Geologic Hazard and Geotechnical Investigation Report

Proposed Improvements to Eureka High School's Cooper Gulch Athletic Facilities, Albee Stadium and Grounds North of Del Norte Street, Eureka, California

Prepared for:

Eureka City Schools District

December 2021

020010.100

Phone: (707) 441-8855 **Email:** info@shn-engr.com
Web: shn-engr.com • 812 W. Wabash St., Eureka, CA 95501-2138

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

Reference: 020010.100

December 7, 2021

Paul Ziegler
Eureka City Schools District
2100 J Street
Eureka, CA 95501

Subject: Geologic Hazard and Geotechnical Investigation and Report, Proposed Improvements to Eureka High School's Cooper Gulch Athletic Facilities, Albee Stadium and Grounds North of Del Norte Street, Eureka, California

Dear Paul Ziegler:

This report presents the results of a geotechnical investigation conducted by SHN for the proposed improvements to Eureka High School's athletic facilities in Cooper Gulch. The improvements include significant upgrades within Albee Stadium, as well as to the grounds north of Del Norte Street. The primary purpose of this investigation was to assess site subsurface conditions and to develop geotechnical recommendations in support of the design and construction of the proposed project. Our investigation included: a) field exploration and laboratory testing program; and b) an engineering analysis to develop geotechnical recommendations, including grading, foundation, and retaining wall recommendations for the planned construction. 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 November 2019.

We appreciate this opportunity to work with you on this project. If you have any questions regarding this report, please call our office.

Respectfully,

SHN

Gary D. Simpson, CEG
Sr. Engineering Geologist
Geosciences Principal

John H. Dailey, PE, GE
Senior Geotechnical Engineer

GDS:JHD:ame
Enclosure: Geotechnical Investigation

Geologic Hazard and Geotechnical Investigation Report

Proposed Improvements to Eureka High School's Cooper Gulch Athletic Facilities, Albee Stadium and Grounds North of Del Norte Street, Eureka, California

Prepared for:

Eureka City Schools District

12/7/21

Gary D. Simpson, CEG

Prepared by:

812 W. Wabash Street
Eureka, CA 95501-2138
(707) 441-8855

12/7/21

John H. Dailey, PE, GE

December 2021

QA/QC: GDS___
Reference: 020010.100

Table of Contents

Appendices

List of Illustrations

Figures **Follows Page**

Tables **Page**

Abbreviations and Acronyms

Units of Measure

bpf	blows per foot
g	acceleration of gravity
km	kilometers
lbs/ft	pounds per foot
m	meter
mm	millimeters
ohms-cm	ohms-centimeter
p	magnitude of the footing load (lbs/ft)
pcf	pounds per cubic foot
pci	pounds per cubic inch
ppm	parts per million
psf	pounds per square foot
psi	pounds per square inch
R^4	the radius from the location on the wall where Δp is, measured to the footing load on the surface
x	centerline distance from the footing load to the wall
z	depth below surface
Δp_h	the lateral stress on the wall at depth z
µm	micrometers

Additional Terms

ABS	acrylonitrile butadiene styrene
ACI	American Concrete Institute
ADA	Americans with Disabilities Act
ASCE	American Society of Civil Engineers
ASTM	ASTM-International
B-#	boring number
Caltrans	California Department of Transportation
CBC	California Building Code
CBSC	California Building Standards Commission
CDMG	California Department of Conservation, Division of Mines and Geology
CMP	corrugated metal pipe
CRR	cyclic resistance ratio
CSR	cyclic stress ratio
CSZ	Cascadia subduction zone
DL	design load
EHS	Eureka High School
F_{PGA}	Site amplification factor at PGA
H:V	horizontal to vertical
ID	internal diameter
k_v	modulus of subgrade reaction
MCE	maximum considered earthquake
MCE_R	Maximum Considered Earthquake Spectral Acceleration
MCS	modified California sampler
M_W	magnitude
NR	no reference
OD	outside diameter
OSHA	Occupational Safety and Health Administration
OSHPD	California Office of Statewide Health Planning and Development
PGA	MCE_G peak ground acceleration
PGA_M	Site modified peak ground acceleration
PTI	Post-Tensioning Institute
PVC	polyvinyl chloride
SDC	Seismic Design Category
SDR	standard dimension ratio
SEAOC	Structural Engineers Association of California
SPT	standard penetration test
T.I.	traffic indices
USGS	United States Geological Survey

Introduction

This geologic hazard and geotechnical investigation report presents the results of a field and laboratory investigation conducted by SHN for design of improvements to Eureka High School's Cooper Gulch athletic facilities, including Albee Stadium and the fields north of Del Norte Street (refer to Project Location Map, Figure 1 and Site Map, Figure 2). Latitude and longitude of the project site are 40.789037°N and -124.155442°W, respectively. The report presents the results of the field exploration and subsequent geotechnical analyses upon which our recommendations are based. The purpose of this work is to provide the project's design team with findings, conclusions, and recommendations regarding the geologic setting and geotechnical engineering criteria to support the design and construction of the proposed development.

