation may result in reconsidered and/or amended recommendations. If the scope of the proposed construction, including the proposed loads, grades, or structural locations, changes from that described in this report, our recommendations should also be reviewed.

Our firm has prepared this report for your exclusive use on this project in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study, including time and budget constraints. No warranty, express or implied, applies to this report. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by our firm during the construction phase in order to evaluate compliance with our recommendations.

If there is a substantial lapse of time between the submission of our report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, we should review our report to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse. This report is applicable only to the project and site studied.

The field and laboratory work was conducted to investigate the site characteristics specifically addressed by this report. Assumptions about other site characteristics, such as, hazardous materials contamination, or environmentally sensitive or culturally significant areas, should not be made from this report.

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Subsurface Exploration Logs

1

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: EHS Ag Building Geotech
LOCATION: Eureka High School
GROUND SURFACE ELEVATION: 119 Feet (Google Earth)
EXCAVATION METHOD: Mud Rotary and Solid Flight Auger
LOGGED BY: P. Sundberg

JOB NUMBER: 016137.100
DATE DRILLED: 3/21/2018
TOTAL DEPTH OF BORING: 51.5 Feet
SAMPLER TYPE: MOD CAL/SPT

BORING NUMBER
B-1

DEPTH (FT)	BULK SAMPLES	SHELBY TUBE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits		REMARKS
											Liquid Limit	Plastic Index	
0.0				SC		ASPHALT (3") AND BASE ROCK (1")							FILL
-1.0						CLAYEY SAND; Dark brown, moist, low plasticity.							
-2.0			5	SM		SILTY SAND; Brown to Yellowish-brown, loose, moist, non-cemented, non-plastic, fine to medium sand.							MARINE TERRACE DEPOSITS
			4										
-3.0			5				18	98					Direct Shear; See Appendix 2
-4.0			8	SM		SILTY SAND; Brown/Light gray/Yellowish-brown (mottled), medium dense, moist, non-cemented, non-plastic, iron staining.							
			8										
-5.0			9				16	105					Direct Shear; See Appendix 2
-6.0			8										
			8	SC		CLAYEY SAND; Light gray (mottled), medium dense, moist, low plasticity.							
-7.0			19	SP-SM		SILTY SAND; Light gray (mottled), medium dense, moist, weak cementation, iron staining, iron-oxide nodules.	21	106					Direct Shear; See Appendix 2
-8.0			8										
			14										
-9.0			21				19	105					
-10.0			15										
			18										
-11.0			20				16	109					
-12.0													
-13.0													
-14.0													
-15.0			9										
			11										
-16.0			10							17			
-17.0													
-18.0													
-19.0													
-20.0													

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

LOG OF BORING

PROJECT: EHS Ag Building Geotech

LOCATION: Eureka High School

GROUND SURFACE ELEVATION: 119 Feet (Google Earth)

EXCAVATION METHOD: Mud Rotary and Solid Flight Auger

LOGGED BY: P. Sundberg

JOB NUMBER: 016137.100

DATE DRILLED: 3/21/2018

TOTAL DEPTH OF BORING: 51.5 Feet

SAMPLER TYPE: MOD CAL/SPT

BORING NUMBER **B-1**

DEPTH (FT)	BULK SAMPLES	SHELBY TUBE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-20.0			12 12 10										No recovery 20-21.5' Drill to 22-use catcher
-21.0 ▼													
-22.0	☒		12 16 17	SM		POORLY GRADED SAND with SILT; Yellowish-brown, dense, wet, non-cemented, fine to medium sand, non-plastic.							SPT w/ catcher
-23.0										12			
-24.0													
-25.0			1 4 21										MCS w/ catcher; heaving sand, took sample then switched to mud rotary
-26.0							24	108		8			
-27.0													
-28.0													
-29.0													
-30.0	☒		P P 2	CL		SANDY LEAN CLAY; Dark brown to light olive brown, soft, moist, low plasticity, non-cemented, fine sand, low to medium dry strength.							SPT w/ catcher
-31.0											27	11	
-32.0													
-33.0													
-34.0													
-35.0			18 21 16	SM		SILTY SAND; Light olive brown, medium dense, moist to wet, non-cemented, bottom of sample is mottled, some iron staining.							MCS w/ catcher
-36.0							23	106		22			
-37.0													
-38.0													
-39.0													
-40.0													

PROJECT: EHS Ag Building Geotech
LOCATION: Eureka High School
GROUND SURFACE ELEVATION: 119 Feet (Google Earth)
EXCAVATION METHOD: Mud Rotary and Solid Flight Auger
LOGGED BY: P. Sundberg

JOB NUMBER: 016137.100
DATE DRILLED: 3/21/2018
TOTAL DEPTH OF BORING: 51.5 Feet
SAMPLER TYPE: MOD CAL/SPT

BORING NUMBER
B-1

DEPTH (FT)	BULK SAMPLES	SHELBY TUBE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits		REMARKS
											Liquid Limit	Plastic Index	
-40.0			14 16 22			becomes dense							SPT w/ catcher
-41.0													
-42.0													
-43.0													
-44.0													
-45.0			35 50/6"			becomes very dense							MCS w/ catcher
-46.0													
-47.0													
-48.0													
-49.0													
-50.0			18 21 23			becomes dense							SPT w/ catcher
-51.0													
-52.0						Boring terminated at a depth of 51.5 feet. Groundwater encountered at a depth of 21 feet. Borehole backfilled with cement grout and capped with concrete.							
-53.0													
-54.0													
-55.0													
-56.0													
-57.0													
-58.0													
-59.0													

Consulting Engineers & Geologists, Inc.

812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: EHS Ag Building Geotech

LOCATION: Eureka High School

GROUND SURFACE ELEVATION: 120 Feet (Google Earth)

EXCAVATION METHOD: Mud Rotary and Solid Flight Auger

LOGGED BY: P. Sundberg

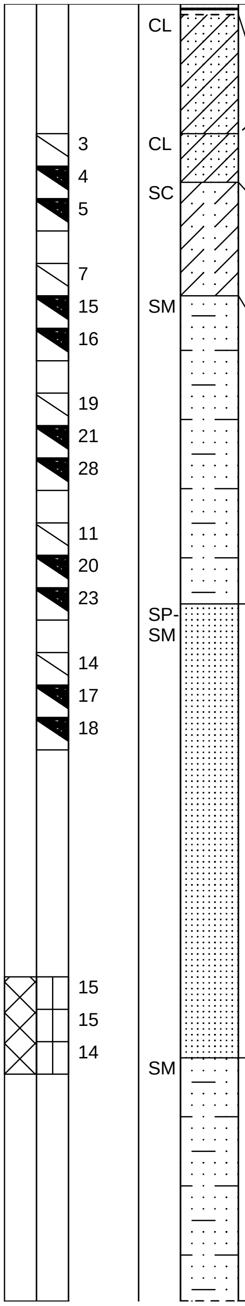
JOB NUMBER: 016137.100

DATE DRILLED: 3/21/2018

TOTAL DEPTH OF BORING: 36.5 Feet

SAMPLER TYPE: MOD CAL/SPT

BORING NUMBER
B-2



DEPTH (FT)	BULK SAMPLES	SHELBY TUBE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	REMARKS
0.0				CL		ASPHALT (2")							
-1.0						SANDY CLAY; Brown, moist, medium plasticity.							
-2.0			3	CL		SANDY CLAY; Brown, medium stiff, moist, low plasticity.							
			4										
-3.0			5	SC		CLAYEY SAND; Brown to yellowish-brown, loose, moist, weak cementation, low plasticity, slightly mottled.	20	101	928				
										45			
-4.0			7										
			15	SM		SILTY SAND; Brown to yellowish-brown, medium dense, moist, weak cementation, non-plastic, fine sand, slightly mottled, iron stained.							
-5.0			16				17	110					
-6.0			19										
			21										
-7.0			28				14	112					
-8.0			11										
			20										
-9.0			23										
				SP-SM		POORLY GRADED SAND with SILT; Brown, medium dense, moist, non-cemented, fine sand.							
-10.0			14										
			17										
-11.0			18										
-12.0													
-13.0													
-14.0													
-15.0			15										SPT w/ catcher
			15										
-16.0			14										
				SM		SILTY SAND; Light gray, medium dense, moist, non-cemented.				16			
-17.0													
-18.0													
-19.0													
-20.0													

LOG OF BORING

PROJECT: EHS Ag Building Geotech
LOCATION: Eureka High School
GROUND SURFACE ELEVATION: 120 Feet (Google Earth)
EXCAVATION METHOD: Mud Rotary and Solid Flight Auger
LOGGED BY: P. Sundberg

JOB NUMBER: 016137.100
DATE DRILLED: 3/21/2018
TOTAL DEPTH OF BORING: 36.5 Feet
SAMPLER TYPE: MOD CAL/SPT

BORING NUMBER
B-2

DEPTH (FT)	BULK SAMPLES	SHELBY TUBE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-20.0 ▼	☒		7 11 14			POORLY GRADED SAND with SILT; Light gray, medium dense, moist, non-cemented				11			switch to mud rotary
-21.0													
-22.0													
-23.0													
-24.0													
-25.0	☒		11 16 17			becomes dense							SPT w/ catcher
-26.0													
-27.0													
-28.0													
-29.0													
-30.0	☒		12 14 16			SILTY SAND; Light gray, dense, moist, non-cemented				25			SPT w/ catcher
-31.0													
-32.0						wood encountered at 32'							
-33.0													
-34.0						CLAYEY SAND; Light bluish-gray, medium stiff to stiff, moist to wet, low plasticity.							
-35.0	☒		5 8 13	SC SM		SILTY SAND; Brownish-gray, medium dense, moist to wet, non-cemented, non-plastic.							SPT w/ catcher
-36.0													
-37.0						Boring terminated at a depth of 36.5 feet. Groundwater encountered at a depth of 20.5 feet. Borehole backfilled with cement grout and capped with cold patch.							
-38.0													
-39.0													

LOG OF BORING

BUSCH GEOTECHNICAL CONSULTANTS

Project: Eureka City Schools
Location: Del Norte Street
Drilling Co: Clearheart
Equipment: Deep Rock 10K
Sampler: 1" I.D. standard split spoon

Job # 96-072
By MV
Date 9/20/96
Completion Depth 51.5 ft
Water Level 23 ft
Foreman Teller

Log # 100
Page 1 of 5

Materials Description	Depth feet	SAMPLE			Remarks
		type	recov.	# blows	
Topsoil, silt, sandy (fine), soft, damp, black, (ML).	- - - 1				
Sand, (fine), silty, medium dense, reddish brown, damp, (SM).	- - - 2				
Sand (fine), slightly silty, medium dense, moist, dark yellowish brown with few faint mottles, (SM-SP).	- - - 3 - - - 4 - - - 5 - - - 6				Description of upper 10' from hand auger holes BGC-1, -2, and -3 in BGC, 1996b.
Sand (fine), slightly clayey, slightly silty, loose, moist, dark yellowish brown with common distinct mottles, (SM).	- - - 7 - - - 8				
Sand (fine to medium), medium dense to loose, slightly silty, moist, gray, (SP-SM). Few faint laminations at 10.0'.	- - - 9 - - -				

Notes: 1) SPT sampler = 1" I.D., Modified CA = 2.5" I.D.
2) The number of blows is a measure of the resistance to penetration for every 6 inch increment.
3) Blowcounts have been adjusted for effective overburden pressure and sampler diameter. They have also been normalized to a 2000 psf standard overburden pressure.

Materials Description	Depth feet	SAMPLE type	recov.	# blows	Remarks
	10				
few manganese concretions in top 3"	-			5	
	-				
Sand (fine), slightly silty, medium dense, damp, dark yellowish brown, (SM-SP).	-	SPT	100%	9	N1 = 22
	11				
	-			8	
becomes slightly clayey in bottom 4"	-				
	-				
	12				
	-				
	-				
	-				
	13				
	-				
	-				
	-				
	14				
	-				
	-				
	-				
	15				
	-				
	-				
	-				
	16				
	-				
	-				
	-				
	17				
	-				
	-				
	-				
	18				
	-				
	-				
	-				
	19				
	-				
	-				
	-				
	20				
	-			3	
	-				
Sand (fine), slightly silty, medium dense, wet, yellowish brown to light gray (SM-SP).	-	SPT	100%	4	N1 = 10
	21				
3" thick dark yellowish brown slightly clayey sand at 21 ft	-			7	
	-				

Materials Description	Depth feet	SAMPLE			Remarks
		type	recov.	# blows	
	-				
	22				
	-				
	-				
	-				
▼	23				Free water at 23'.
at drilling	-				
	-				
	-				
	24				
	-				
	-				
	-				
	25				
	-			7	
	-				
	-	SPT	100%	18	N1 = 34
Sand (fine), slightly silty, medium dense, wet, yellowish brown, (SP-SM).	26				
	-			25	
	-				
	-			6	
	27				
	-	SPT	100%	10	N1 = 13
	-				
	-			7	
	28				
	-			4	
	-				
	-	SPT	100%	8	N1 = 11
	29				
	-			7	
	-				
	-			3	
	30				
	-	SPT	100%	3	N1 = 4
	-				last blow
	-			3	dropped
Wood, bark intact.	31				abruptly
	-			1	
Silt, clayey, soft, wet, gray to dark brown, (ML-MH).	-				
few thin beds of slightly sandy, clayey silt (light brown)	-	SPT	100%	1	N1 = 1
	32				
	-			1	
	-				
	-				
	33				
	-				

BUSCH GEOTECHNICAL CONSULTANTS

Log # 100
Page 4 of 5

Materials Description	Depth feet	**SAMPLE**			**Remarks**
		type	recov.	# blows	
	-				
	-				
	-				
	34				
	-				
	-				
	-				
	35				
	-				
	-				
	-				
	36				
	-				
	-				
	-				
	37				
	-				
	-				
	-				
	38				
	-				
	-				
	-				
	39				
	-				
	-				
	-				
	40				
	-			4	
Sand (fine), silty, medium dense, wet, gray, (SM).	-				
	-	SPT	100%	8	N1 = 11
	41				
	-			9	
	-				
	-				
	42				
	-				
	-				
	-				
	43				
	-				
	-				
	-				
	44				
	-				
	-				
	-				

Log # 100
Page 5 of 5

Materials Description	Depth feet	**SAMPLE** type	recov.	# blows	**Remarks**
	-				
	45				
	-				
	-				
	-				
Sand (fine), medium dense to dense, wet, yellowish brown, (SP).	46				
	-				
	-				
	-				
	47				
	-				
	-				
	-				
	48				
	-				
	-				
	-				
	49				
	-				
	-				
	-				
	50				
Sand (fine to medium), medium dense, wet,	-			3	
gray, (SP).	-				
2" layer of clayey silt at 50.5 ft	-	SPT	100%	14	N1 = 23
	51				
Bottom of hole in same.	-			28	
	-				
	-				
	52				
	-				
	-				
	-				
	53				
	-				
	-				
	-				
	54				
	-				
	-				
	-				
	55				
	-				
	-				
	-				
	56				
	-				

Laboratory Results

2

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DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	EHS Agriculture Bldg	**Project Number:**	016137.100
Performed By:	JMA	**Date:**	4/18/2018
Checked By:	NAN	**Date:**	4/22/2018
Project Manager:	GDS		

Lab Sample Number	18-173	18-174	18-177	18-179	18-183
Boring Label	B1	B1	B1	B1	B2
Sample Depth (ft)	9-9.5	11-11.5	26-26.5	36-36.5	7-7.5
Diameter of Cylinder, in	2.38	2.38	2.38	2.38	2.38
Total Length of Cylinder, in.	6.00	6.00	6.00	6.00	6.00
Length of Empty Cylinder A, In.	0.00	0.35	1.55	1.45	0.00
Length of Empty Cylinder B, in.	0.00	0.78	1.40	1.50	0.00
Length of Cylinder Filled, in	6.00	4.87	3.05	3.05	6.00
Volume of Sample, in^3	26.69	21.67	13.57	13.57	26.69
Volume of Sample, cc.	437.42	355.04	222.35	222.35	437.42

Pan #	s19	s20	a6	ss8	s8
Weight of Wet Soil and Pan	1103.0	944.0	563.2	655.5	1047.1
Weight of Dry Soil and Pan	961.1	842.9	470.9	569.2	941.1
Weight of Water	141.9	101.1	92.3	86.3	106.0
Weight of Pan	224.7	222.0	87.4	193.6	160.2
Weight of Dry Soil	736.4	620.9	383.5	375.6	780.9
Percent Moisture	19.3	16.3	24.1	23.0	13.6
Dry Density, g/cc	1.68	1.75	1.72	1.69	1.79
Dry Density, lb/ft^3	105.1	109.2	107.7	105.5	111.5

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DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	**EHS Agriculture Bldg**	**Project Number:**	**016137.100**
Performed By:	**JMA**	**Date:**	**4/18/2018**
Checked By:	**NAN**	**Date:**	**4/22/2018**
Project Manager:	**GDS**		

Lab Sample Number	18-182				
Boring Label	B2				
Sample Depth (ft)	5-5.5				
Diameter of Cylinder, in	2.42				
Total Length of Cylinder, in.	6.00				
Length of Empty Cylinder A, in.	0.00				
Length of Empty Cylinder B, in.	0.69				
Length of Cylinder Filled, in	5.31				
Volume of Sample, in^3	24.42				
Volume of Sample, cc.	400.23				

Pan #	a4				
Weight of Wet Soil and Pan	909.0				
Weight of Dry Soil and Pan	790.9				
Weight of Water	118.1				
Weight of Pan	87.8				
Weight of Dry Soil	703.1				
Percent Moisture	16.8				
Dry Density, g/cc	1.76				
Dry Density, lb/ft^3	109.7				

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PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name: EHS Agriculture Bldg. **Project Number:** 016137.100
Performed By: ESP **Date:** 4/16/2018
Checked By: NAN **Date:** 4/22/2018
Project Manager: GDS

Lab Sample Number	18-175	18-176	18-177	18-179	18-181
Boring Label	B1	B1	B1	B1	B2
Sample Depth	15-16.5	22-23.5	26	36	3'
Pan Number	ss8	ss5	ss2	a7	A1
Dry Weight of Soil & Pan	336.2	366.0	333.3	222.9	285.9
Pan Weight	193.1	195.5	194.4	86.7	86.0
Weight of Dry Soil	143.1	170.5	138.9	136.2	199.9
Soil Weight Retained on #200&Pan	312.3	344.9	322.8	192.5	195.3
Soil Weight Passing #200	23.9	21.1	10.5	30.4	90.6
Percent Passing #200	**16.7**	**12.4**	**7.6**	**22.3**	**45.3**

Lab Sample Number	18-184	18-185	18-186		
Boring Label	B2	B2	B2		
Sample Depth	15-16.5	20-21.5	30-31.5		
Pan Number	a9	ss9	ss6		
Dry Weight of Soil & Pan	302.5	389.5	397.9		
Pan Weight	88.3	196.5	196.1		
Weight of Dry Soil	214.2	193.0	201.8		
Soil Weight Retained on #200&Pan	267.7	368.0	347.7		
Soil Weight Passing #200	34.8	21.5	50.2		
Percent Passing #200	**16.2**	**11.1**	**24.9**		