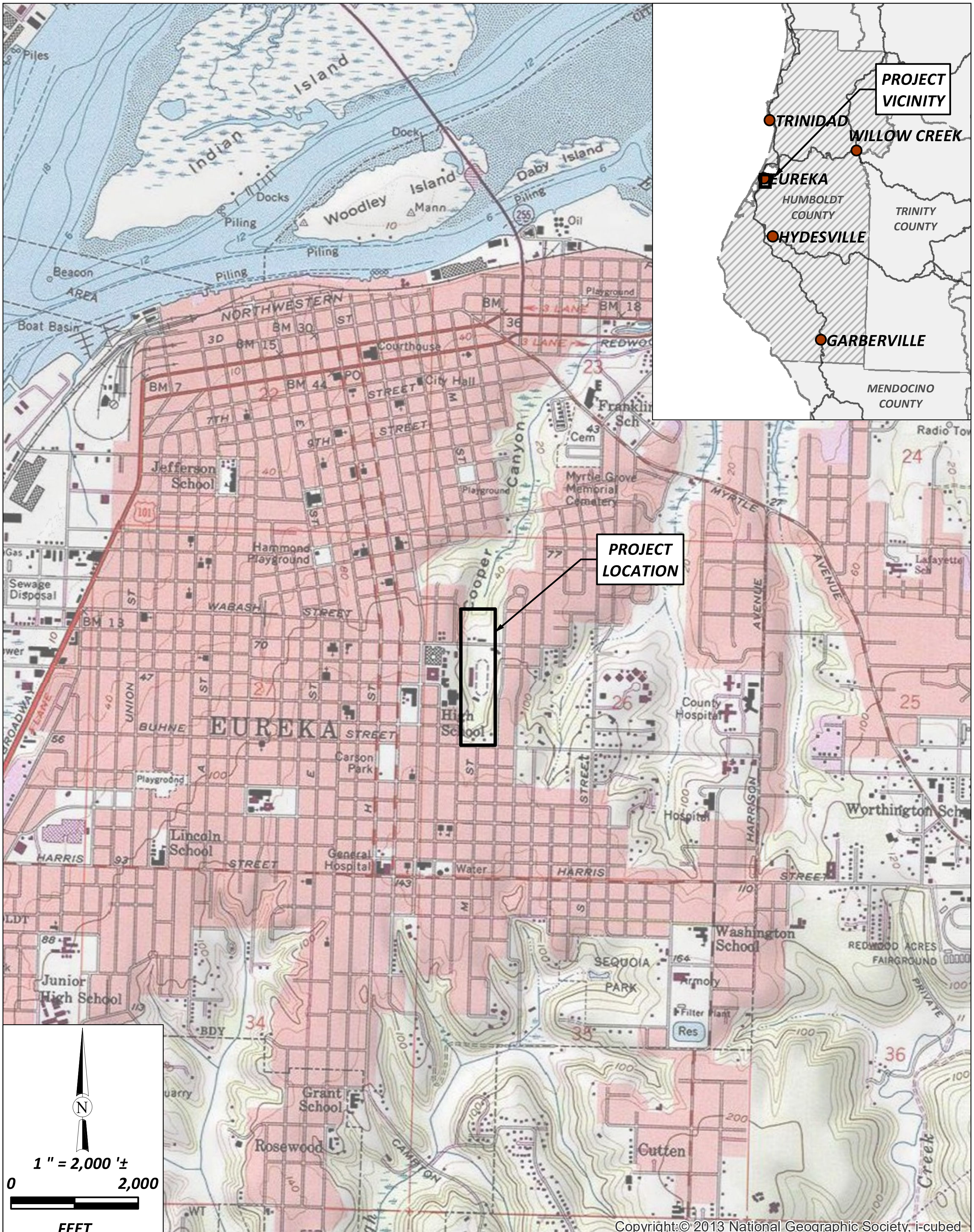
The report is intended to satisfy the requirements of the 2019 California Building Code (CBC; CBSC, 2019), as well as county standards, while maintaining the professional standard of care for this type of work. The report content and format follow the *California Geological Survey's Note 48: Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings*, dated November 2019. Three previous Note 48-level reports have been prepared for the Eureka High School (EHS) campus, most recently in 2018 (SHN, 2004, 2016, and 2018). The previous investigations evaluated structures proposed in the main campus area on the adjacent Eureka terrace upland directly west of the improvements described herein; this is the first geohazard report for improvements within Cooper Gulch. Although the scope of the project is relatively modest, Note 48-level reporting is required due to the inclusion of specific design elements (retaining walls adjacent to the proposed structures, rehabilitation of an existing structure); this represents an expanded version of an earlier, more focused geotechnical investigation report.

Project Description

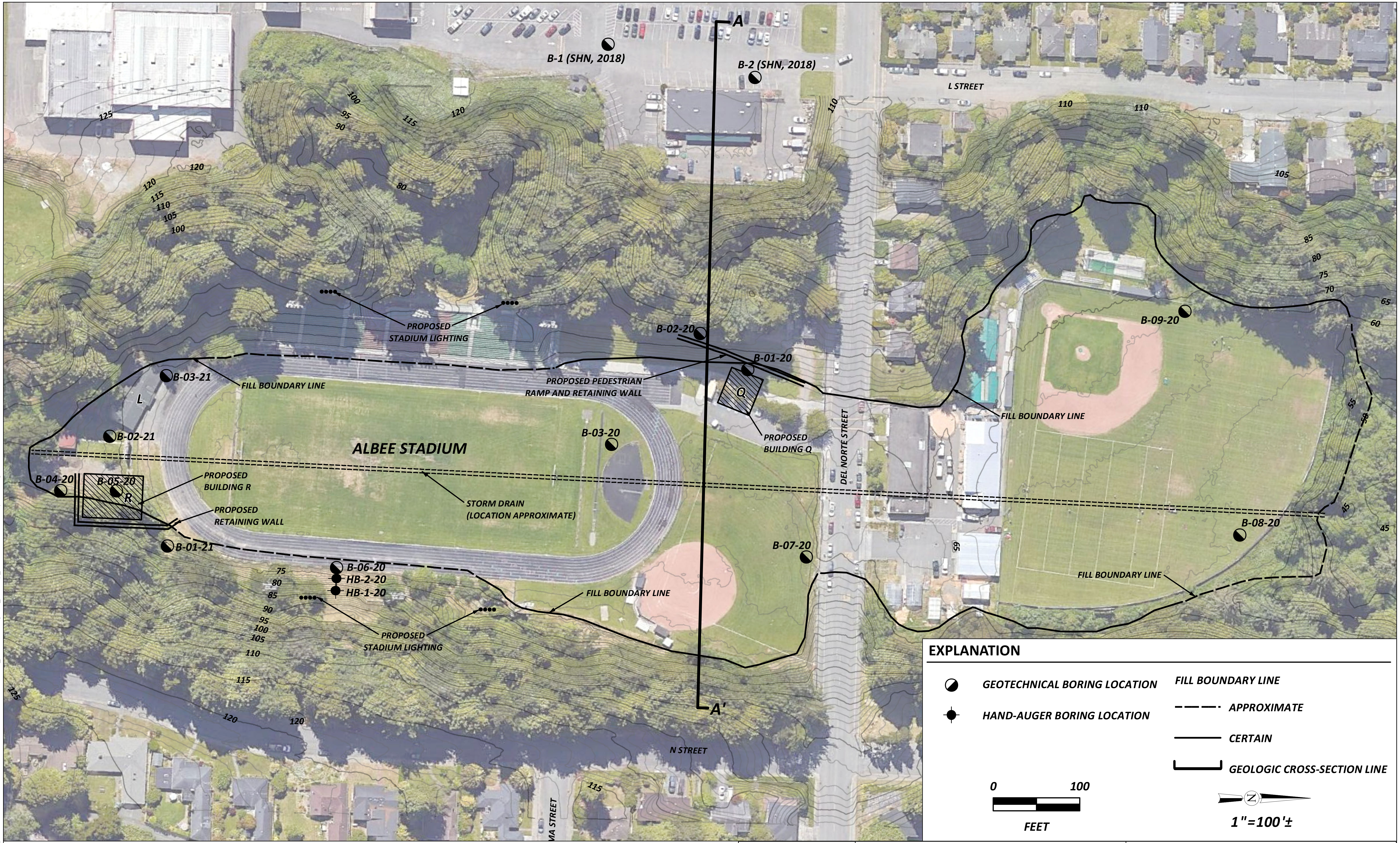
The proposed project consists of a variety of improvements in Albee Stadium and near the former Agricultural building north of Del Norte Street. Our understanding of the proposed improvements is based on the current project development plans, which we received from FF&J Architects on August 20, 2021, as well as subsequent conversations with Gary Mallory, the project architect. Relevant development elements from the current project plan set are shown on Figure 2. We are aware that these improvements are occurring in the context of planned repair of the existing, antiquated subdrainage system that extends beneath the entire project area. Our current understanding of the project improvements includes the following improvements:

- Two new buildings, one at the north end of Albee Stadium (Building Q), and one at the south end (Building R). The locations of the proposed buildings are shown on Figure 2. The northern building has a proposed footprint of 1,760 square feet and is planned for use as a concession stand/restroom; the southern building has a proposed footprint of 3,250 square feet and is planned for use as a team locker room. Both buildings will be single-story, wood-framed structures with slabs-on-grade supported on conventional spread footings. Each building is currently designed to be inset into modest slopes, with associated retaining walls.
- New Albee Stadium lighting will be located along the eastern side of the existing track and western side of the existing bleachers (currently planned for two poles on either side).
- Repairs/replacement will be made to the Albee Stadium track, which will serve as an emergency vehicle access and will need to accommodate loads from a fire truck.

Path: \\eureka\projects\2020\020010-EHSvoidInvest\100-field-investi\GIS\PROJ_MXD\2021\Figure1_ProjectLocationMap.mxd User Name: psundberg DATE: 9/7/21, 3:24PM



	Eureka City Schools Albee Stadium Geotechnical Investigation Eureka, California	Project Location Map 020010.100
September 2021	Figure1_ProjectLocationMap	Figure 1



NOTE: ALL PROPOSED IMPROVEMENTS ARE APPROXIMATELY LOCATED
IMAGE SOURCE: GOOGLE EARTH, DATED 6/3/2018
ELEVATION DATA SOURCE: CALIFORNIA COASTAL CONSERVANCY, 2009-2011

Eureka City Schools Albee Stadium Geotechnical Investigation Eureka, California	Site Map with Geotechnical Boring Locations and Proposed Improvements SHN 020010.100
September 2021	Figure2_SiteMap_rev2
	Figure 2

- New artificial turf football field in Albee Stadium
- Rehabilitation of Building L, an older "field house", developed in about 1940, including Americans with Disabilities Act (ADA) ramps at both ends of the structure
- New flatwork (parking) at the north end of the track in Albee Stadium and in the softball field (dugouts)
- Demolition of the defunct Agricultural Building north of Del Norte Street and construction of a new parking lot
- Flat work around the margins of the baseball field north of Del Norte Street

Purpose and Scope of Work

The primary purpose of the investigation was to evaluate the physical and engineering properties of the site subsurface materials to provide recommendations for site-specific geotechnical design criteria for the wide range of proposed improvements outlined above. As current SHN staff have been investigating geotechnical conditions at Eureka High School since at least 2004, this assessment builds on a significant amount of previous work, including three Note 48-level reports. This investigation, however, is the first comprehensive geotechnical investigation within the Cooper Gulch portion of the campus (there have been previous limited-scope investigations) and provides a valuable opportunity to understand geologic conditions over the larger campus area.

The scope of SHN's services consisted of geologic and geotechnical research, subsurface exploration at the project site, laboratory testing, engineering analysis, and preparation of this report with geotechnical recommendations to aid in project planning, design, and construction.

Specifically, the following information, recommendations, and design criteria are provided in this report:

- description of previous reporting;
- description of site terrain and local geology;
- description of soil and groundwater conditions interpreted based on our field exploration and laboratory testing;
- logs of geotechnical borings (Appendix 1) and the results of laboratory tests conducted for this investigation (Appendix 2);
- recommendations for new site improvements, including site and subgrade preparation, fill material, placement and compaction requirements, and foundation support;
- assessment of foundation load-bearing soil conditions (for buildings and light poles), including:
    - allowable bearing pressures or capacities (dead plus long-term live and seismic loads),
    - minimum foundation embedment,
    - estimates of settlement (total and differential), and
    - allowable lateral passive and sliding resistance characteristics;
- design and construction of asphalt pavement areas;
- and recommendations for materials testing and inspection during site preparation and grading, and installation of foundations.

Past Work

SHN has completed a series of geotechnical investigations for the Eureka High School campus, since at least 2004. These include the following Note 48-level reports:

- SHN, 2004: "Geologic Hazard and Geotechnical Report: Planned Woodshop Building, Concession Stand and Restroom, Gymnasium, and Single-story Classroom." This report included investigation in both the main campus area and within Cooper Gulch.
- SHN, 2016: "Geologic Hazard and Geotechnical Report: Jay Willard Gymnasium Seismic Retrofit or Replacement, Eureka High School, Eureka, California." This investigation occurred in the main campus area.
- SHN, 2018: "Geologic Hazard and Geotechnical Report: Proposed New Agricultural Building, Eureka High School, Eureka, California." This investigation occurred on the main campus, just west of Cooper Gulch, and was intended to replace a settlement-damaged building located within Cooper Gulch.

We are aware from previous investigations that settlement-related distress damaged the previous Agricultural building in Cooper Gulch, as the structure was built over uncontrolled fill materials without appropriate design considerations (reinforcement or deep foundation elements, for example). We are also aware of ongoing evaluation and design relative to improvements to the storm drain system beneath the project area. SHN (2018) documents a series of evaluations of erosion-related depressions that have formed beneath the EHS athletic facilities over the past several years. We interpret these settlement events to be associated with the failure of drainage laterals and subsequent erosion along the clogged or collapsed drain pipe (that is, sediment flushing to the main storm drain). These interpretations have been confirmed during in-line camera surveys over the past year that show significant degradation and flushing where laterals meet the main storm drain.