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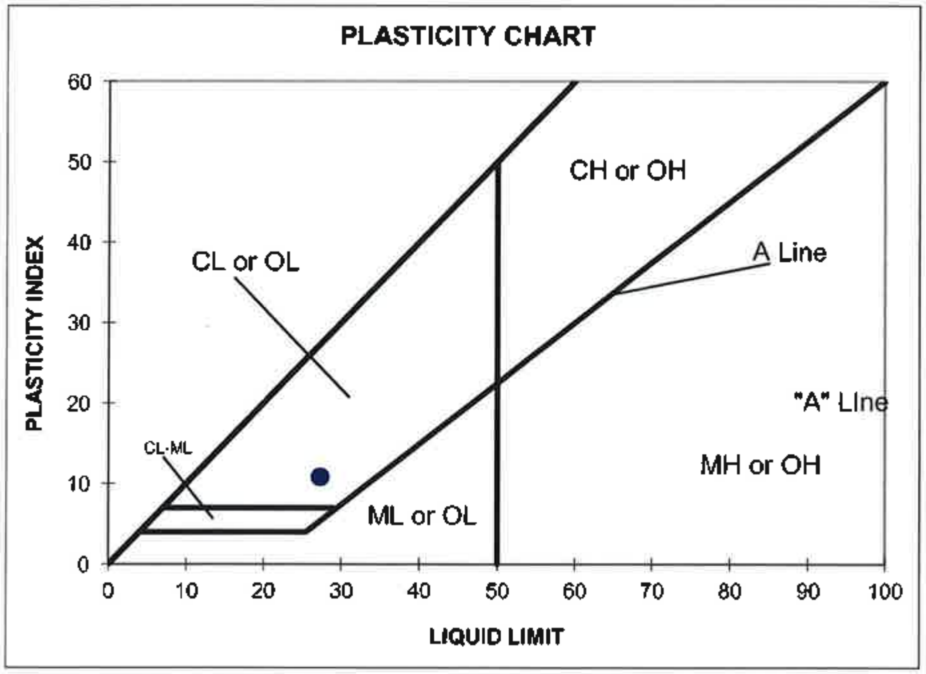
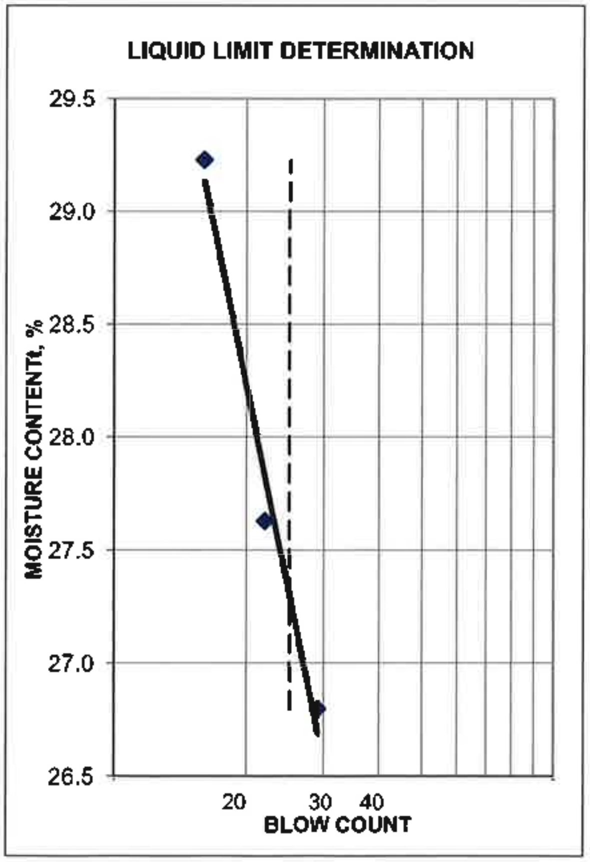
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME: **EHS Agriculture Bldg.** JOB#: **016137.100** LAB SAMPLE #: **18-178**
SAMPLE ID: **B-1 at 30-31 1/2 B** PERFORMED BY: **JMA** DATE: **4/20/2018**
PROJECT MANAGER: **JPB** CHECKED BY: **NAN** DATE: **4/22/2018**

LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	15	16	4	5	6
B	PAN WT. (g)	20.660	21.040	29.390	28.810	29.690
C	WT. WET SOIL & PAN (g)	27.850	27.850	37.860	36.340	37.560
D	WT. DRY SOIL & PAN (g)	26.820	26.900	36.070	34.710	35.780
E	WT. WATER (C-D)	1.030	0.950	1.790	1.630	1.780
F	WT. DRY SOIL (D-B)	6.160	5.860	6.680	5.900	6.090
G	BLOW COUNT	--	--	29	22	16
H	MOISTURE CONTENT (E/F*100	16.7	16.2	26.8	27.6	29.2

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
27.3	**11**	16.5





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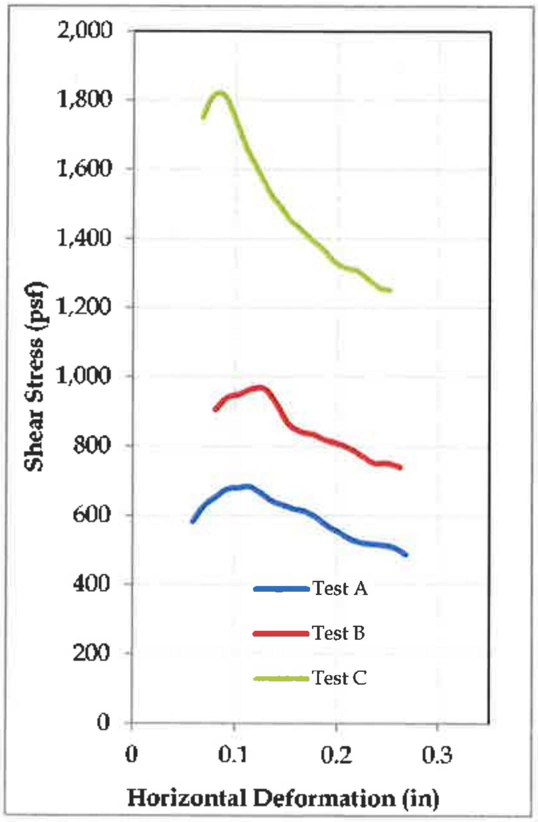
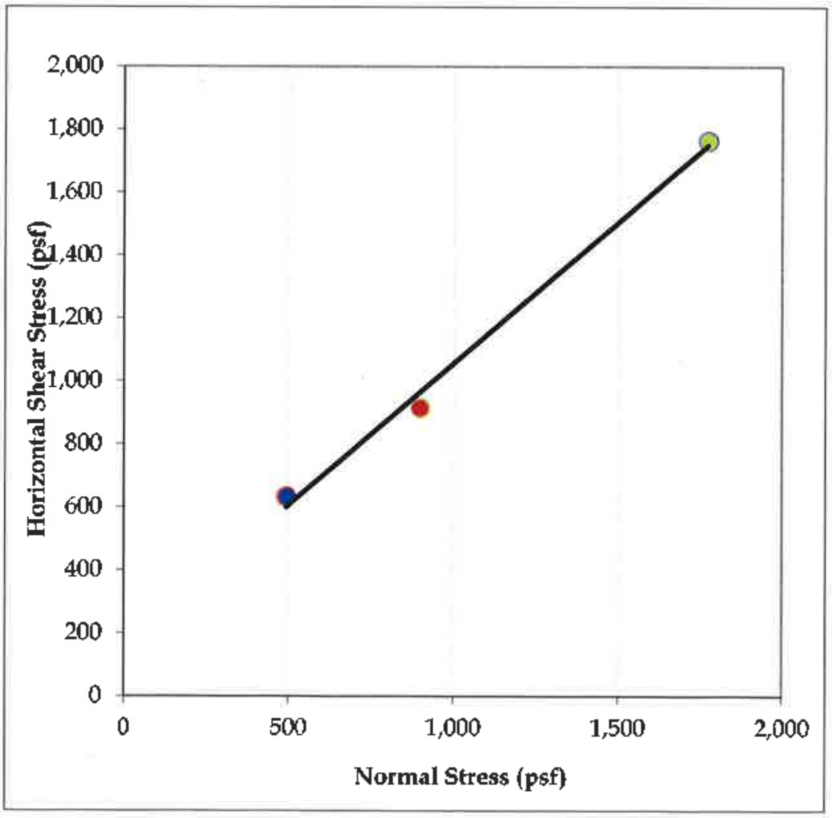
DIRECT SHEAR TEST RESULTS
ASTM D6528

JOB NAME: EHS Agriculture Bldg
JOB #: 016137.100
LAB #: 18-170,171,172
SAMPLE ID: B1 @ 3',5', 7'
PERFORMED BY: JMA
DATE: 4/20/18
PROJECT MNGR: GDS
CHECKED BY: DJG
DATE: 4/23/18

SAMPLE DESCRIPTION: Silty SAND
REMARKS: Consolidated Undrained

Test Number	Initial Water Content (%)	Initial Dry Density (pcf)	Vertical Load (psf)	Horizontal Shear Stress (psf)	C (psf)	f (deg)	Tan f (deg)
18-170	18.0	97.6	493	634	159	41.9	0.90
18-171	15.5	105.2	898	916			
18-172	20.7	105.9	1,774	1,765			





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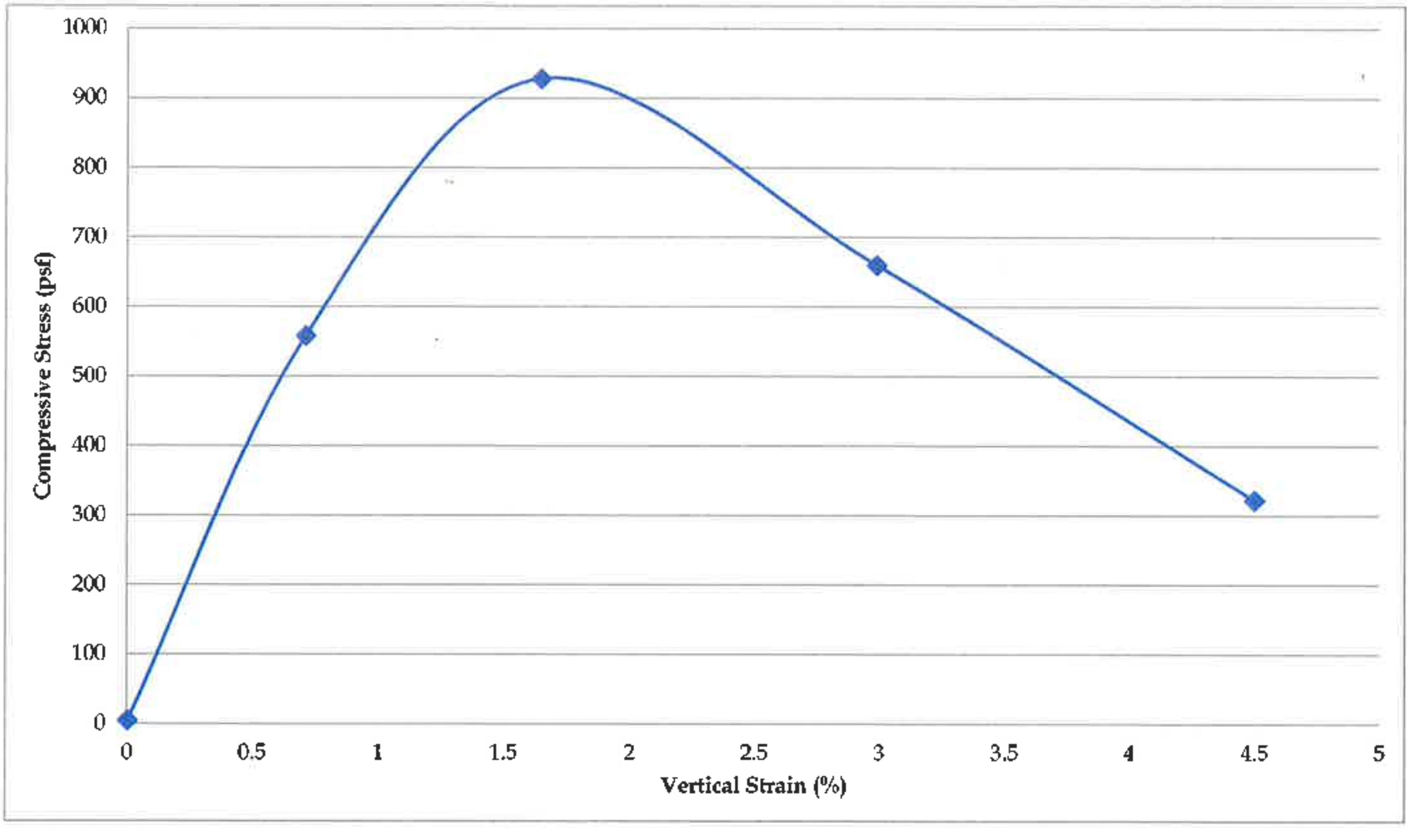
UNCONFINED COMPRESSION TEST REPORT
ASTM D2166

Job Name:	EHS Agriculture Bldg		
Job Number:	016137.100	**Tested By:** JMA	**Date:** 4/16/2018
Project Manager:	GDS	**Checked By:** NAN	**Date:** 4/22/2018



Symbol		◊				
Lab Sample #		18-180				
Initial	**Diameter (in)**	2.42				
	Height (in)	5.2				
	Water Content (%)	19.8				
	Dry Density (pcf)	100.8				
	Saturation (%)	79.5				
	Void Ratio	0.67				
Unc. Comp. Strength (psf)		928				
Undrnd. Shear Strength (psf)		646				
Time to Failure (min)		1.75				
Strain Rate (%/min)		1				
Est. Specific Gravity		2.7				
Boring No. & Depth		B2 @2.5-3				
Sample Type		2.5" Liner				
Description		Silty SAND				
Remarks						

COOPER
TESTING LABORATORY

Corrosivity Tests Summary

CTL # 054-175 **Date:** 4/26/2018 **Tested By:** PJ **Checked:** PJ

Client: SHN **Project:** Eureka High AG Bldg **Proj. No:** 016137.100

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
1	-	-	-	-	10,180	10	44	0.0044	4.8	518	21	-	0.0	Yellowish Brown Clayey SAND

Site-Specific Ground Motion Analyses

3

Development of Design Ground Motions For The Eureka High School Jay Willard Gymnasium

Final Report
10/6/2016
Pacific Engineering and Analysis
Prepared by

Walter Silva
Robert Darragh

Prepared for

SHN Engineers & Geologists
812 W. Wabash Avenue
Eureka, CA 95501-2138

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Tables

Figures

1.0 INTRODUCTION

Site-specific horizontal component design ground motions in terms of 5% damped response spectra and corresponding S_{DS} and S_{D1} values were developed for the Eureka High School Jay Willard Gymnasium following the California Geological Survey (CGS) Note 48 (CGS, 2013). The site is located in Eureka, California at 40.7882^{o}N latitude and 124.1578^{o}W longitude; within about 15 km rupture distance of the Cascadia subduction zone. At the high school the gym may either be replaced at a nearby location (within about 300 ft) or retrofit.

For Risk Categories I, II, and III the California Building Code (CBC, 2013) specifies Seismic Design Category E for seismic hazard conditions where the mapped spectral parameter S_1 exceeds 0.75g. For the Eureka High School Jay Willard Gymnasium the mapped S1 value for annual exceedence frequency (AEF) 4.04 x 10^{-4} (or 2% exceedence in 50 yrs (2,475 yr return period)) is 1.225g (Table 5), necessitating site-specific procedures specified in ASCE 7-10.

In addition, for the retrofit of existing buildings, ASCE 41-06, as modified by CBC 2013, requires the development of Basic Safety Earthquake (BSE) BSE-2 and BSE-1 level response spectra. According to CBC 2013, ASCE 41-06 is modified to define BSE-2 as the MCE_R spectra defined using ASCE 7-10 (Table 4). The BSE-1 is the lessor of the 10% in 50 year probability spectrum (475 yr return period) and two-thirds of the BSE-2 spectrum (Table 6). For the Eureka High School Jay Willard Gymnasium site the BSE ground motion parameters are listed in Table 7.

2.0 SITE DESCRIPTION

The Eureka High School Jay Willard Gymnasium site is located in northern California on soils (Figure 1). The site was recently the subject of a geotechnical investigation and geologic hazards study (SHN, 2016a) indicating that both site locations are classified as NEHRP Site Class D.

2.1 NEHRP Site Category and V_S (30m)

To assign a representative V_S (30m) to NEHRP Site Class D, a range of estimates was considered that accommodated a likely range in stiffness. For NEHRP Site Class D, the median V_S (30m) is 260m/s with an uncertainty of 0.21 (σ_{ln}) (Chiou et al., 2008) and reflects deep soils which dominate the strong motion database (Chiou et al., 2008). To accommodate uncertainty in

V_S (30m), values of 260m/s, the deep soil median estimate, as well as 210m/s, and 321m/s were taken to reflect a reasonable range of estimates at these sites.

3.0 SEISMIC SETTING

Figure 2 shows the Eureka High School Jay Willard Gymnasium site along with the significant faults which control the seismic hazard: Cascadia Subduction Zone (megathrust) dipping under the site, Little Salmon fault zone, San Andreas located to the south, and several other nearby crustal faults. The site has experienced ground shaking likely reflecting a wide range of intensity levels over historical times. Since the 1700's a number of significant earthquakes are known to have occurred on these active faults: The **M** 9 Cascadia megathrust earthquake of January 26, 1700, the 1906 **M** 7.9 Great San Francisco earthquake and 1992 **M** 7.2 Cape Mendocino earthquake, among others. To accommodate the recurrence of earthquakes on twenty-six identified capable faults were included in the analysis. Table 1 lists the source characterization for the critical subset of the crustal faults used in the hazard analysis. For unidentified potential crustal sources, a background zone of uniform seismicity appropriate for the Coast Ranges (Youngs et al., 1992) with a M_{max} of 6.5 ± 0.3 was also included in the hazard evaluation. The logic tree for the Cascadia Subduction Zone used in the hazard analysis is shown in Figures 3 and 4.

4.0 GROUND MOTION PREDICTION EQUATIONS (GMPEs).

NGA West1 and West2 GMPEs were used for crustal sources (Figure 2, Table 1) in the site-specific seismic hazard analysis. Specifically the GMPEs which included V_S (30m) scaling were included in the analyses. The NGA West1 suite included Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008), with equal weights. These earlier relationships were included as required by the California Building Code (2013). The NGA West2 suite included Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014), with equal weights. For these GMPEs appropriate default values for depth to top-of-rupture based on magnitude was used. For both the NGA West1 and West2 GMPEs, depths to 1.0 km/s and 2.5 km/s bedrock or basement material were set to default estimates based on Vs (30m). An envelope of the spectra from NGA West1 and West2 GMPEs was used for this project.