Field Investigation and Laboratory Testing

The geotechnical field exploration and laboratory testing programs performed for the investigation are summarized below. Detailed descriptions of the results from the field and laboratory programs are presented in Appendices 1 and 2, respectively.

Field Exploration Program

Subsurface investigation consisted of the advancement of 12 geotechnical machine borings and 3 shallow hand-auger borings. Nine geotechnical machine borings were completed between July 28, 2020, and July 30, 2020, and three more on July 12, 2021. The geotechnical machine borings were drilled and sampled to depths ranging from 11.5 feet to 51.5 feet. The approximate boring locations are as shown on the Site Map, Figure 2. Boring locations were selected based on existing access, and we note that not all areas were accessible due to the substantial conifer forest on the valley wall slopes and the existing structures.

An initial phase of nine borings were drilled by Taber Drilling based in West Sacramento, California, utilizing a track-mounted drill rig using both solid flight auger (above the water table) and mud rotary drilling methods (below the water table). These were geotechnical borings intended to provide comprehensive data and samples of site materials to the full depth of exploration. Three subsequent focused borings were drilled by Clear Heart Drilling of Santa Rosa, California, utilizing an all-terrain drill rig using solid flight augers. These were shallow borings intended to inform specific data gaps (laboratory sample collection for the retaining wall at Building R; determination of the depth and character of fill beneath Building L). Upon completion of drilling, the borings were backfilled with cement grout.

Relatively undisturbed soil samples were obtained by driving a 2.5-inch internal diameter (ID), 3.0-inch outside diameter (OD), Modified California Sampler (MCS) containing steel liners and a 1.4-inch ID, 2.0-inch OD standard penetration test (SPT) sampler without liners in accordance ASTM-International (ASTM) D1586 standards. The samplers were advanced using a 140-pound CME auto-hammer falling 30 inches per blow. The number of blows required to drive the samplers the last 12 inches of an 18-inch drive is provided on the boring logs as penetration resistance (blows per foot [bpf]). The penetration resistance values (bpf) recorded for SPT sampler drives and provided on the boring logs are actual penetration resistance (N-values) that are uncorrected for depth and the energy transfer ratio of the automatic hammers used. The penetration resistance values provided on boring logs for the MCS sampler drives are field blow counts and should not be construed as SPT N-values. Equivalent SPT N-values for the MCS sampler should be considered lower by a factor of approximately 0.6.

Visual classifications of the earth materials encountered were made in general accordance with the Manual-Visual Classification Method (ASTM D 2488). The final boring logs, presented in Appendix 1, were prepared based on the field logging, examination of samples in the laboratory, and the results of laboratory testing.

Laboratory Testing

Selected soil samples were tested in SHN's certified soils-testing laboratories in Eureka, California, to determine selected index properties and strength characteristics of the subsurface materials. Samples were tested for in-place moisture content, dry density, Atterberg limits, unconfined compressive and triaxial shear strength, and percent passing the #200 sieve. Samples were also collected for corrosion testing and an R-value test. The results of the laboratory tests are provided at the corresponding sample locations on the boring logs in Appendix 1 and are included as Appendix 2.

Site Conditions

The following sections describe the project site and current surface conditions, the geologic setting of the site, and subsurface soil and groundwater conditions encountered at the time of our field exploration.

Site Description

The project area is located within Cooper Gulch. Whereas the main EHS campus lies on an upland surface to the west of Cooper Gulch, the campus athletic facilities have been located in the adjacent gulch for many years. Historical photographs (see front and back report covers) indicate that Albee Stadium was developed early in the 20th century and has been used continuously since that time. The athletic facilities were developed within Cooper Gulch by filling the valley bottom. An existing 30-inch concrete storm drain runs the length of the project area, from south to north, with a series of drainage laterals that deliver runoff from various inlets throughout the improved areas. The storm drain is a few feet below grade at the south end of Albee Stadium and deepens to the north. The side slopes of the gulch have been, and continue to be, forested with a mixed stand of conifer (mostly second growth redwood).

Originally, the Cooper Gulch athletic facilities were located entirely south of Del Norte Street (Albee Stadium), but eventually the baseball field and other improvements north of Del Norte Street were developed. We understand from previous work and review of aerial photographs that the development of the area north of Del Norte Street was achieved by importing and placing large volumes of loose beach sand to create playing surfaces. The storm drain was extended at this time and reaches a depth of about 20 feet at its northern end.

The eastern slope of Cooper Gulch is a smooth continuous valley wall slope of moderate gradient through the entire project area. Although areas of substantial seepage are noted, there are no well-established watercourses on the eastern slope. The western slope of Cooper Gulch is characterized by a drainage canyon that occurs west of the stadium bleachers. The small watershed area associated with this stream suggests it is spring fed and likely receives substantial runoff from storm drains in the main campus area. The stream drains to an inlet that appears to lead to a drainage lateral extending beneath the bleachers to the main axial Cooper Gulch storm drain. Storm drain inlets on the campus apparently feed a small impoundment (pond) that is present along the west slope at the north end of Albee Stadium (just west of the proposed Building Q). An apparent spring-fed stream flows from the western valley wall slope adjacent to the baseball field north of Del Norte Street. This stream also feeds into an inlet that appears to extend to the main storm drain line.

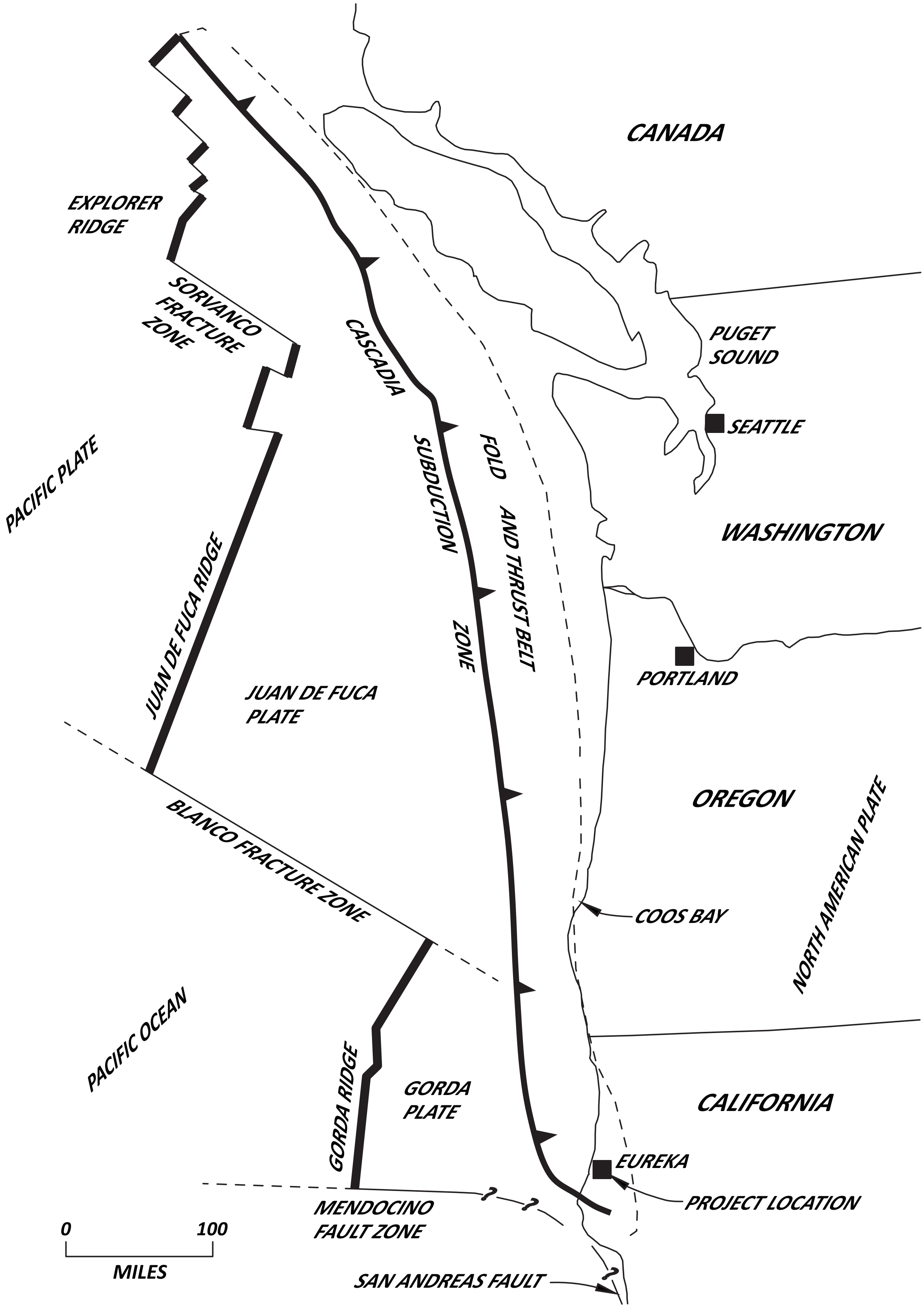
The existing athletic facilities in Albee Stadium include concrete bleachers along the western side of the stadium, a track and grass playing field (football, soccer), and a small field house at the south end of the stadium. A low (±5 feet high) retaining wall runs along the eastern side of the track. Stadium lighting consists of large reinforced concrete structures behind the bleachers and along the east valley wall slope. The eastern light structures, in particular, are associated with massive foundations consisting of intersecting wing walls (we assume these may be pile-supported). A softball field is present in the northeast corner of Albee Stadium.

North of Del Norte Street, the abandoned Agricultural building (to be demolished and replaced with a parking lot) and current woodshop building (supported on a deep foundation) occur adjacent to the existing baseball field ("Cloney Field"), which represents the northernmost campus development in Cooper Gulch. The outlet of the 1,500-foot-long storm drain beneath the project area occurs just north of the outfield of the baseball field.

The EHS campus is not associated with a "mapped geologic hazard zone" in either City of Eureka or Humboldt County general plans.

Geologic Setting

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



SHN	Eureka City Schools Albee Stadium Geotechnical Investigation Eureka, California	Regional Tectonic Setting SHN 020010.100
September 2021	Figure3_RegionalTectonicSetting	Figure 3

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.

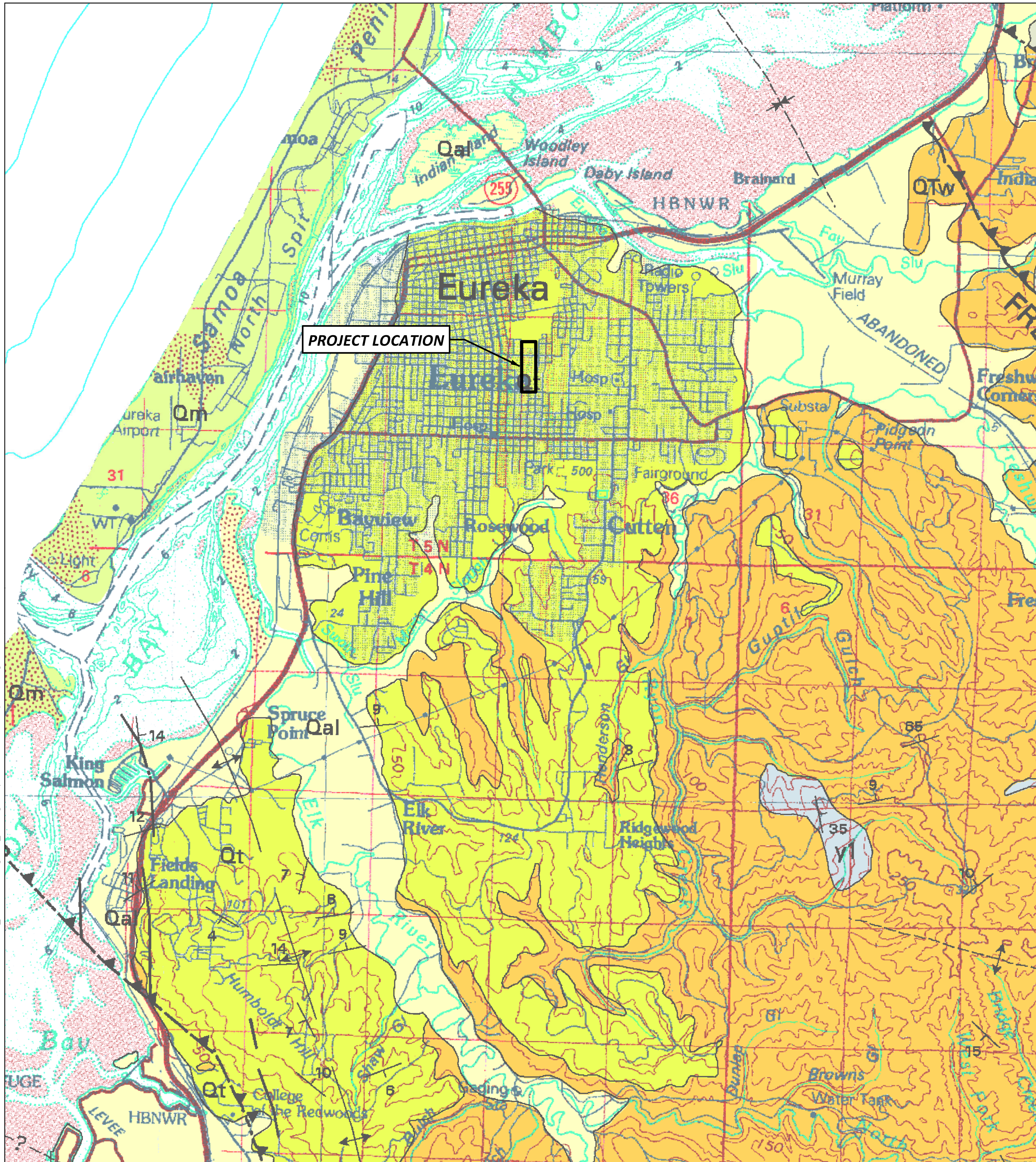
Regional geologic mapping of the Eureka area is included as Figure 4 (McLaughlin and others, 2000). Basement rock beneath Humboldt Bay and the City of Eureka is the Paleocene-Eocene Yager terrane, a part of the Coastal belt of the Franciscan Complex (Blake et al., 1985; Clarke, 1992). The Franciscan 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; it is likely several hundred feet beneath the site based on available deep exploratory well data (Woodward-Clyde Consultants, 1980). The nearest exposure of Yager terrane bedrock occurs about 3.5 miles southeast of Eureka High (Figure 4).