The GMPEs from Abrahamson et al. (2015) (BC Hydro (2012)), Zhao et al. (2006) and Atkinson and Boore (2003) were used for the Cascadia Subduction Zone source (Figures 2-4) in the site-specific seismic hazard analysis. The Abrahamson et al. (2015) (BC Hydro (2012)) equation was assigned a weight of 0.7, the Zhao et al. (2006) equation was assigned a weight of 0.2 and the Atkinson and Boore (2003) was assigned a weight of 0.1.

For intraslab sources (Figure 2) the same three GMPEs were used with slightly different weights. The Abrahamson et al. (2015) (BC Hydro (2012)) equation was assigned a weight of 0.7. Both the Zhao et al. (2006) and Atkinson and Boore (2003) equations were assigned a weight of 0.15

5.0 CONTROLLING EARTHQUAKES

Figure 5 illustrates the hazard curves contributions by source at 0.01s, peak ground acceleration (PGA). Figure 6 shows the source contributions at 1.0s. From Figures 5 and 6 the Cascadia Subduction Zone (Figures 2-4), and crustal faults including the Little Salmon, Ferndale, Russ, Mad River and San Andreas faults (Figure 2 and Table 1) reflect the dominant contribution to shaking hazard at the site. The intraslab and the background zone showing relatively less contributions to the hazard.

5.1 Hazard Deaggregations

Figures 8 and 9 show the hazard deaggregations at a return period of 2,475 years (AEF 4.04×10^{-4}) for 0.01s (PGA) and 1.0s respectively for Vs (30m) of 260 m/s. The two figures indicate that the dominant **M** around 9 is controlled by the Cascadia Subduction Zone at a closest rupture distance of 15 km. At 1.0s the contribution from a crustal fault with **M** near 7 at a closest rupture distance of 5 km is also significant.

6.0 DEVELOPMENT OF DESIGN MOTIONS

Following the CGS Note 48 (CGS, 2013), the California Building Code (2013), and ASCE 7-10, site-specific design motions (5% damped) were developed for the Eureka High School Jay Willard Gymnasium site. The specified process for developing site-specific design motions considers both probabilistic Uniform Hazard Spectra (UHS) adjusted for the maximum horizontal component (using results from Shahi and Baker (2013) for NGA West2 GMPEs) and risk targeted as well as deterministic 84th percentile spectra computed for the largest acceleration on all known active faults within the region, also adjusted for the maximum horizontal component.

6.1 Effects of V_S (30m)

To assess the effects of soil stiffness (V_S (30m)), both the UHS at a return period of 2,475 years (Figure 9) as well as deterministic 84th percentile spectra (e.g. Figure 10) were developed for V_S (30m) of 210, 260, and 321m/s (Section 2.0) using the GMPEs (Section 4.0). For the GMPEs which have depth to 1.0 km/s or 2.5 km/s bedrock or basement material, developer recommended default values based on V_S (30m) estimates were used (Section 4.0). As Figures 9 and 10 illustrate, the UHS as well as the deterministic 84th percentile spectra from the Little Salmon Fault Zone scenario cross at a period around 1.0s with no single V_S (30m) having the largest response spectral value across period. As a result an envelope over the three V_S (30m) estimates was selected to characterize the hazard. Further site characterization would be required to reduce the uncertainty in V_S (30m) for the soil.

Figure 11 shows the 84th percentile deterministic hazard for the Cascadia megathrust **M** 9 earthquake from the three GMPEs appropriate for subduction events and their weighted mean. The weighted mean is similar to the BC Hydro GMPE spectrum (Abrahamson et al. (2015)) because this equation was assigned the largest weight of 0.7.

The deterministic 84th percentile spectra (Figure 12) were based on the envelope of the spectra from the Little Salmon Fault Zone rupture with **M** 7.5 with a closest rupture distance of 3.4 km on the hanging wall (R_{JB} of 0.0 km) with a reverse mechanism (Figure 10, Table 1 lists dip of 30°NE) and a Cascadia Subduction Zone megathrust earthquake with **M** 9.0 with a closest rupture distance of 15 km (Figure 11). The spectrum from the Little Salmon Fault Zone earthquake has the largest response at nearly all periods. The spectrum from the Cascadia Subduction Megathrust event dominates at only 0.10 and 0.15s.

6.2 Deterministic MCE_R Ground Motions

Site-specific deterministic 84th percentile motions developed from the envelope of the **M** 7.5 Little Salmon Fault Zone rupture and the **M** 9 Cascadia megathrust earthquake are shown in Figure 13 for the geometric mean as well as maximum horizontal direction (correction factors from Shahi and Baker, 2013) along with the minimum deterministic spectrum (ASCE 7-10). The MCE_R was taken as the deterministic 84th percentile spectrum adjusted for maximum direction. The associated spectra and maximum direction factors (Shahi and Baker, 2013) are listed in Table 2.

6.3 Probabilistic MCE_R Ground Motions

Figure 14 shows the 2% in 50 year, 2,475 year return period, UHS developed for the geometric mean as well as adjusted for both maximum direction (Shahi and Baker, 2013) and risk coefficients (ASCE 7-10). The MCE_R was taken as the 2,475 year UHS adjusted for maximum horizontal direction and risk coefficients. The geometric mean, MCE_R, maximum direction factor, and risk coefficients are listed on Table 3.

6.4 Site-Specific MCE_R

The site-specific MCE_R was taken as the lesser of the deterministic MCE_R (Section 6.2) and probabilistic MCE_R (Section 6.3) and is shown in Figure 15 and listed in Table 4.

6.5 Design Response Spectrum (DRS)

The design response spectrum, taken as 2/3 of the site-specific MCE_R, is shown in Figure 16 with all values listed in Table 4. The lower limit of the DRS is 80% of the code based design spectrum with NEHRP site class D adjustments.

Finally, Figure 17 shows the ASCE 7-10 design spectra. They are the site-specific MCE_R (Section 6.4) and the DRS which is 2/3 of the site-specific MCE_R (Section 6.5). Table 5 summarizes the recommended design parameters for the site based on ASCE 7-10.

6.6 BSE-2 and BSE-1 Level Response Spectra

CBC 2013 modifies ASCE 41-06 to define the site-specific BSE-2 as the MCE_R. The BSE-2 spectrum is shown in Figure 15 (as the MCE_R) and listed in Table 4. Tables 4 and 6 also list two-thirds of the BSE-2 used in the BSE-1 calculation.

Figure 18 shows the 10% in 50 year, 475 year return period, UHS developed for the geometric mean as well as adjusted for both maximum direction (Shahi and Baker, 2013) and risk coefficients (ASCE 7-10). The geometric mean, maximum direction factor, and risk coefficients are listed on Table 6.

BSE-1 is defined as the lessor of the 10% in 50 year probability spectrum (475 yr return period, maximum direction and risk adjusted) and two-thirds of the BSE-2 spectrum. Figure 19 shows these three spectra with all values listed in Table 6. Table 7 summarizes the recommended design parameters S_{XS}, S_{X1} and T_S and for the BSE-1 and BSE-2 at the site based on ASCE 41-06 as modified by CBC 2013.

7.0 LIMITATIONS

The development of design ground motions for the Eureka High School Jay Willard Gymnasium strictly followed specified protocols documented specifically in the current CGS Note 48 (2013), California Building Code (2013), and ASCE 7-10. Assessment of earthquake shaking hazard is a rapidly changing field both technically as knowledge broadens as well as in changes in controlling standards. While this report reflects current knowledge, standard practice, and appropriate codes, changes over time may invalidate recommendations in this report.

8.0 References:

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ASCE 41-06, 2007, Seismic rehabilitation of existing buildings, American Society of Civil Engineers, ASCE/SEI 41-06 (*Section 1.6- Seismic Hazard).*

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Atkinson, G.M., and Boore, D.M., 2003, Empirical ground-motion relations for subduction zone earthquakes and their applications to Cascadia and other regions: Bulletin of the Seismological Society of America, v. 93, p. 1703-1729.

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Table 1. Crustal Fault Sources (selected)

Fault Name	**Rupture Model**	**Section Name**	**Rupture Length (km)**	**M_{max}[1] (M)**	**Sense of Slip**	**Dip[2,3] (degrees)**	**Rupture Depth (km)[4]**	**Slip Rate mm/yr[3]**	**Selected References**
Right Lateral Faults									
San Andreas fault zone									
	4 segments (0.15)	Shelter Cove (SAO)	135	7.3	RL	90	12	21 (0.2) 24 (0.6) 27 (0.2)	Prentice *et al.*, 1999
		SAN	190	7.4	RL	90	12	21 (0.2) 24 (0.6) 27 (0.2)	Cao *et al.*, 2003
		SAP	85	7.1	RL	90	13	13 (0.2) 17 (0.6) 21 (0.2)	Cao *et al.*, 2003
		SAS	62	7.0	RL	90	15	13 (0.2) 17 (0.6) 21 (0.2)	Cao *et al.*, 2003
	2 segments (0.15)	SAO+SAN	325	7.7	RL	90	12	21 (0.2) 24 (0.6) 27 (0.2)	Cao *et al.*, 2003
		SAP+SAS	147	7.4	RL	90	14	13 (0.2) 17 (0.6) 21 (0.2)	Cao *et al.*, 2003
	unsegmented (0.7)	1906 rupture	472	8.1	RL	90	12	15 (0.2) 21 (0.6) 27 (0.2)	
Garberville-Briceland	unsegmented (1.0)		60	6.8	RL-R	80 ± 10 N	12	1 (0.2) 5 (0.6) 9 (0.2)	Freymueller *et al.*, 2002
Grogan fault	unsegmented (0.5)		139	7.6	RL-R	75 E (0.4) 90 (0.4) 75 W (0.2)	12	0.2 (0.2) 0. 4(0.6) 1.0 (0.2)	Kelsey and Carver, 1988
	segmented (0.5)	floating	45	7.0	RL-R	75 E (0.4) 90 (0.4) 75 W 0.2)	12	0.2 (0.2) 0. 4(0.6) 1.0 (0.2)	Kelsey and Carver, 1988; WWC, 1995
Eaton Roughs fault	unsegmented (1.0)		66	7.1	RL	90	12	1.0 (0.2) 2.0 (0.6) 4.0 (0.2)	Kelsey and Carver, 1988
Reverse Faults									
Bald Mountain- Big Lagoon fault									
	unsegmented (0.3)		125	7.5	R	30 ± 5 E	12	1 (0.2) 1.5 (0.6) 2.0 (0.2)	Hart, 1999; Carver, 1992; McCrory, 1996; Geomatrix, 1995
	segmented (0.7)								
		Bald Mountain (onshore)	60	7.1	R	30 ± 5 E	12	0.01 (0.2) 1.0 (0.6) 2.0 (0.2)	Hart, 1999; Carver, 1992; McCrory, 1996
		Big Lagoon (onshore)	18	6.5	R	30 ± 5 E	12	0.01 (0.2) 1.0 (0.6) 2.0 (0.2)	Hart, 1999; Carver, 1992; McCrory, 1996

Fault Name	**Rupture Model**	**Section Name**	**Rupture Length (km)**	**M_{max}[1] (M)**	**Sense of Slip**	**Dip[2,3] (degrees)**	**Rupture Depth (km)[4]**	**Slip Rate mm/yr[3]**	**Selected References**
		offshore	65	7.2	R	30 E (0.5) 70 E (0.5)	12	0.01 (0.1) 0.1 (0.2) 1.0 (0.6) 2.0 (0.1)	Geomatrix, 1995; Hart, 1999
Mad River fault	unsegmented (1.0)		42	7.0	R	30 ± 5 E	12	0.5 (0.2) 0.7 (0.6) 0.9 (0.2)	Kelsey and Carver, 1988; McCrory, 1996
		offshore	40	6.9	R	40 E	12	1.0 (0.2) 2.0 (0.6) 3.0 (0.2)	McCrory,1996
Little Salmon fault zone	segmented (0.8)	onshore	34	6.9	R	30N E	13	2 (0.2) 4.3 (0.5) 8 (0.3)	Cao *et al.*, 2003; McCrory 1996; Field et al. (2013)
		offshore	46	7.1	R	30NE	13	1 (0.2) 2.3 (0.6) 4.5 (0.2)	
	unsegmented (0.2)		80	7.3	R	30NE	13	2 (0.2) 4.3 (0.5) 8 (0.3)	
Little Salmon Fault zone	unsegmented (1.0)		81	7.2	R/RL	75 N 0.7) 75 S (0.3)	10 (0.3) 13 (0.6) 20 (0.1)	0.1 (0.2) 0.26 (0.6) 1.5 (0.2)	McCrory 1996; Field et al. (2013); USGS Quaternary fault database
Ferndale Fault	unsegmented (1.0)		25	6.7	R	90 (0.2) 60 (0.6) 45 (0.2)	13	0.01 (0.3) 0.1 (0.4) 0.2 (0.3)	McLaughlin et al. (2000); Field et al. (2013); USGS Quaternary fault database

[1] Preferred values calculated using empirical relationships of rupture length and magnitude for all fault types (Wells and Coppersmith, 1994) and rupture area and magnitude (Hanks and Bakun, 2002). For normal and reverse faults, we use only Wells and Coppersmith; for strike-slip faults, we use both Wells and Coppersmith and Hanks and Bakun, each weighted equally.

[2] Average crustal dips. Faults are assumed to be planar.

[3] Unless otherwise noted, weightings are 0.2 for the upper and lower bounds of the range and 0.6 for the mean.

[4] Depths have an uncertainty of ± 2 km unless otherwise noted.

Table 2
Calculation of Deterministic MCE as per ASCE 7-10, Chapter 21

Period	84th deterministic spectrum, Geometric Mean	Maximum Direction Factor (Shahi and Baker, 2013)	84th deterministic spectrum, Maximum Direction	Minimum deterministic Spectrum per ASCE 7-10	Site-Specific Deterministic MCE
(sec)	Sa(g)		Sa(g)	Sa(g)	Sa(g)
0.01	1.25	1.19	1.49	0.60	1.49
0.02	1.26	1.19	1.50	0.75	1.50
0.03	1.35	1.19	1.60	0.83	1.60
0.05	1.49	1.19	1.77	0.98	1.77
0.075	1.69	1.19	2.01	1.16	2.01
0.1	1.96	1.19	2.33	1.35	2.33
0.15	2.29	1.20	2.74	1.50	2.74
0.2	2.48	1.21	3.00	1.50	3.00
0.25	2.73	1.22	3.33	1.50	3.33
0.3	3.00	1.22	3.66	1.50	3.66
0.4	3.25	1.23	4.00	1.50	4.00
0.5	3.21	1.23	3.94	1.50	3.94
0.75	2.71	1.24	3.36	1.20	3.36
1.0	2.22	1.24	2.75	0.90	2.75
1.5	1.78	1.24	2.21	0.60	2.21
2.0	1.47	1.24	1.82	0.45	1.82
3.0	0.89	1.25	1.11	0.30	1.11
4.0	0.55	1.26	0.69	0.23	0.69
5.0	0.38	1.26	0.47	0.18	0.47

Table 3
Calculation of Probabilistic MCE_R as per ASCE 7-10, Chapter 21

Period	2,475-yr UHS, Geometric Mean	Maximum Direction Factor (Shahi and Baker, 2013)	2,475-yr UHS, Maximum Direction	Risk Coefficient per ASCE 7-10	Site-Specific Probabilistic MCE_R
(sec)	Sa(g)		Sa(g)		Sa(g)
0.01	1.75	1.19	2.08	0.85	1.78
0.02	1.80	1.19	2.15	0.85	1.83
0.03	1.84	1.19	2.18	0.85	1.87
0.05	1.89	1.19	2.25	0.85	1.92
0.07	2.32	1.19	2.77	0.85	2.36
0.10	2.74	1.19	3.26	0.85	2.79
0.15	3.25	1.20	3.91	0.85	3.34
0.20	3.62	1.21	4.38	0.85	3.74
0.25	3.97	1.22	4.83	0.85	4.13
0.30	4.27	1.22	5.21	0.85	4.46
0.40	4.55	1.23	5.60	0.86	4.79
0.50	4.28	1.23	5.27	0.86	4.51
0.75	3.50	1.24	4.34	0.86	3.73
1.00	2.78	1.24	3.45	0.86	2.97
1.50	2.11	1.24	2.62	0.86	2.25
2.00	1.65	1.24	2.04	0.86	1.76
3.00	0.94	1.25	1.18	0.86	1.02
4.00	0.58	1.26	0.73	0.86	0.63
5.00	0.40	1.26	0.51	0.86	0.43

Table 4
Calculation of Site-Specific MCE$_R$ (BSE-2) and DRS as per ASCE 7-10, Chapter 21 and ASCE 41-06

Period	Site-Specific MCE$_R$ (BSE-2)	Two-thirds MCE$_R$ (2/3rds BSE-2)	80% General Code DRS, Site Class D	Site-Specific DRS
(sec)	Sa(g)	Sa(g)	Sa(g)	Sa(g)
0.01	1.49	0.99	0.76	0.99
0.02	1.50	1.00	0.85	1.00
0.03	1.60	1.07	0.93	1.07
0.05	1.77	1.18	1.11	1.18
0.08	2.01	1.34	1.33	1.34
0.10	2.33	1.55	1.54	1.55
0.12	2.48	1.65	1.69	1.69
0.15	2.74	1.83	1.69	1.83
0.20	3.00	2.00	1.69	2.00
0.25	3.33	2.22	1.69	2.22
0.30	3.66	2.44	1.69	2.44
0.40	4.00	2.67	1.69	2.67
0.50	3.94	2.63	1.69	2.63
0.75	3.36	2.24	1.31	2.24
1.00	2.75	1.84	0.98	1.84
1.50	2.21	1.48	0.65	1.48
2.00	1.76	1.17	0.49	1.17
3.00	1.02	0.68	0.33	0.68
4.00	0.63	0.42	0.25	0.42
5.00	0.43	0.29	0.20	0.29

Table 5
Site-Specific Ground Motion Parameters according to ASCE 7-10

Parameter	Description	Value
S_S	Mapped Short Period (0.2 sec) Spectral Acceleration Value (Site Class B)	3.16g
S_1	Mapped Long Period (1.0 sec) Spectral Acceleration Value (Site Class B)	1.225g
Seismic Design Category	Based on 1.0 sec mapped spectral acceleration, S1, as per 2013 CBC Section 1613A.3.5	E
F_a	Site Class D	1.0
F_v	Site Class D	1.5
C_{RS}	Short period risk coefficient	0.854
C_{R1}	Long Period risk coefficient	0.861
S_{DS}, Site-Specific	Spectral value at 0.2 seconds (but not less than 90% SA for all periods greater than 0.2 seconds)	2.40
S_{D1}, Site-Specific	Spectral value at 1.0 seconds (but not less than twice the 2.0 sec Sa)	2.35
S_{MS}, Site-Specific	$1.5*S_{DS}$	3.60
S_{M1}, Site-Specific	$1.5*S_{D1}$	3.52
PGA_M	Site-Specific MCE_G peak ground acceleration	1.25
M	Magnitude for liquefaction analysis based on deterministic analysis	7.5
R	Distance for liquefaction analysis based on deterministic analysis	1.0 km

Table 6
Calculation of BSE-1 as per ASCE 41-06 as modified by CBC 2013

Period	475-yr UHS, Geometric Mean	Max Direction Factor (Shahi and Baker, 2013)	Risk Coefficient per ASCE 7-10	Site-Specific maximum direction, risk adjusted 475-year UHS	Two-thirds BSE-2 (MCE_R)	BSE-1
(sec)	**Sa(g)**			**Sa(g)**	**Sa(g)**	**Sa(g)**
0.01	0.82	1.19	0.85	0.84	0.99	0.84
0.05	0.93	1.19	0.85	0.95	1.18	0.95
0.1	1.40	1.19	0.85	1.42	1.55	1.42
0.15	1.67	1.20	0.85	1.71	1.83	1.71
0.2	1.82	1.21	0.85	1.88	2.00	1.88
0.3	1.92	1.22	0.85	2.01	2.44	2.01
0.4	1.90	1.23	0.86	2.00	2.67	2.00
0.5	1.74	1.23	0.86	1.83	2.63	1.83
0.75	1.41	1.24	0.86	1.51	2.24	1.51
1.0	1.19	1.24	0.86	1.27	1.84	1.27
1.5	0.86	1.24	0.86	0.92	1.48	0.92
2.0	0.66	1.24	0.86	0.70	1.17	0.70
3.0	0.39	1.25	0.86	0.42	0.68	0.42
4.0	0.26	1.26	0.86	0.29	0.42	0.29
5.0	0.18	1.26	0.86	0.20	0.29	0.20

Table 7
Site-Specific Ground Motion Parameters according to ASCE 41-06 as modified by CBC 2013

Parameter	Description	Value
BSE-1 S_{XS}, Site-Specific	Spectral value at 0.2 seconds from BSE-1 (but not less than 90% Sa for all other periods)	1.88
BSE-1 S_{X1}, Site-Specific	Spectral value at 1.0 seconds from BSE-1 (but not less than that which would produce Sa=Sx1/T at each period not less than 90% of that obtained from the BSE-1 spectrum at that period)	1.27
BSE-1, Ts, Site-Specific	$=S_{X1}/S_{XS}$	0.68
BSE-2 S_{XS}, Site-Specific	Spectral value at 0.2 seconds from BSE-2 (but not less than 90% Sa for all other periods)	3.00
BSE-2 S_{X1}, Site-Specific	Spectral value at 1.0 seconds from BSE-2 (but not less than that which would produce Sa=Sx1/T at each period not less than 90% of that obtained from the BSE-2 spectrum at that period)	3.17
BSE-2, Ts, Site-Specific	$=S_{X1}/S_{XS}$	1.06

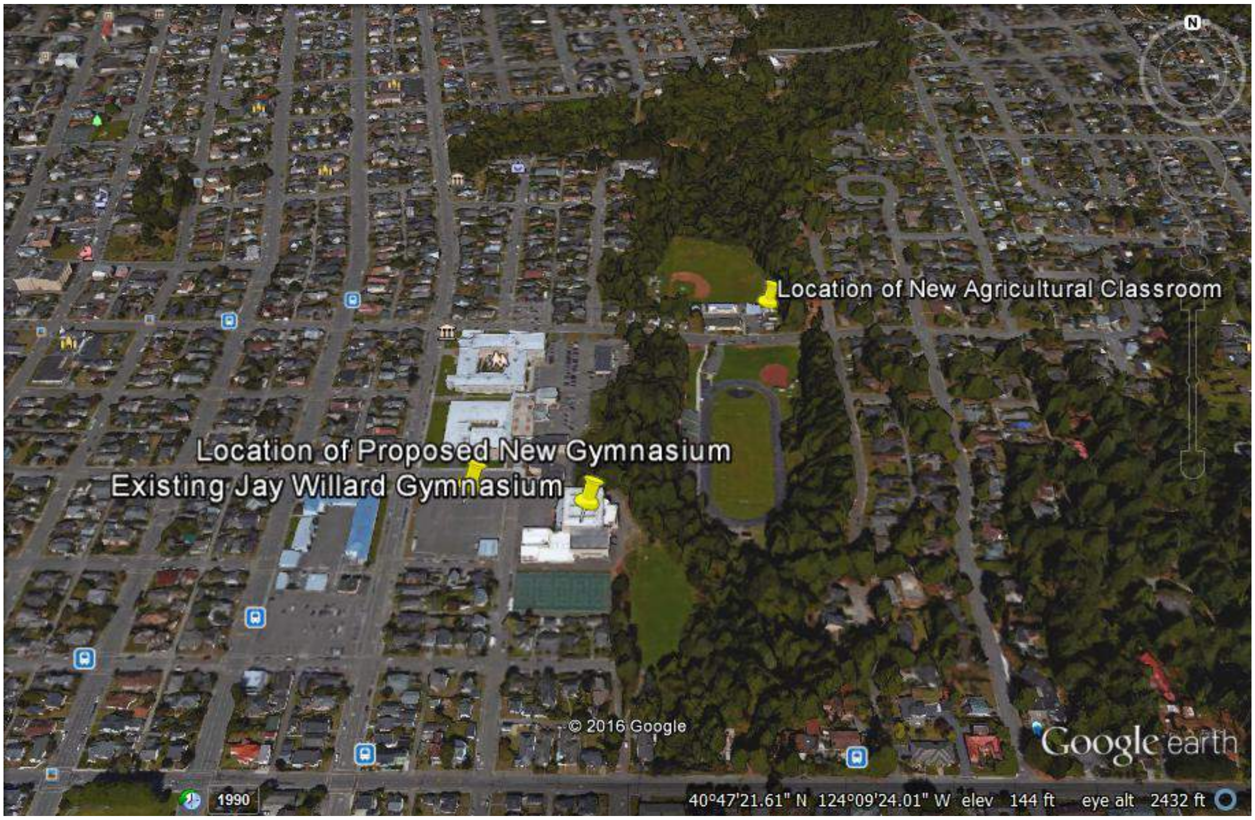


Figure 1. Site location from SHN (personal communication, 2016).

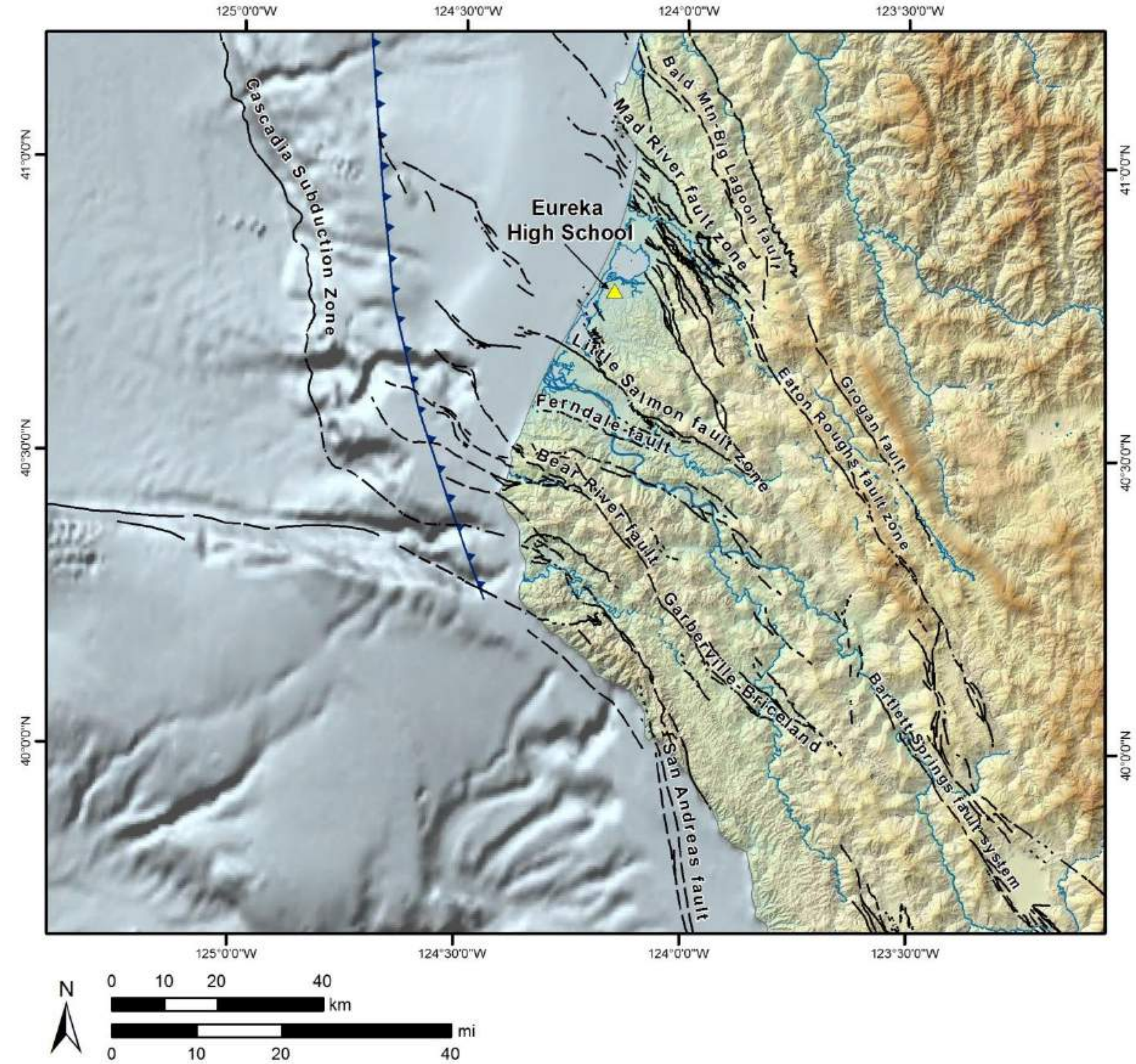
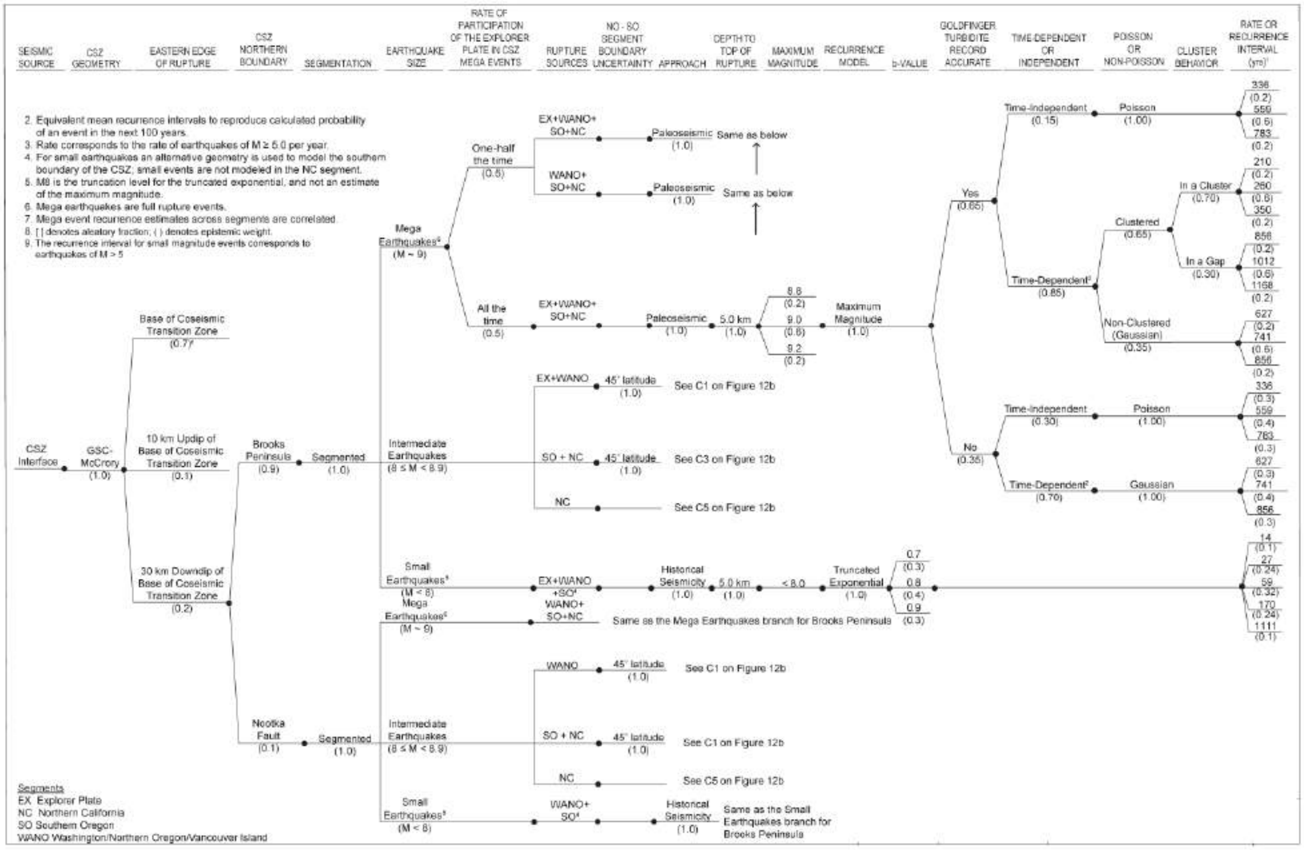


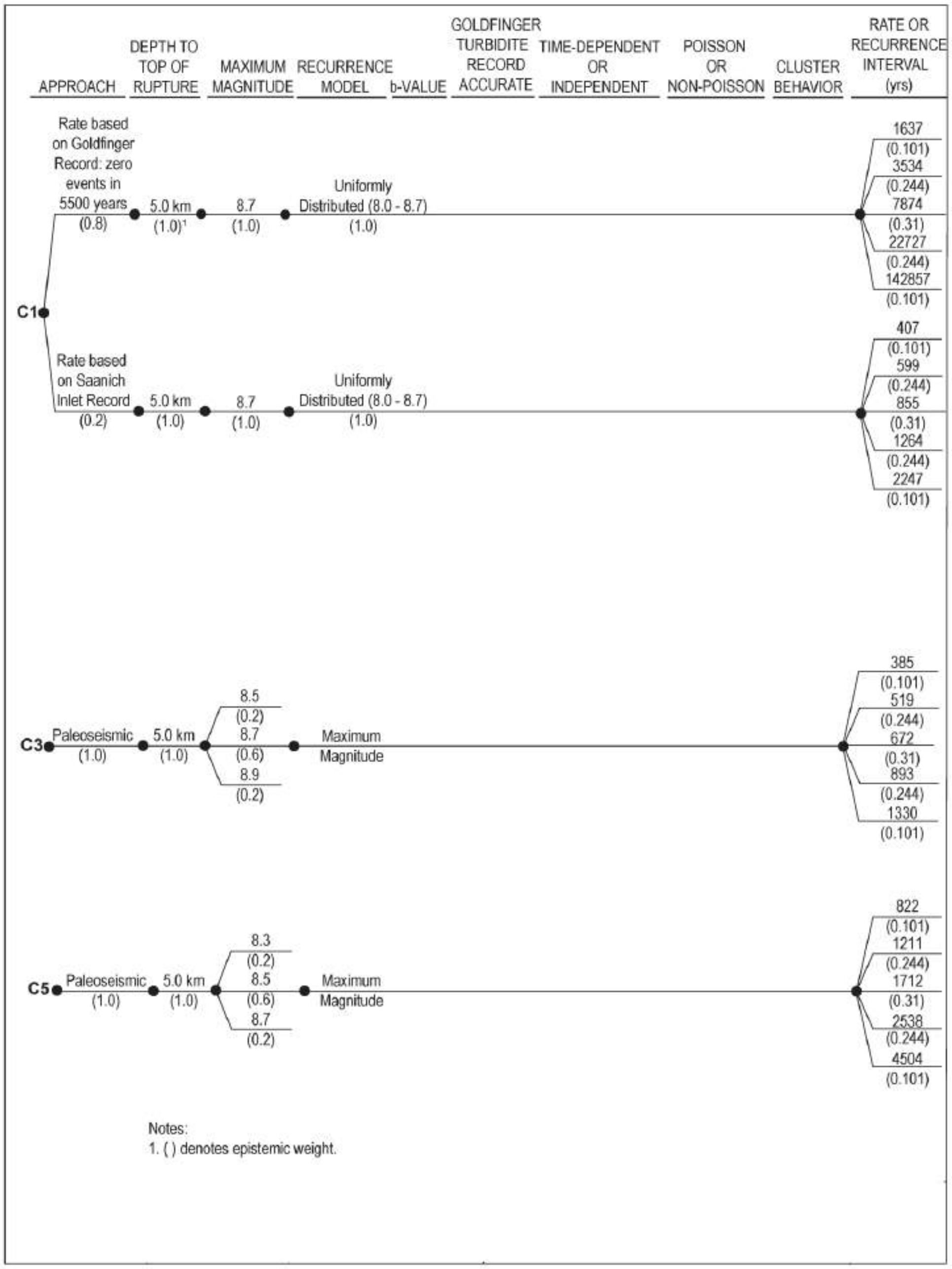
Figure 2. Site location and nearby earthquake faults.



2. Equivalent mean recurrence intervals to reproduce calculated probability of an event in the next 100 years.
3. Rate corresponds to the rate of earthquakes of M ≥ 5.0 per year.
4. For small earthquakes an alternative geometry is used to model the southern boundary of the CSZ; small events are not modeled in the NC segment.
5. M8 is the truncation level for the truncated exponential, and not an estimate of the maximum magnitude.
6. Mega earthquakes are full rupture events.
7. Mega event recurrence estimates across segments are correlated.
8. [ ] denotes aleatory fraction; ( ) denotes epistemic weight.
9. The recurrence interval for small magnitude events corresponds to earthquakes of M > 5

Segments
EX Explorer Plate
NC Northern California
SO Southern Oregon
WANO Washington/Northern Oregon/Vancouver Island

Figure 3. Cascadia subduction zone (megathrust) logic tree.



Notes:
1. ( ) denotes epistemic weight.

Figure 4. Cascadia subduction zone (megathrust) logic tree (continued).