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), shown as unit QTw to the east of Ryan Creek on Figure 4. 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 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 (unit Qt on Figure 4). 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 into the Hookton Formation. 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 as much as 70 feet of predominantly silty sand covering the basal abrasion platform; these sediments are referred to as “marine terrace deposits” in this report.

The main (upper) campus at Eureka High is situated on a marine terrace surface loosely correlated with the “McKinleyville terrace”, a regionally mapped terrace that is reported by Carver and Burke (1992) to be associated with marine isotope stage 5b with an approximate age of 100,000 years. Albee Stadium



EXPLANATION

QUATERNARY AND TERTIARY OVERLAP DEPOSITS

Qal

Alluvial deposits (Holocene and late Pleistocene?)-Clay, silt, sand, gravel, and boulders, deposited in stream beds, alluvial fans, terraces, flood plains and ponds; and soils formed on these deposits. Includes largely Holocene deposits in modern stream channels and on flood plains

Qm

Undeformed marine shoreline and aolian deposits (Holocene and late Pleistocene)-Gravel and sand deposited in marine terraces, on benches, and on dunes along present shorelines. In northern Eureka quadrangle, near Arcata, includes older late Pleistocene dune sands (Carver and others, 1984)

Qt

Undifferentiated nonmarine terrace deposits (Holocene and Pleistocene)-Dissected and (or) uplifted gravel, sand, silt, and clay, deposited in fluvial settings. In western Eureka quadrangle (Sheet 1) unit includes minor shallow marine intertongues and warped and tilted beds of late Pleistocene Hookton and Rohnerville Formations of Ogle (1953), in addition to younger late Pleistocene and Holocene fluvial terrace units a few feet to a few tens of feet higher than normal modern high-water level

QTw

Marine and nonmarine overlap deposits (late Pleistocene to middle Miocene)-Thin-bedded to massive, weakly lithified siltstone, fine- to medium-grained sandstone, silty to diatomaceous mudstone and locally soft, scaly mudstone. Locally includes lenses of pebble to boulder conglomerate, carbonate concretions, abundant molluscan fossils, woody debris, and horizons of rhyolitic volcanic ash that are greater than 1 meter thick in some areas. Includes the Wildcat Group (Ogle, 1953), the Bear River beds (Haller, 1980), and related outlier Neogene deposits isolated along faults near Briceland, Garberville, Benbow, Piercy, Bridgeville and northeast of Weott. Unit also includes minor fault-bounded blocks along or near the coast between Bear River and the Mattole River that are incorporated into melange of the Coastal terrane; the Neogene Falor Formation northeast of Eureka (Manning and Ogle, 1950); and equivalent deposits in the offshore area deposited in shelf, slope, and slope basin settings. A few poorly exposed erosional remnants of shallow marine to brackish water strata mapped along high ridge crests overlying the Franciscan Complex in the 1:24,000 Zenia quadrangle are tentatively assigned to this unit. South of this map, unit correlates with valley-fill, perched gravel and shallow marine to nonmarine coal-bearing sedimentary rocks of Quaternary and Tertiary age in the Round Valley area of Covelo 1:100,000 quadrangle (Jayko and others, 1989)

Yager terrane (Eocene to Paleocene?)

Sedimnetary rocks of the Yager terrane (Eocene to Paleocene?)-Argillite and arkosic sandstone rhythmically interbedded, thin to medium bedded; massive to thickly bedded arkosic sandstone with minor interbeds of argillite; and minor lenses of polymict boulder to pebble conglomerate. Southwest of Garberville, unit highly folded, but locally may be penetratively sheared or broken. Argillite and interbedded fine-grained sandstone is commonly calcareous and may have abundant plant debris in places. Sandstone characteristically contains prominent detrital muscovite. Based on fossil dinoflagellates and on spores and pollen from carbonate concretions in argillite, age of terrane is late to middle Eocene. Locally the lower beds of the terrane may be as old as Paleocene (McLaughlin and others, 1994). The Yager terrane is divided into 3 subunits based principally on topographic expression in aerial photographs and outcrop data:

y1

Sheared and highly folded mudstone-Includes minor rhythmically interbedded sandstone, locally with lenses of conglomerate. Exhibits irregular topography lacking a well-incised system of sidehill drainages

y2

Highly folded broken mudstone, sandstone, and conglomeratic sandstone-Exhibits topography with sharp ridge-crests and well-incised sidehill drainages

y3

Highly folded, little-broken sandstone, conglomerate, and mudstone-Exhibits sharp-crested topography with a regular, well-incised system of sidehill drainages

cm2

Melange-Subequal amounts of metasandstone and meta-argillite. Exhibits irregular topography that lacks well incised sidehill drainages, but is less lumpy than unit cm1