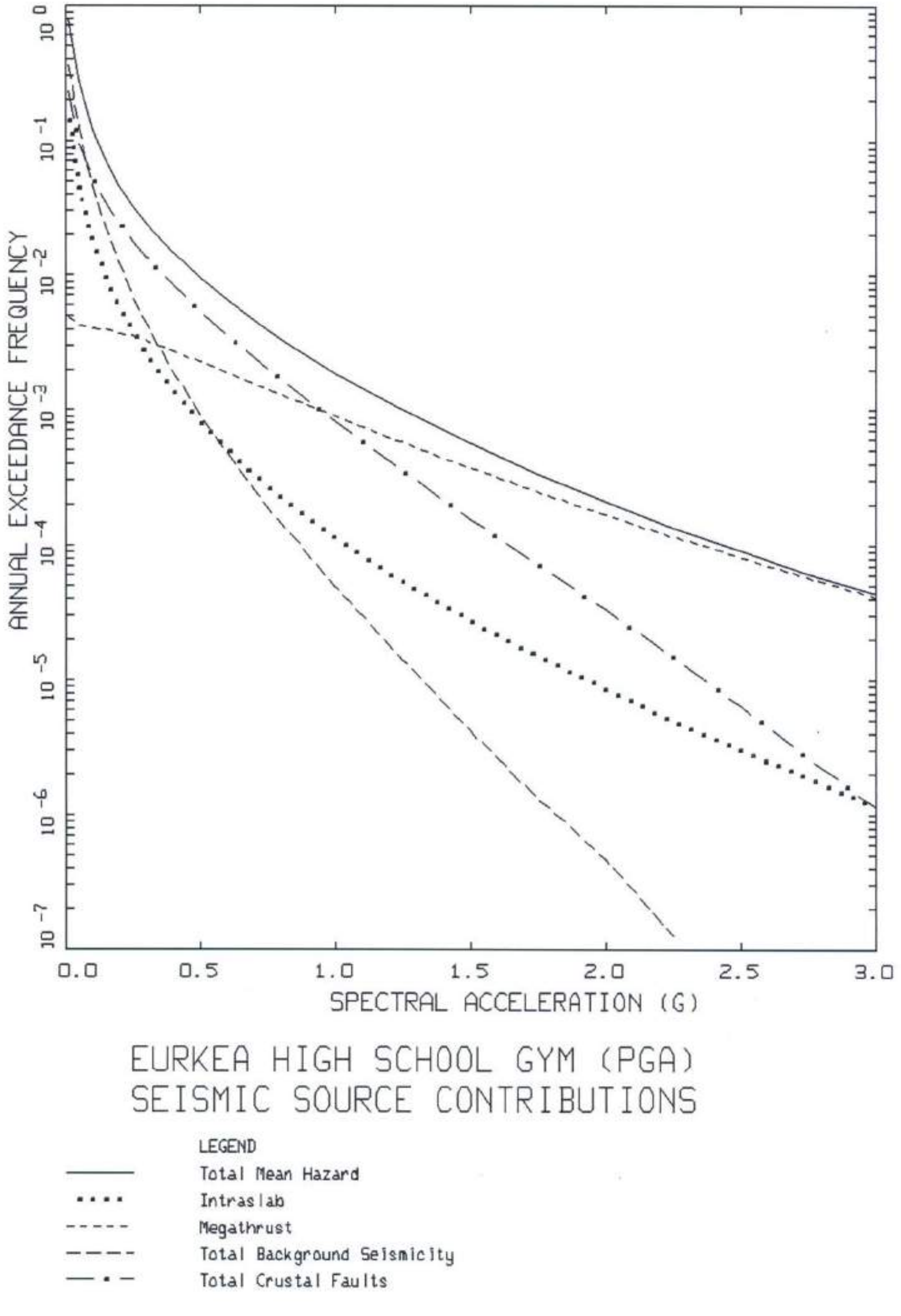


Figure 5. Annual Exceedence Frequencies of main contributors to peak acceleration (0.01s). V_S (30m) = 260m/s.

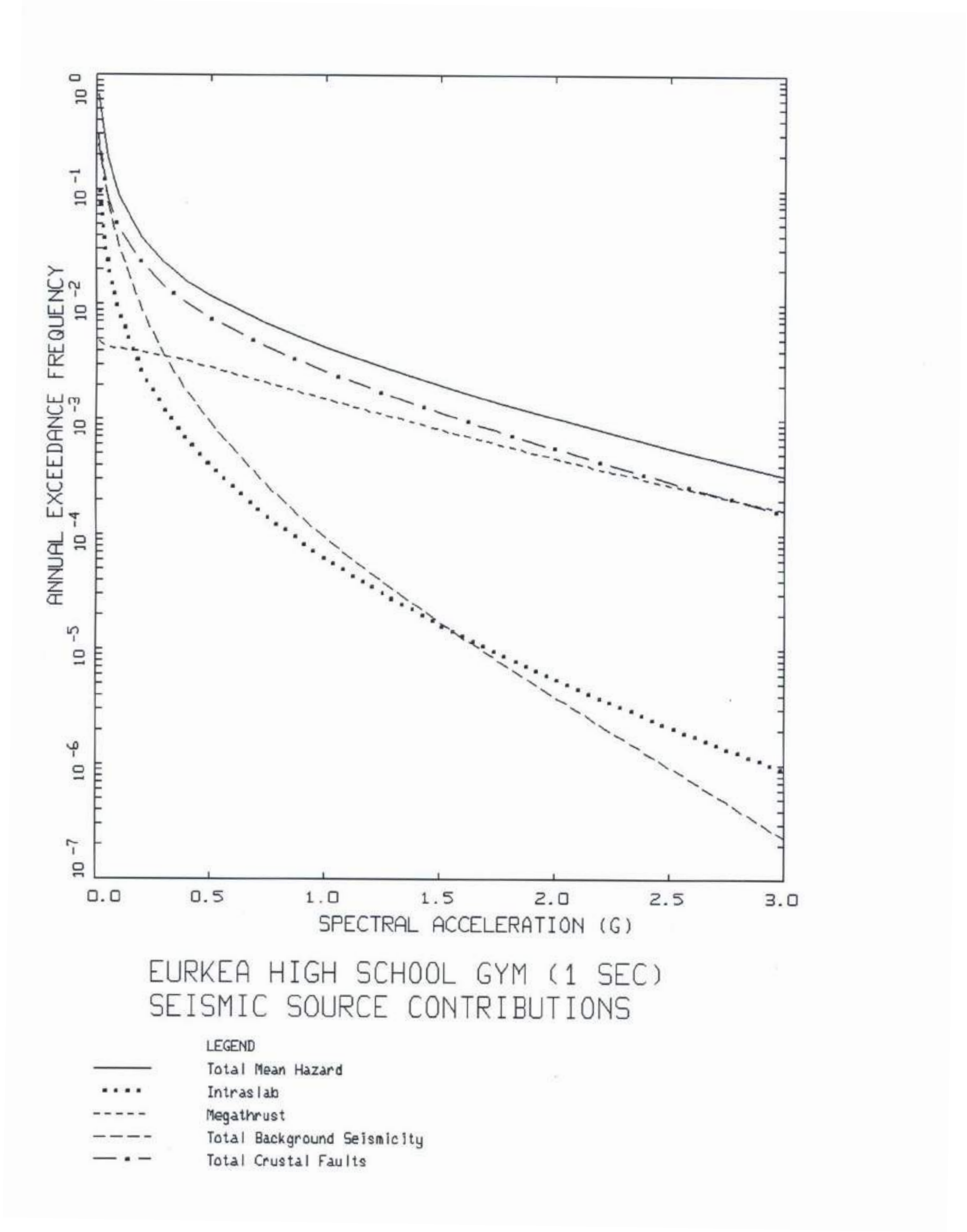


Figure 6. Annual Exceedence Frequencies of main contributors to spectral acceleration at 1.0s.
V_S (30m) = 260m/s.

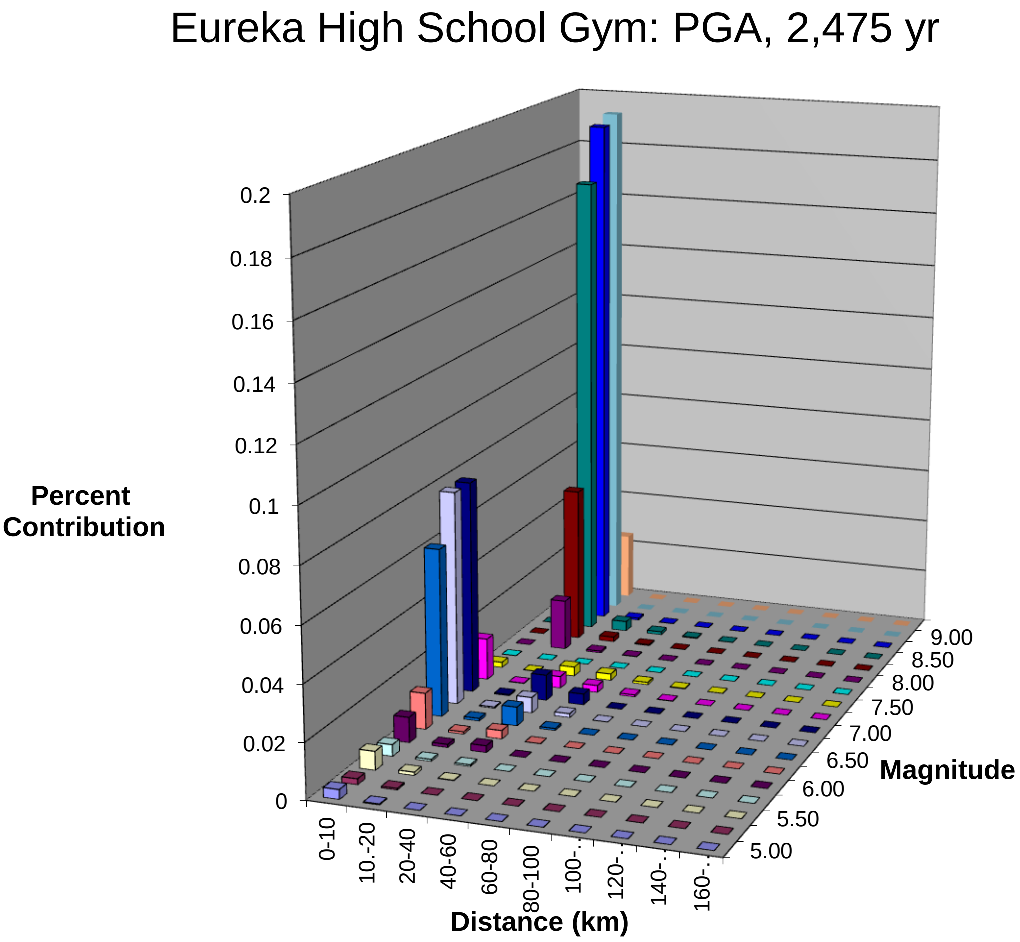


Figure 7. Magnitude and distance deaggregation at AEF 4.04 x 10^{-4} (2,475 yr return period) for structural frequency 0.01s (PGA). V_S (30m) = 260m/s.

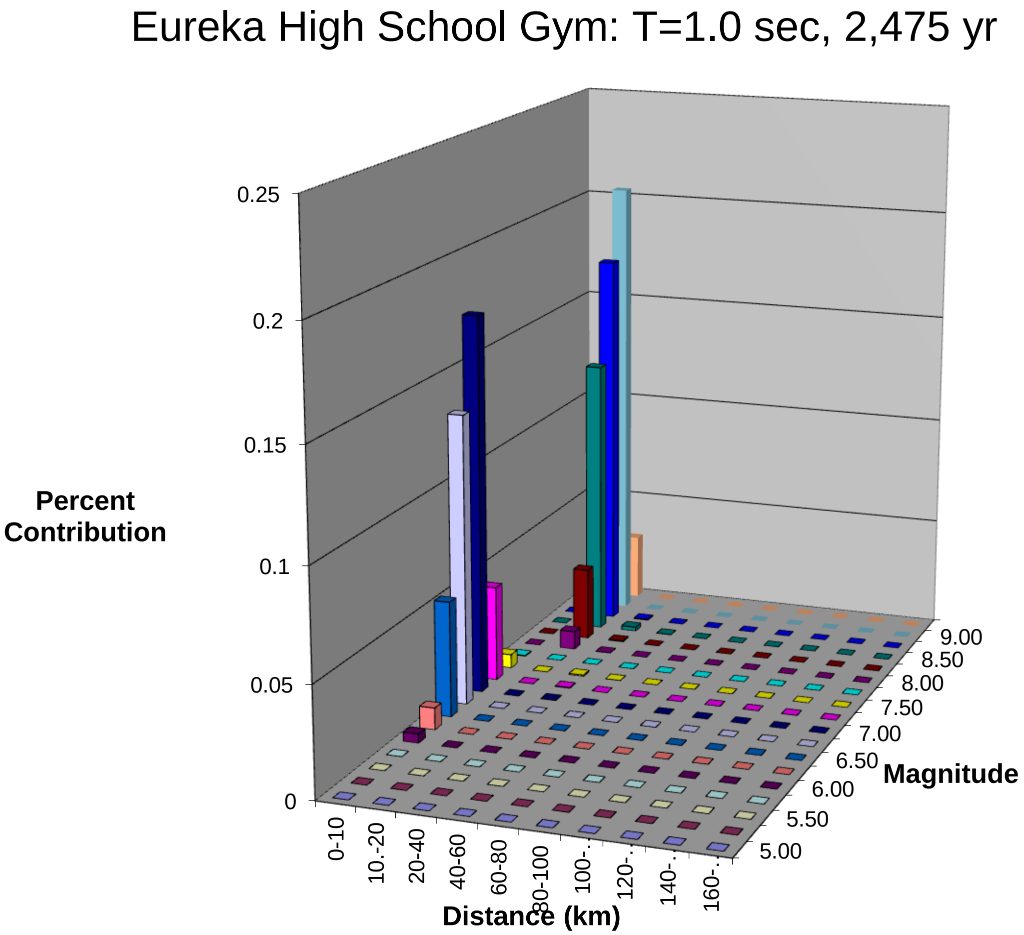


Figure 8. Magnitude and distance deaggregation at AEF 4.04 x 10^{-4} (2,475 yr return period) for structural frequency 1.0s. V_S (30m) = 260m/s.

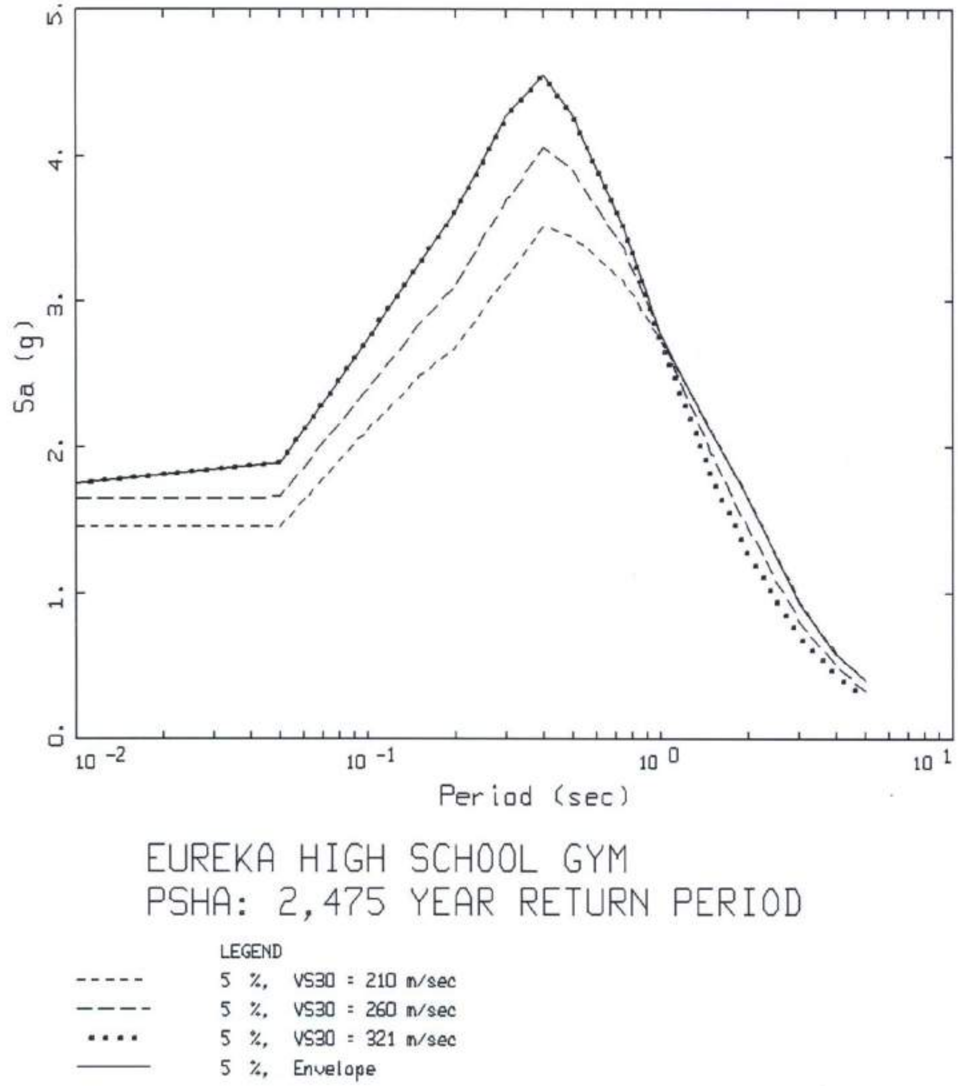


Figure 9. Uniform Hazard Spectra at a return period of 2,475 yr; comparison of V_S(30m).

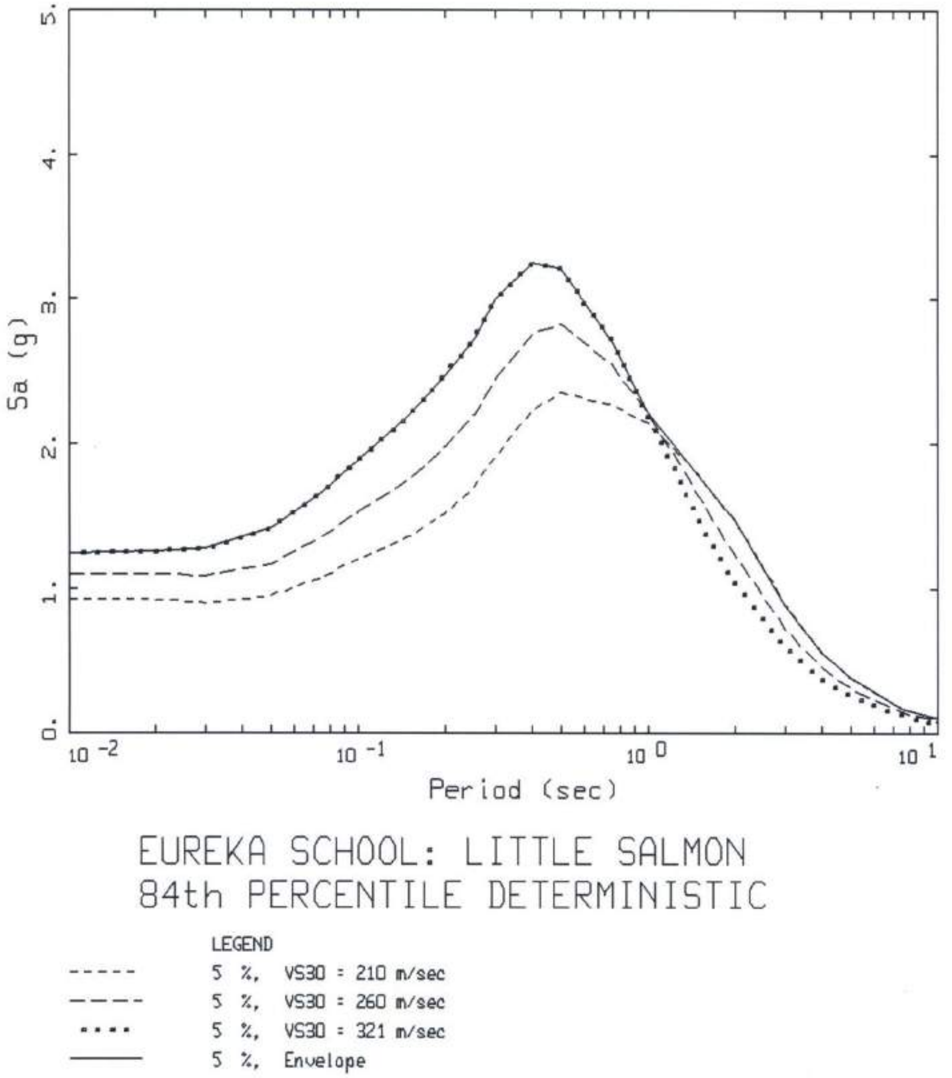


Figure 10. Deterministic 84th percentile spectra for Little Salmon Fault Zone earthquake with **M** 7.5, rupture distance 3.4 km, hanging wall, reverse mechanism, crustal GMPEs; comparison of V_S (30m).