MAP SYMBOLS

?**Contact**-Dashed where approximate, dotted where concealed, queried where uncertain

?**Fault**-Dashed where approximate, dotted where concealed, queried where uncertain

?**Thrust fault**-Barbs on upper plate, dashed where approximate, dotted where concealed, queried where uncertain

?**Trace of the San Andreas fault associated with 1906 earthquake rupture**-Dashed where approximate, queried where uncertain

Strike and dip of bedding:

10 20 **Inclined**-Ball denotes top of beds is known from sedimentary features

Vertical-Ball denotes top of beds is known from sedimentary features

Horizontal

10 20 **Overturned**-Ball denotes that top of beds is known from sedimentary features

20 **Approximate**-Based on photo interpretation or estimated dip in field

10 **Joint**-Strike and dip of joint

10 **Strike and dip of cleavage**-Ball denotes that top of flow is known from sedimentary or volcanic features

Shear foliation:

10 Inclined

Vertical

Folds:

Synclinal or synformal axis-showing direction of plunge

Anticlinal or antiformal axis-showing direction of plunge

Overturned syncline

Qls **Landslide**-Arrows indicate direction of movement

Melange Blocks:

Serpentinite

Chert

Blueschist

N

1"=1 MILE

0 1

MILES

Eureka City Schools Albee Stadium Geotechnical Ingestigation Eureka, California		Geologic Map (McLaughlin et al., 2000) SHN 019146.010
October 2021	Figure4_GeologicMap	Figure 4

Path: \\eureka\projects\2020\020010-EHSvoidInvest\100-field-invest\GIS\PROJ_MXD\2021\Figure4_GeologicMap.mxd User Name: psundberg DATE: 10/5/21, 12:26PM occurs within Cooper Gulch, an erosional stream valley that extends to the north toward the Eureka slough at the north edge of Eureka (Figure 1). We interpret that the gulch penetrates beneath the depth of the marine terrace sediment veneer, into the underlying Hookton Formation.

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 (CSZ; 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.

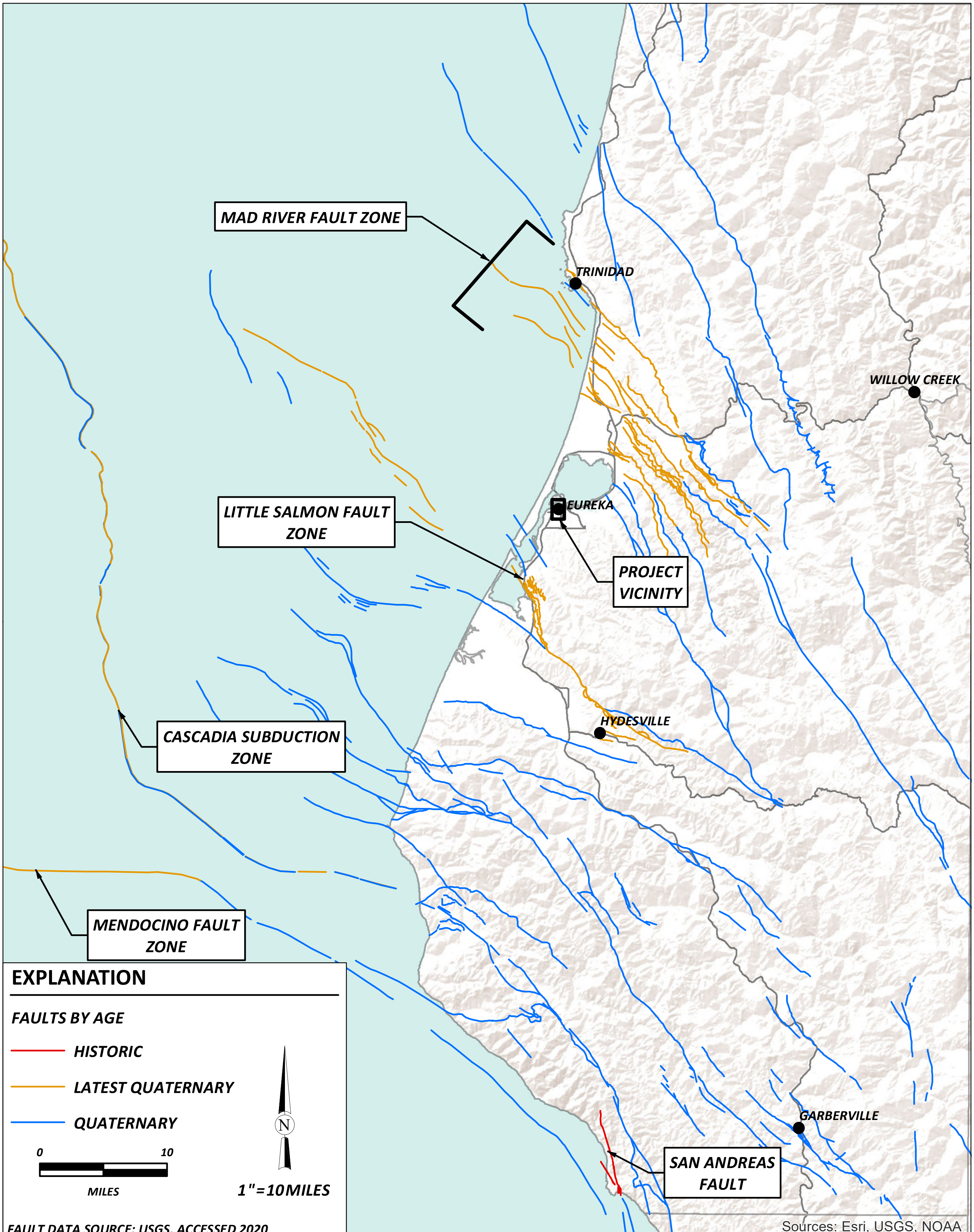
The Mendocino fault is the second most frequent source of earthquakes in the region. The fault represents the plate boundary between the Gorda and Pacific plates, and typically generates right lateral strike-slip displacement. Significant historic Mendocino fault earthquakes have ranged from M5 to M7.5. 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.