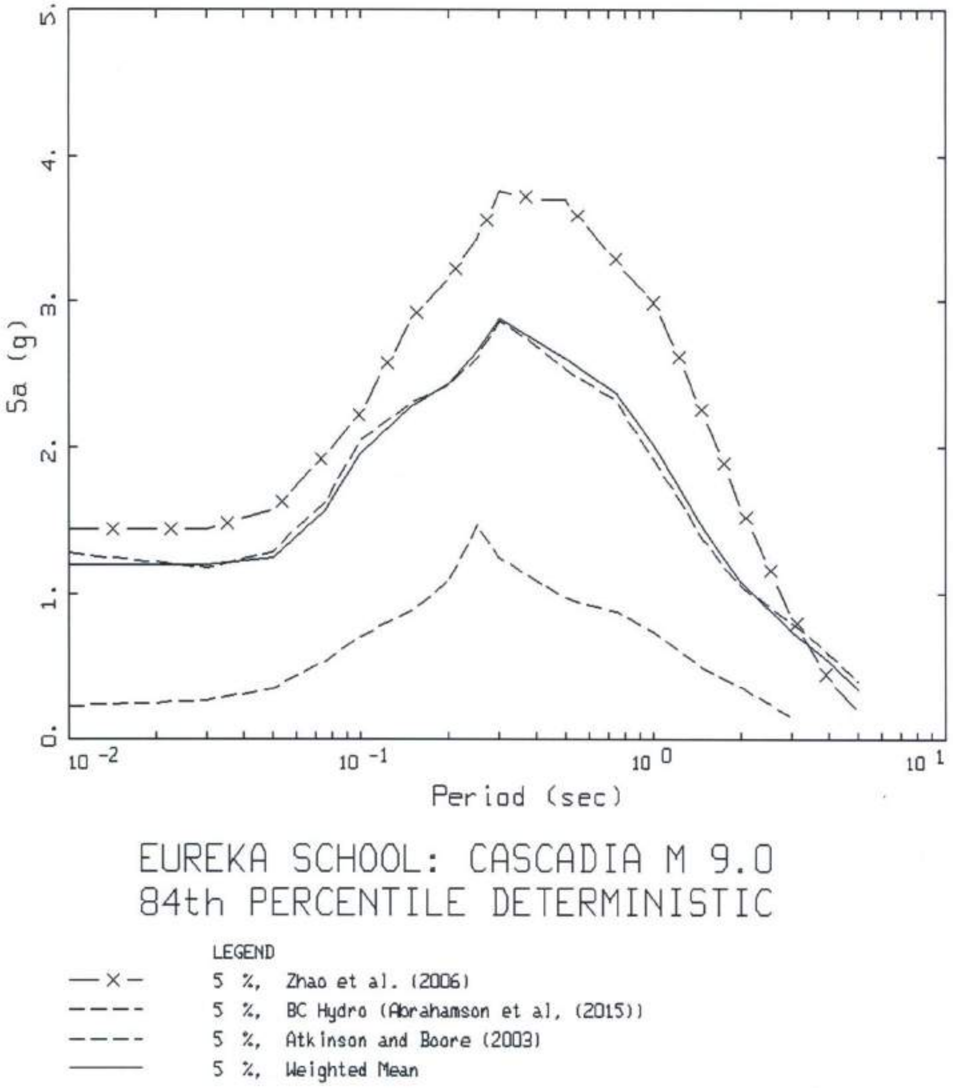


Figure 11. Deterministic 84th percentile spectra for **M** 9.0, rupture distance 15.0 km, thrust mechanism, Cascadia Subduction zone GMPEs; weighted mean.

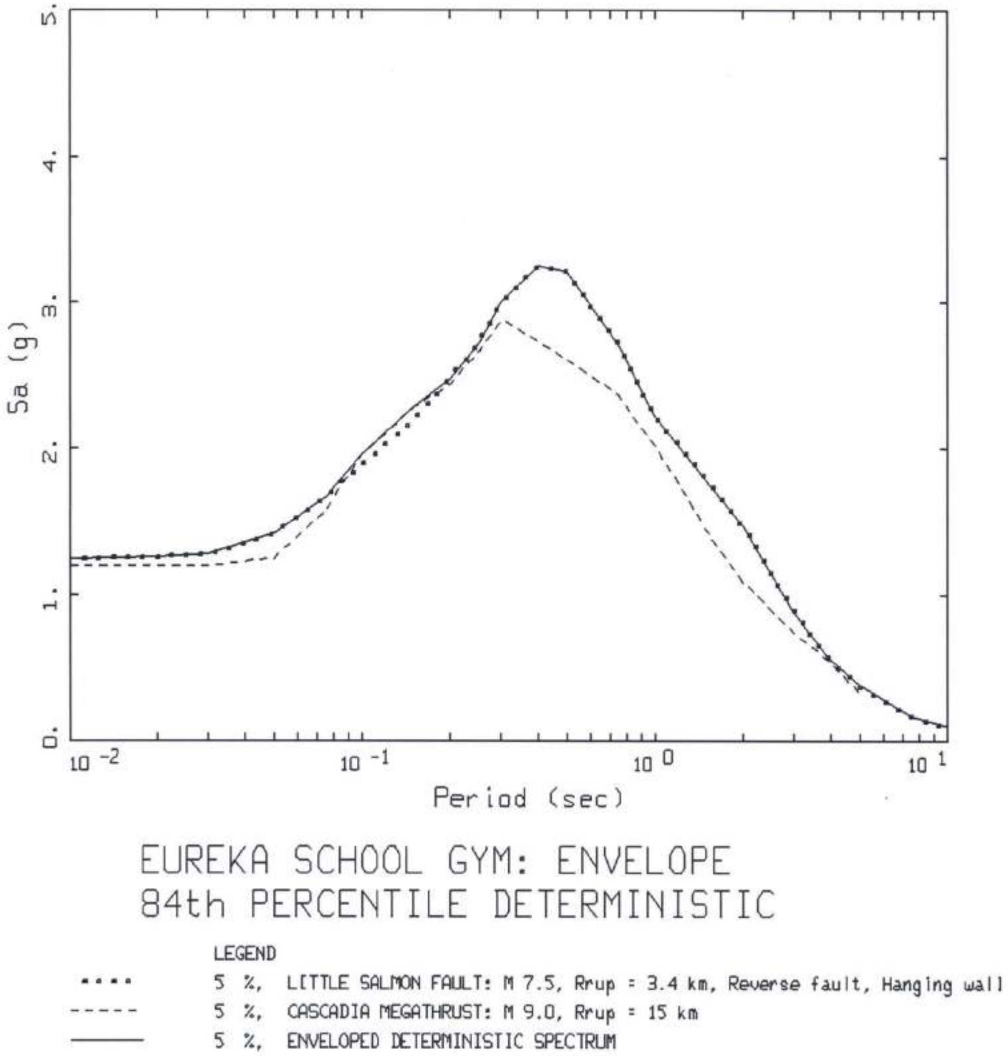


Figure 12. Deterministic envelope of Little Salmon Fault Zone (Figure 10) and Cascadia megathrust (Figure 11).

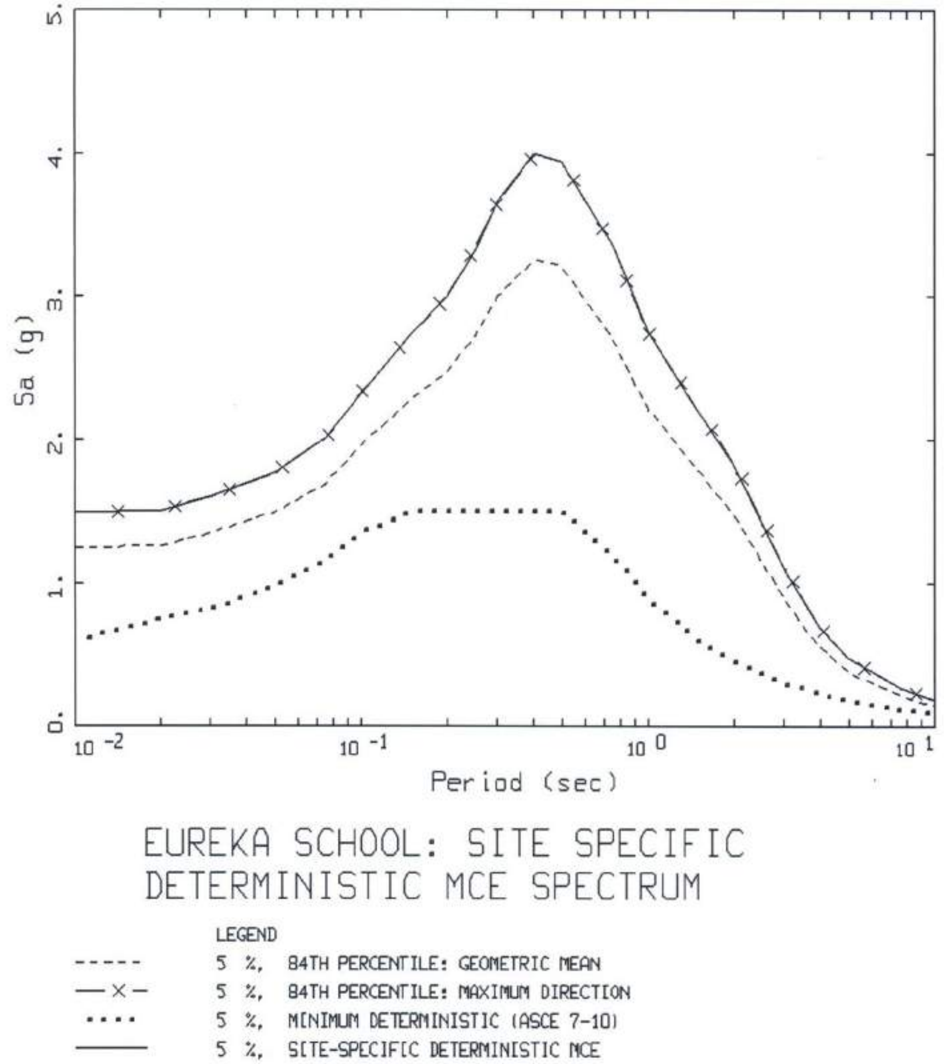


Figure 13. Development of site-specific deterministic MCE_R; envelop Little Salmon Fault Zone and Cascadia megathrust; envelop V_S (30m).

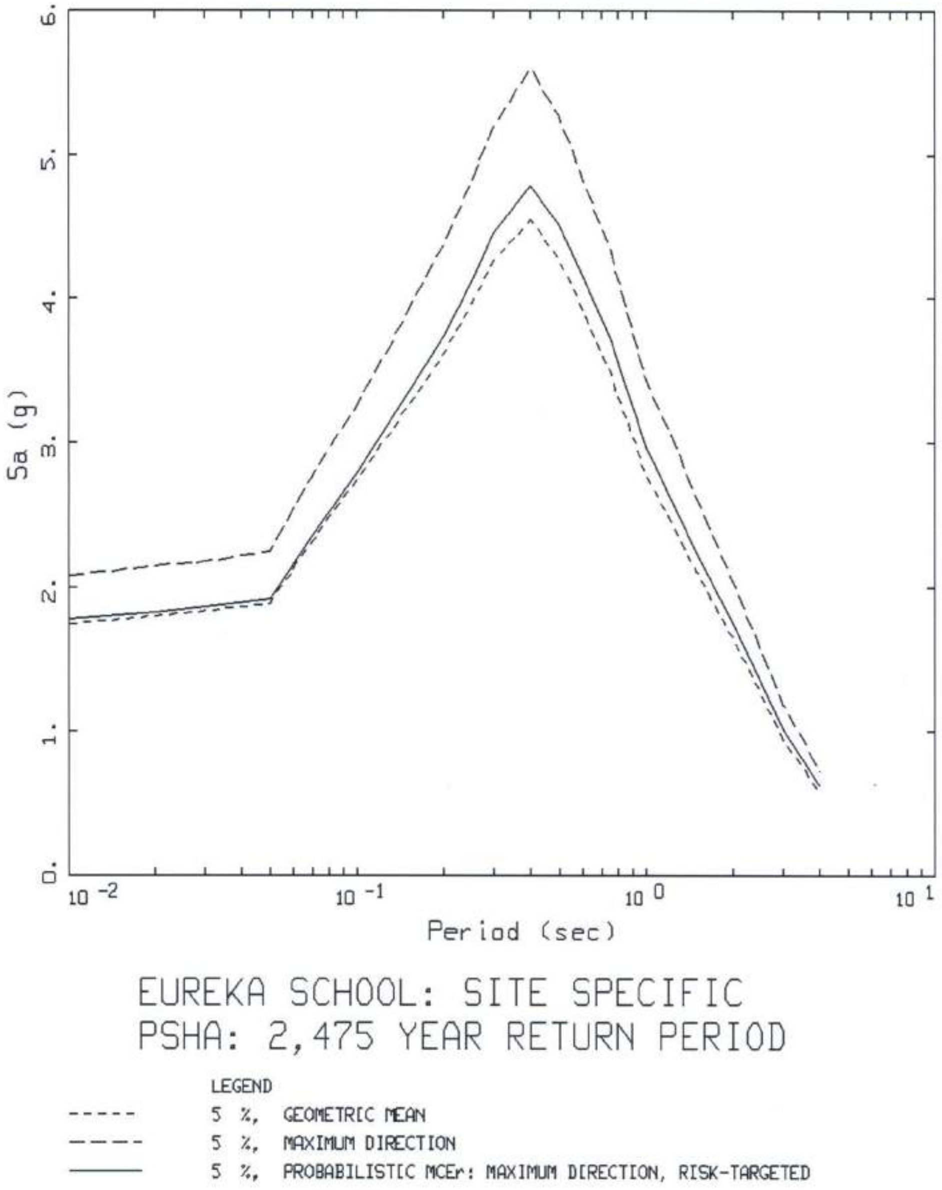


Figure 14. Development of site-specific probabilistic MCE_R: crustal, intraslab and Cascadia subduction zone sources; envelop V_S (30m).

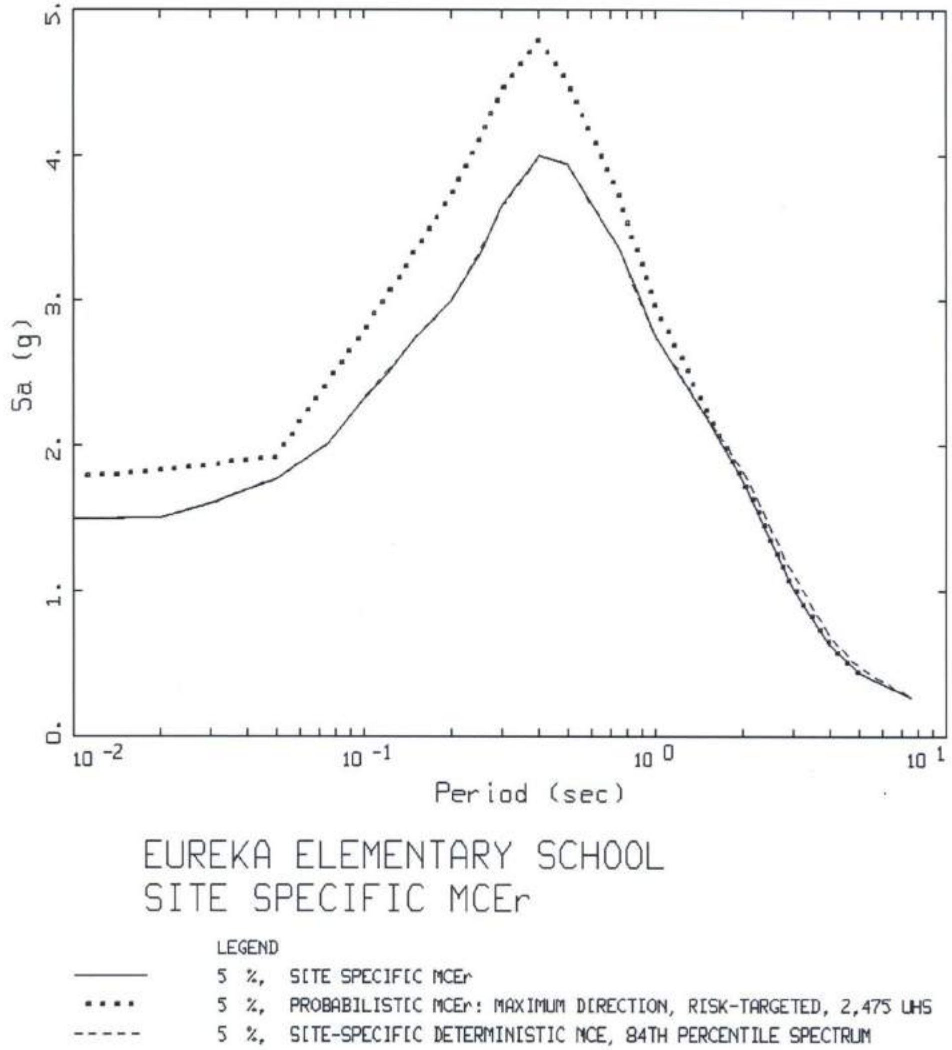


Figure 15. Development of site-specific MCE_R

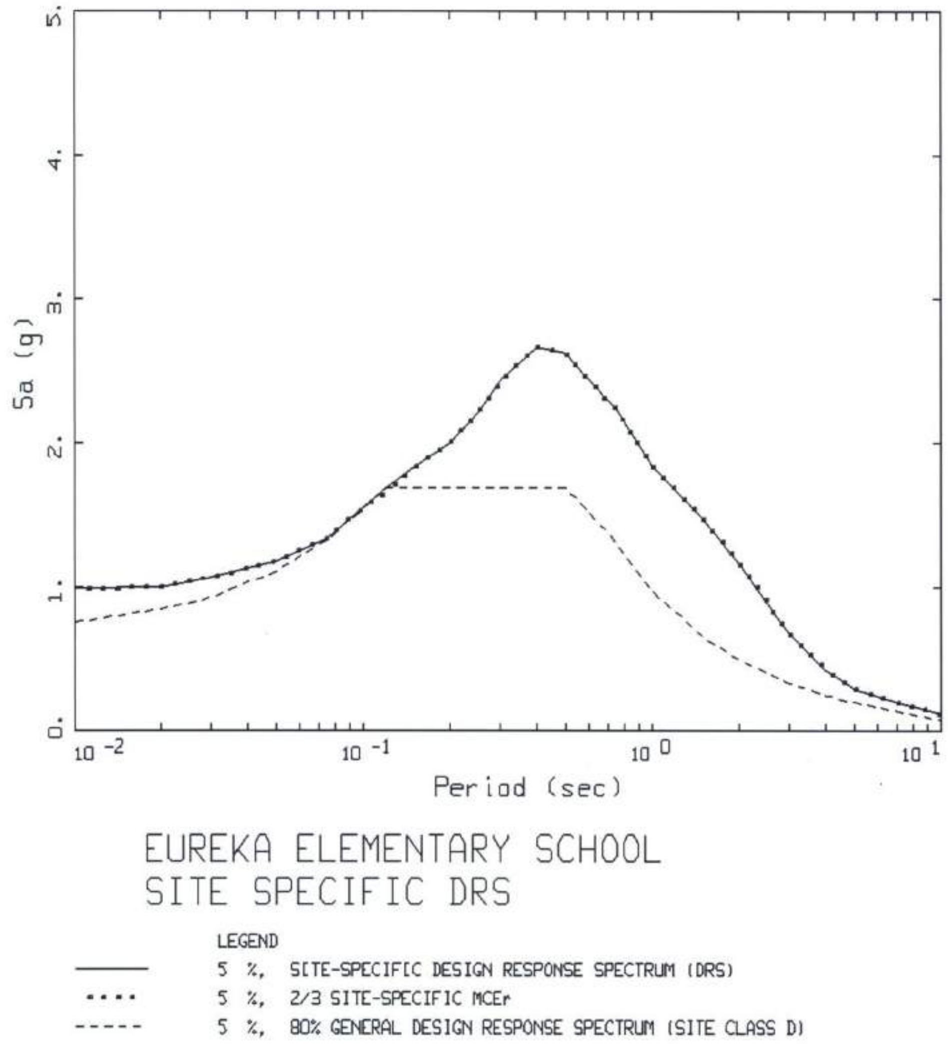


Figure 16. Development of site-specific design response spectrum (DRS).

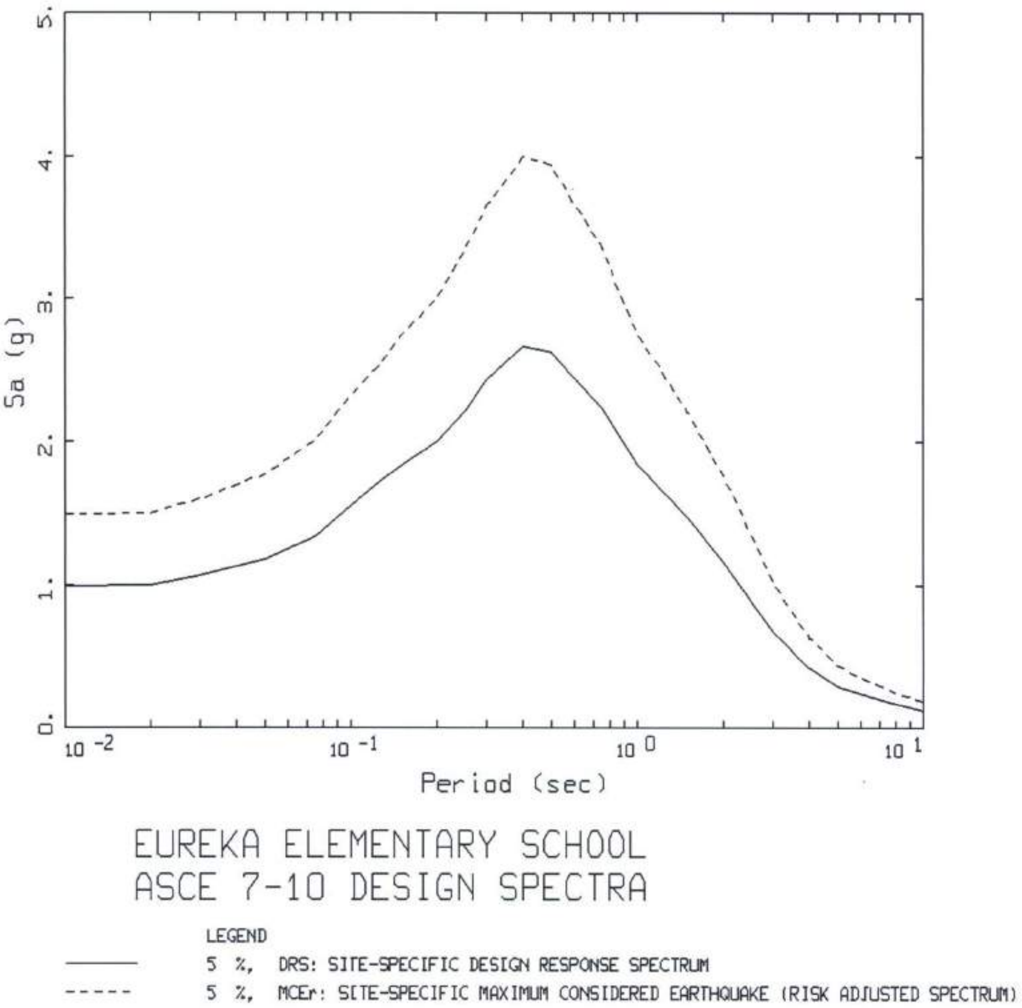


Figure 17. ASCE 7-10 design response spectra.

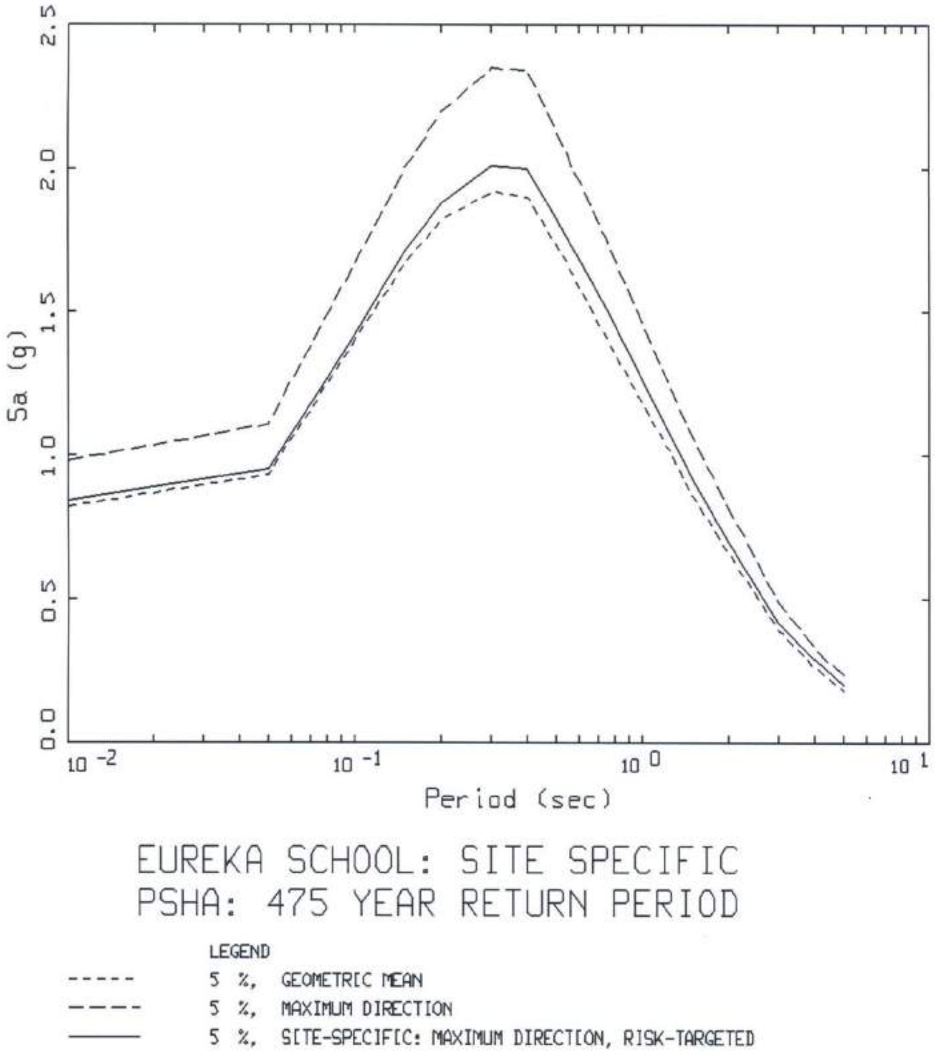


Figure 18. Development of site-specific probabilistic UHS for 475 year return period: crustal, intraslab and Cascadia subduction zone sources; envelop V_S (30m).

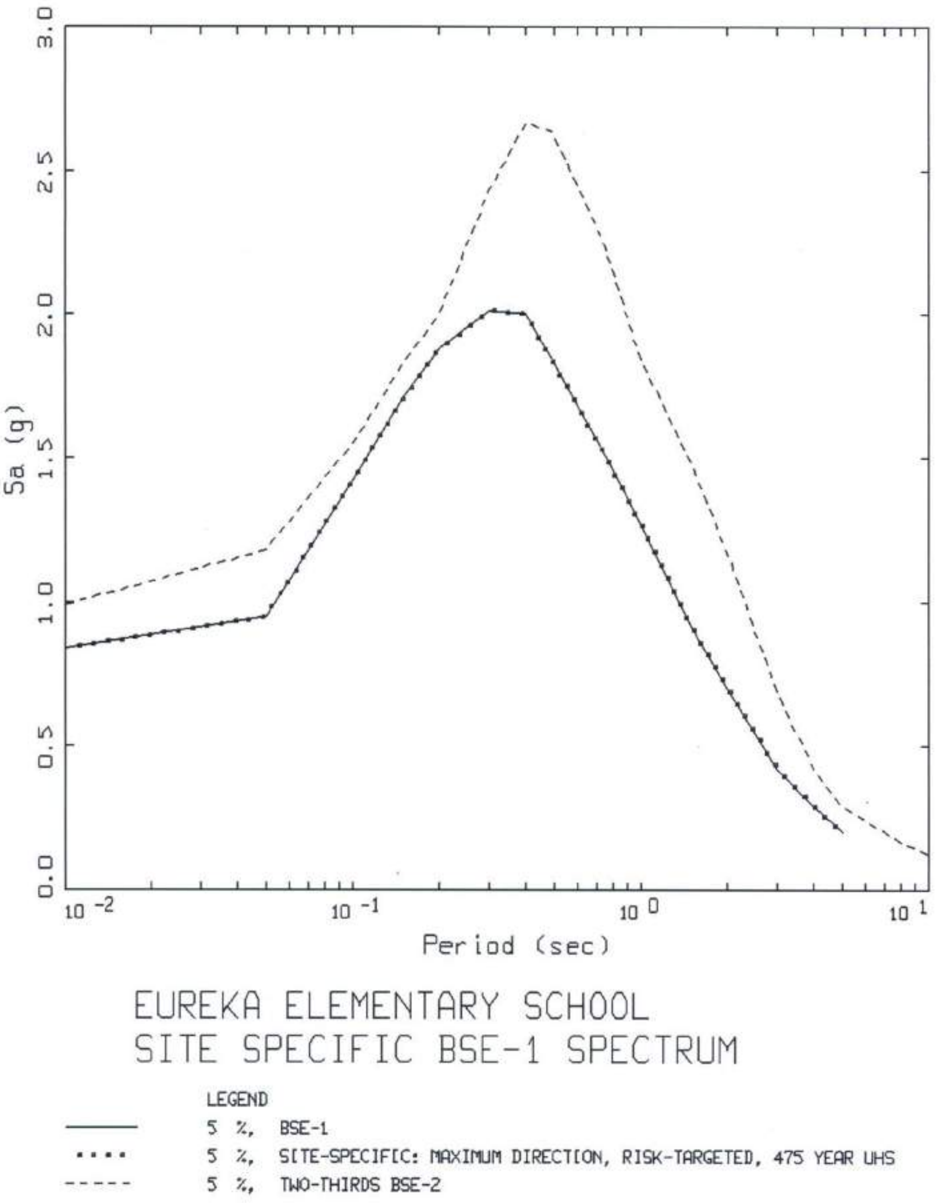


Figure 19. Development of site-specific BSE-1 spectrum.

Liquefaction Analysis Results

4

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LIQUEFACTION ANALYSIS REPORT

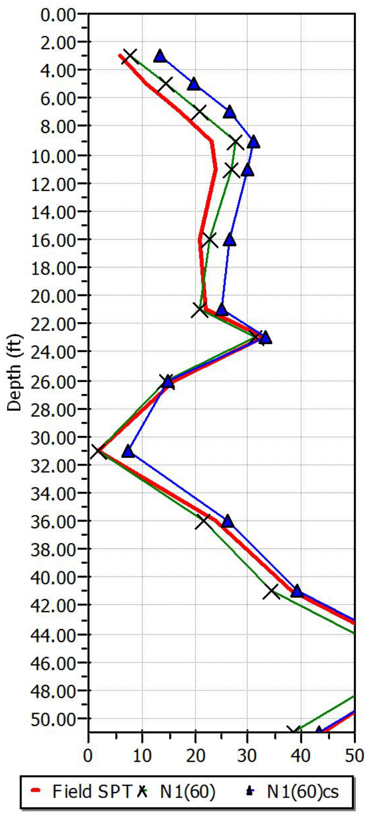
Project title : Eureka High School - Ag Building

Project subtitle : B-1 (2018)

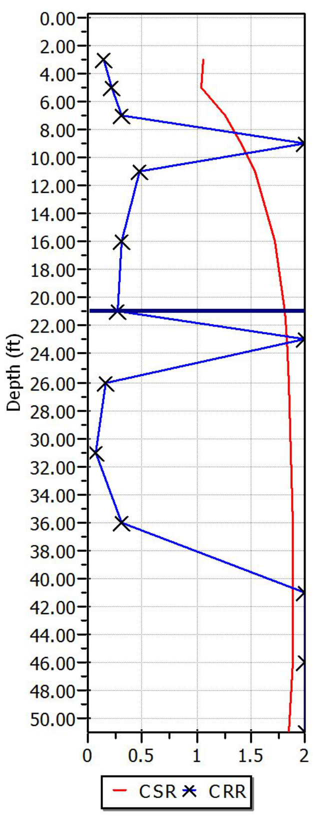
Input parameters and analysis data

In-situ data type:	Standard Penetration Test	Depth to water table:	21.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.50
Analysis method:	NCEER 1998	Peak ground accelaration:	1.25 g
Fines correction method:	Boulanger & Idriss	User defined F.S.:	1.30

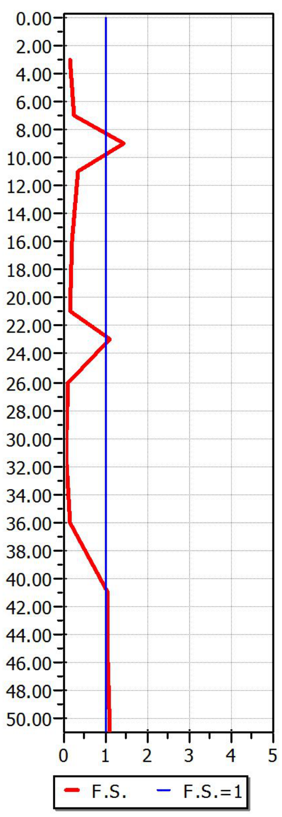
SPT data graph



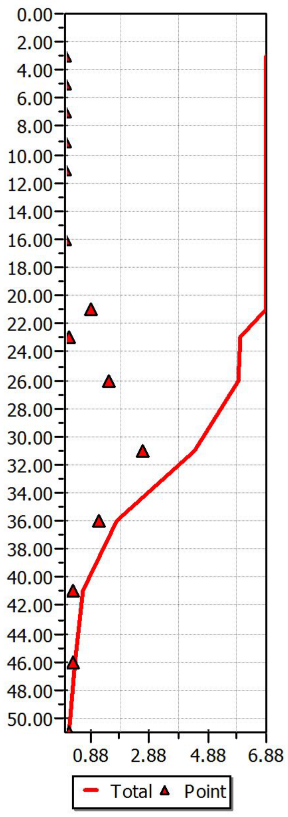
Shear stress ratio



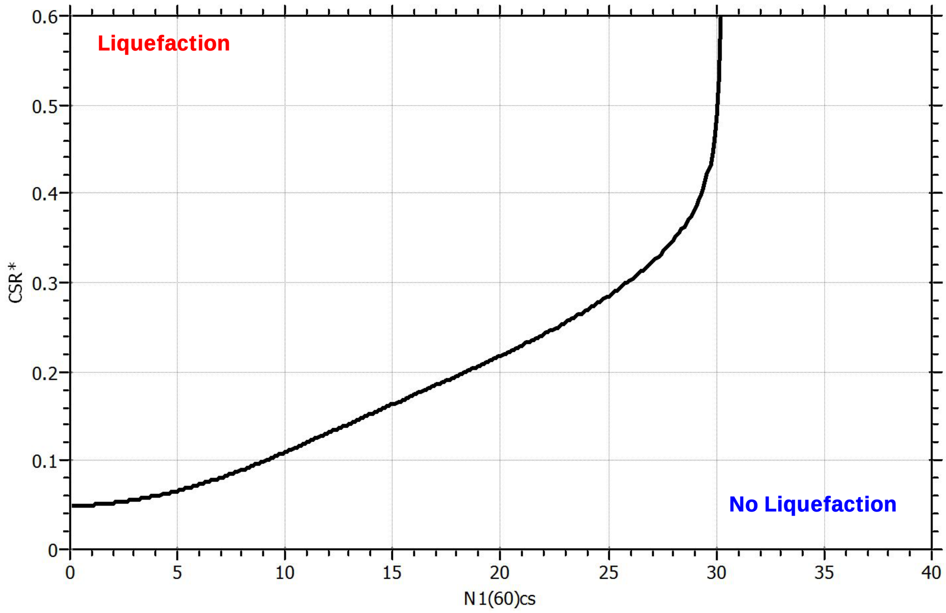
Factor of safety



Settlements (in)



$M_w=7^{1/2}$, sigma'=1 atm base curve



:: Field input data ::

Point ID	Depth (ft)	Field N_{SPT} (blows/feet)	Unit weight (pcf)	Fines content (%)
1	3.00	6.00	98.00	30.00
2	5.00	11.00	105.00	30.00
3	7.00	17.00	106.00	30.00
4	9.00	23.00	106.00	15.00
5	11.00	24.00	105.00	15.00
6	16.00	21.00	109.00	17.00
7	21.00	22.00	109.00	17.00
8	23.00	33.00	109.00	12.00
9	26.00	16.00	109.00	8.00
10	31.00	2.00	109.00	80.00
11	36.00	24.00	109.00	22.00
12	41.00	38.00	109.00	22.00
13	46.00	64.00	109.00	22.00
14	51.00	44.00	109.00	22.00

Depth : Depth from free surface, at which SPT was performed (ft)
Field SPT : SPT blows measured at field (blows/feet)
Unit weight : Bulk unit weight of soil at test depth (pcf)
Fines content : Percentage of fines in soil (%)

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Point ID	Depth (ft)	Sigma (tsf)	u (tsf)	Sigma' (tsf)	r_d	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*
1	3.00	0.15	0.00	0.15	1.00	0.81	1.00	0.81	1.00	0.81
2	5.00	0.25	0.00	0.25	1.00	0.81	1.00	0.81	1.00	0.81
3	7.00	0.36	0.06	0.30	0.99	0.97	1.00	0.97	1.00	0.97
4	9.00	0.46	0.12	0.34	0.98	1.09	1.00	1.09	1.00	1.09
5	11.00	0.57	0.19	0.38	0.98	1.19	1.00	1.18	1.00	1.18
6	16.00	0.84	0.34	0.50	0.96	1.32	1.00	1.32	1.00	1.32
7	21.00	1.11	0.50	0.61	0.94	1.39	1.00	1.39	1.00	1.39
8	23.00	1.22	0.56	0.66	0.94	1.41	1.00	1.41	0.99	1.42
9	26.00	1.39	0.66	0.73	0.92	1.43	1.00	1.43	0.99	1.43
10	31.00	1.66	0.81	0.85	0.90	1.44	1.00	1.44	1.00	1.44
11	36.00	1.93	0.97	0.96	0.88	1.44	1.00	1.44	0.99	1.46
12	41.00	2.20	1.12	1.08	0.86	1.43	1.00	1.43	0.98	1.45
13	46.00	2.48	1.28	1.20	0.84	1.41	1.00	1.41	0.97	1.45
14	51.00	2.75	1.44	1.31	0.81	1.39	1.00	1.39	0.97	1.42

Depth : Depth from free surface, at which SPT was performed (ft)
Sigma : Total overburden pressure at test point, during earthquake (tsf)
u : Water pressure at test point, during earthquake (tsf)
Sigma' : Effective overburden pressure, during earthquake (tsf)
r_d : Nonlinear shear mass factor
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$ CSR adjusted for M=7.5
K_{sigma} Effective overburden stress factor
CSR^* CSR fully adjusted

:: Cyclic Resistance Ratio calculation $CRR_{7.5}$ ::

Point ID	Field SPT	C_n	C_e	C_b	C_r	C_s	$N_{1(60)}$	DeltaN	$N_{1(60)cs}$	$CRR_{7.5}$
1	6.00	1.70	0.90	1.15	0.75	1.00	7.92	5.36	13.28	0.14
2	11.00	1.70	0.90	1.15	0.75	1.00	14.52	5.36	19.88	0.22
3	17.00	1.59	0.90	1.15	0.75	1.00	20.95	5.36	26.31	0.31
4	23.00	1.36	0.90	1.15	0.85	1.00	27.55	3.26	30.81	2.00
5	24.00	1.27	0.90	1.15	0.85	1.00	26.72	3.26	29.98	0.48
6	21.00	1.09	0.90	1.15	0.95	1.00	22.60	3.85	26.45	0.31

:: Cyclic Resistance Ratio calculation $CRR_{7.5}$ ::

Point ID	Field SPT	C_n	C_e	C_b	C_r	C_s	$N_{1(60)}$	DeltaN	$N_{1(60)cs}$	$CRR_{7.5}$
7	22.00	0.97	0.90	1.15	0.95	1.00	21.04	3.85	24.88	0.28
8	33.00	0.96	0.90	1.15	0.95	1.00	31.25	2.07	33.32	2.00
9	16.00	0.92	0.90	1.15	0.95	1.00	14.51	0.36	14.88	0.16
10	2.00	0.84	0.90	1.15	1.00	1.00	1.74	5.54	7.28	0.08
11	24.00	0.87	0.90	1.15	1.00	1.00	21.51	4.77	26.28	0.31
12	38.00	0.87	0.90	1.15	1.00	1.00	34.25	4.77	39.01	2.00
13	64.00	0.91	0.90	1.15	1.00	1.00	60.53	4.77	65.29	2.00
14	44.00	0.84	0.90	1.15	1.00	1.00	38.44	4.77	43.21	2.00

C_n :	Overburden corretion factor
C_e :	Energy correction factor
C_b :	Borehole diameter correction factor
C_r :	Rod length correction factor
C_s :	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT}
DeltaN :	Addition to corrected N_{SPT} value due to the presence of fines
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines
$CRR_{7.5)}$:	Cyclic resistance ratio for M=7.5

:: Settlements calculation for saturated sands ::

Point ID	$N_{1(60)}$	N_1	FS_L	e_v (%)	Settle. (in)
1	13.28	11.07	0.14	3.31	0.00
2	19.88	16.57	0.21	2.52	0.00
3	26.31	21.93	0.24	1.98	0.00
4	30.81	25.67	1.41	0.08	0.00
5	29.98	24.98	0.31	1.70	0.00
6	26.45	22.04	0.18	1.97	0.00
7	24.88	20.74	0.16	2.08	0.88
8	33.32	27.77	1.09	0.38	0.11
9	14.88	12.40	0.09	3.08	1.48
10	7.28	6.07	0.04	4.38	2.63
11	26.28	21.90	0.16	1.98	1.19
12	39.01	32.51	1.06	0.42	0.25
13	65.29	54.41	1.06	0.41	0.25
14	43.21	36.01	1.08	0.34	0.10

Total settlement : 6.88

$N_{1,(60)}$:	Stress normalized and corrected SPT blow count
N_1:	Japanese equivalent corrected value
FS_L:	Calculated factor of safety
e_V:	Post-liquefaction volumentric strain (%)
Settle.:	Calculated settlement (in)

:: Liquefaction potential according to Iwasaki ::

Point ID	F	w_z	I_L
1	0.86	9.54	7.53
2	0.79	9.24	4.47
3	0.76	8.93	4.12
4	0.00	8.63	0.00
5	0.69	8.32	3.49
6	0.82	7.56	9.43
7	0.84	6.80	8.74
8	0.00	6.49	0.00
9	0.91	6.04	5.04
10	0.96	5.28	7.68
11	0.84	4.51	5.76

:: Liquefaction potential according to Iwasaki ::

Point ID	F	w_z	I_L
12	0.00	3.75	0.00
13	0.00	2.99	0.00
14	0.00	2.23	0.00

Overall potential I_L : 56.27

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

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LIQUEFACTION ANALYSIS REPORT

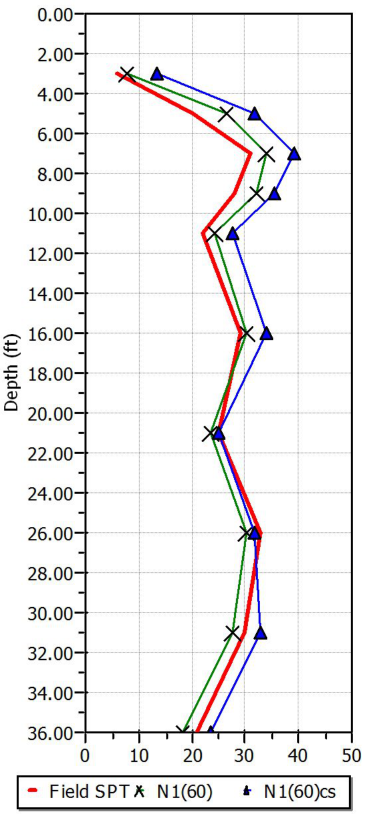
Project title : Eureka High School - Ag Building

Project subtitle : B-2 (2018)

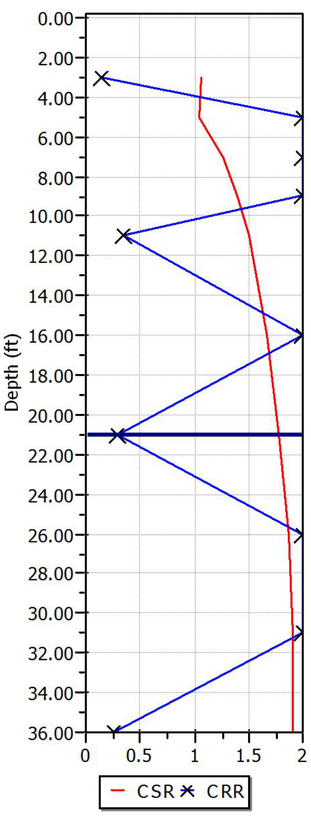
Input parameters and analysis data

In-situ data type:	Standard Penetration Test	Depth to water table:	21.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.50
Analysis method:	NCEER 1998	Peak ground accelaration:	1.25 g
Fines correction method:	Boulanger & Idriss	User defined F.S.:	1.30

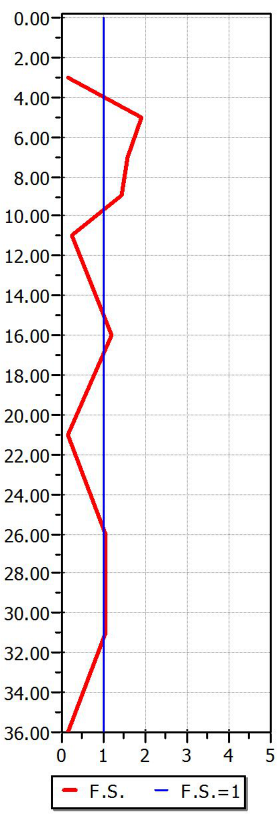
SPT data graph



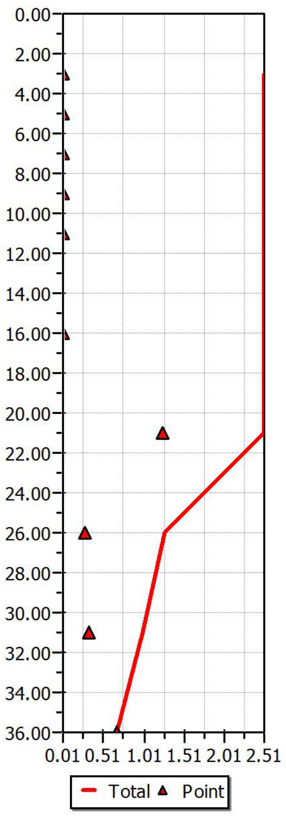
Shear stress ratio



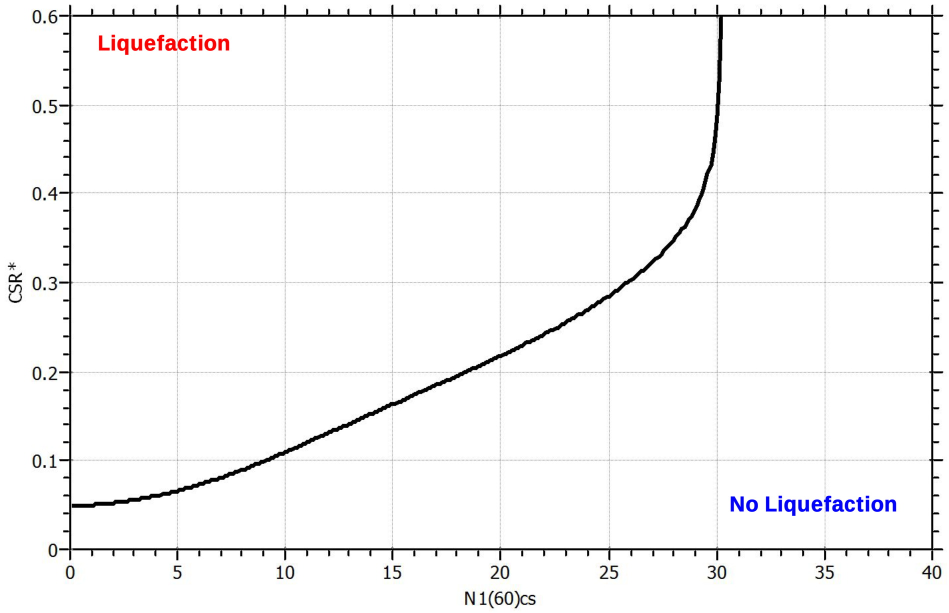
Factor of safety



Settlements (in)



M_w=$7^{1/2}$, sigma'=1 atm base curve



:: Field input data ::

Point ID	Depth (ft)	Field N_{SPT} (blows/feet)	Unit weight (pcf)	Fines content (%)
1	3.00	6.00	101.00	45.00
2	5.00	20.00	110.00	30.00
3	7.00	31.00	112.00	30.00
4	9.00	28.00	112.00	15.00
5	11.00	22.00	112.00	15.00
6	16.00	29.00	112.00	16.00
7	21.00	25.00	112.00	11.00
8	26.00	33.00	112.00	11.00
9	31.00	30.00	112.00	25.00
10	36.00	21.00	112.00	25.00

Depth :	Depth from free surface, at which SPT was performed (ft)
Field SPT :	SPT blows measured at field (blows/feet)
Unit weight :	Bulk unit weight of soil at test depth (pcf)
Fines content :	Percentage of fines in soil (%)

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Point ID	Depth (ft)	Sigma (tsf)	u (tsf)	Sigma' (tsf)	r_d	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*
1	3.00	0.15	0.00	0.15	1.00	0.81	1.00	0.81	1.00	0.81
2	5.00	0.26	0.00	0.26	1.00	0.81	1.00	0.81	1.00	0.81
3	7.00	0.37	0.06	0.31	0.99	0.97	1.00	0.97	1.00	0.97
4	9.00	0.49	0.12	0.36	0.98	1.08	1.00	1.08	1.00	1.08
5	11.00	0.60	0.19	0.41	0.98	1.16	1.00	1.16	1.00	1.16
6	16.00	0.88	0.34	0.53	0.96	1.28	1.00	1.28	1.00	1.28
7	21.00	1.16	0.50	0.66	0.94	1.35	1.00	1.35	0.98	1.37
8	26.00	1.44	0.66	0.78	0.92	1.38	1.00	1.38	0.96	1.44
9	31.00	1.72	0.81	0.91	0.90	1.39	1.00	1.39	0.95	1.47
10	36.00	2.00	0.97	1.03	0.88	1.39	1.00	1.39	0.95	1.46

Depth :	Depth from free surface, at which SPT was performed (ft)
Sigma :	Total overburden pressure at test point, during earthquake (tsf)
u :	Water pressure at test point, during earthquake (tsf)
Sigma' :	Effective overburden pressure, during earthquake (tsf)
r_d :	Nonlinear shear mass factor
CSR :	Cyclic Stress Ratio
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$	CSR adjusted for M=7.5
K_{sigma}	Effective overburden stress factor
CSR^*	CSR fully adjusted

:: Cyclic Resistance Ratio calculation $CRR_{7.5}$ ::

Point ID	Field SPT	C_n	C_e	C_b	C_r	C_s	$N_{1(60)}$	DeltaN	$N_{1(60)cs}$	$CRR_{7.5}$
1	6.00	1.70	0.90	1.15	0.75	1.00	7.92	5.61	13.52	0.15
2	20.00	1.70	0.90	1.15	0.75	1.00	26.39	5.36	31.76	2.00
3	31.00	1.41	0.90	1.15	0.75	1.00	33.97	5.36	39.33	2.00
4	28.00	1.31	0.90	1.15	0.85	1.00	32.16	3.26	35.42	2.00
5	22.00	1.25	0.90	1.15	0.85	1.00	24.28	3.26	27.54	0.34
6	29.00	1.06	0.90	1.15	0.95	1.00	30.36	3.57	33.93	2.00
7	25.00	0.96	0.90	1.15	0.95	1.00	23.56	1.61	25.17	0.29
8	33.00	0.93	0.90	1.15	0.95	1.00	30.13	1.61	31.74	2.00
9	30.00	0.89	0.90	1.15	1.00	1.00	27.75	5.07	32.82	2.00
10	21.00	0.84	0.90	1.15	1.00	1.00	18.28	5.07	23.35	0.26

:: Cyclic Resistance Ratio calculation $CRR_{7.5}$ ::

Point ID	Field SPT	C_n	C_e	C_b	C_r	C_s	$N_{1(60)}$	DeltaN	$N_{1(60)cs}$	$CRR_{7.5}$

C_n : Overburden corretion factor
C_e : Energy correction factor
C_b : Borehole diameter correction factor
C_r : Rod length correction factor
C_s : Liner correction factor
$N_{1(60)}$: Corrected N_{SPT}
DeltaN : Addition to corrected N_{SPT} value due to the presence of fines
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines
$CRR_{7.5)}$: Cyclic resistance ratio for M=7.5

:: Settlements calculation for saturated sands ::

Point ID	$N_{1(60)}$	N_1	FS_L	e_v (%)	Settle. (in)
1	13.52	11.27	0.14	3.28	0.00
2	31.76	26.46	1.90	0.01	0.00
3	39.33	32.78	1.59	0.03	0.00
4	35.42	29.52	1.43	0.06	0.00
5	27.54	22.95	0.22	1.88	0.00
6	33.93	28.28	1.20	0.18	0.00
7	25.17	20.97	0.16	2.06	1.24
8	31.74	26.45	1.07	0.47	0.28
9	32.82	27.35	1.04	0.55	0.33
10	23.35	19.46	0.14	2.21	0.66

Total settlement : 2.51

$N_{1,(60)}$: Stress normalized and corrected SPT blow count
N_1: Japanese equivalent corrected value
FS_L: Calculated factor of safety
e_v: Post-liquefaction volumentric strain (%)
Settle.: Calculated settlement (in)

:: Liquefaction potential according to Iwasaki ::

Point ID	F	w_z	I_L
1	0.86	9.54	7.51
2	0.00	9.24	0.00
3	0.00	8.93	0.00
4	0.00	8.63	0.00
5	0.78	8.32	3.94
6	0.00	7.56	0.00
7	0.84	6.80	8.69
8	0.00	6.04	0.00
9	0.00	5.28	0.00
10	0.86	4.51	5.94

Overall potential I_L : 26.07

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

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