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



Path: \\eureka\projects\2020\020010-EHSvoidInvest\100-field-investi\GIS\PROJ_MXD\2021\Figure5_RegionalFaultMap.mxd User Name: psundberg DATE: 9/7/21, 4:12PM

SHN	Eureka City Schools Albee Stadium Geotechnical Investigation Eureka, California	Regional Fault Map SHN 020010.100
September 2021	Figure5_RegionalFaultMap	Figure 5

presents fault location and information data collected from the United States Quaternary Faults and Fold Database (USGS, current online edition).

Table 1. Summary of Nearby Active Faults Proposed Albee Stadium Improvements, Eureka High School, Eureka, California

Fault Name	Approximate Distance to Rupture Plane (kilometers)	Maximum Earthquake Magnitude (Mw)
Little Salmon	5.2	7.0
Mad River	13.0	7.1
Fickle Hill	10.0	6.9
McKinleyville	13.6	7.0
Table Bluff	10.0	7.0
Trinidad	20.2	7.3
Big Lagoon/Bald Mtn. Fault Zone	38.4	7.3
Cascadia Subduction Zone	19.5	8.3
Garberville/Briceland	58.1	6.9
Mendocino Fault Zone	63.6	7.4
San Andreas	63.2	7.6
Lake Mountain	80.4	6.7
Maacama	94.4	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 as many as 17 significant subduction zone events occurred along the CSZ in the last 6,700 years (Nelson and others, 2021). This rupture model suggests a recurrence interval over the past 3,000 years of 510-540 years, although some intervals are as much as 850 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 (Atwater and others, 2005). 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 and others, 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.

Historical Seismicity

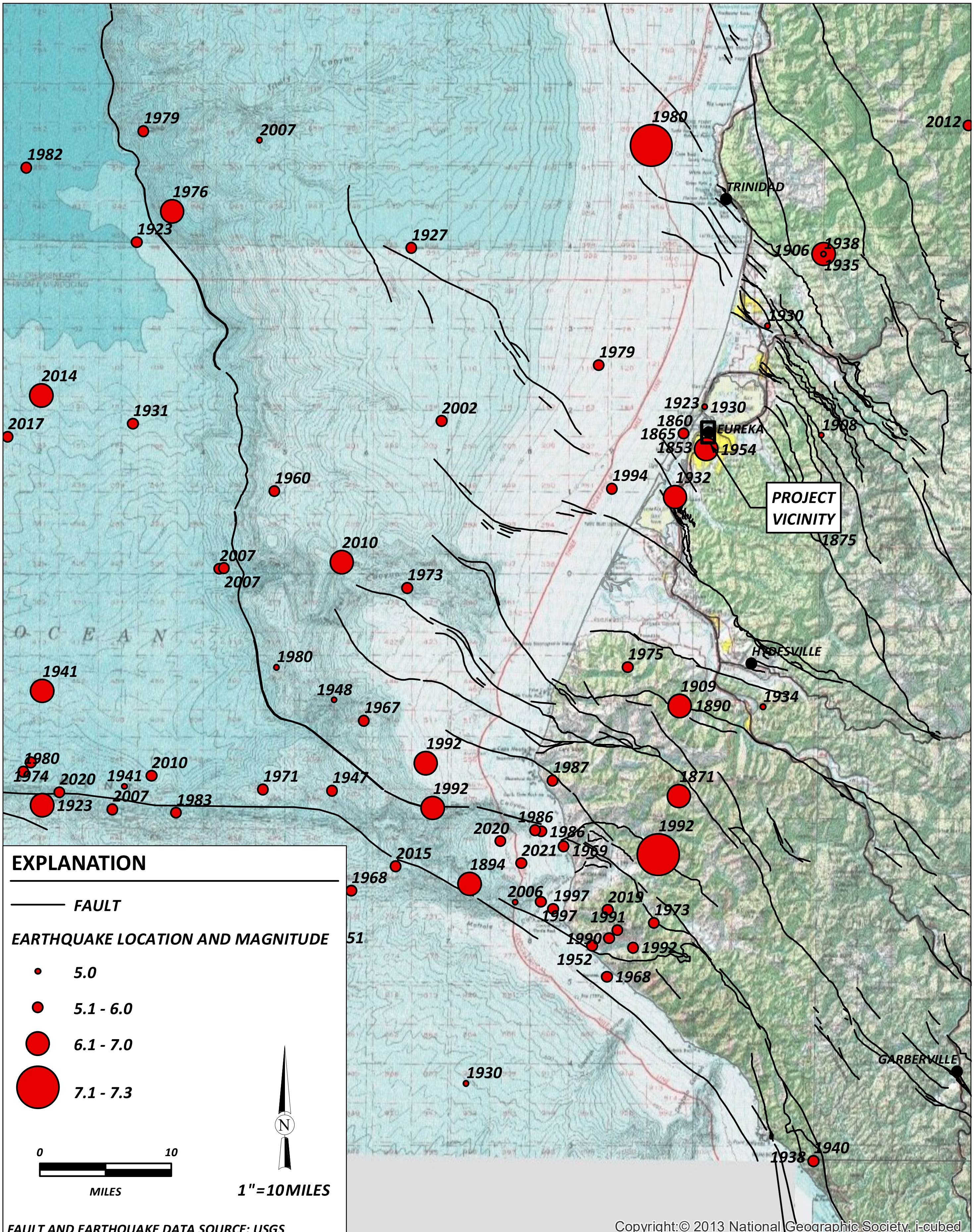
A search of historical earthquake records was performed using the United States Geologic Survey (USGS) Preliminary Determinations of Epicenters Catalog on the USGS website. Our search included historical data from 1918 to the present.

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 EHS campus; and a M7.1 in 1992 near Petrolia, approximately 40 km to the south of EHS. 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.

Earth Materials and Groundwater

Subsurface Materials

The subsurface investigation within Cooper Gulch for the subject investigation has allowed us to extend (deepen) the stratigraphic section relative to previous investigations on the upper, main EHS campus. Specifically, the upper part of the stratigraphic section observed in the current Cooper Gulch borings is



Path: \\eureka\projects\2020\020010-EHSvoidInvest\100-field-invest\GIS\PROJ_MXD\2021\Figure6_RegionalSeismicity.mxd User Name: psundberg DATE: 9/8/21, 2:04PM

SHN	Eureka City Schools Albee Stadium Geotechnical Investigation Eureka, California	Regional Seismicity SHN 020010.100
	September 2021 / Figure6_RegionalSeismicity	Figure 6

equivalent to the lowest part of the section observed in borings from the main campus. This additional stratigraphic section has allowed us to refine previous interpretations and increased our overall understanding of subgrade conditions within the project site.

Subsurface materials encountered in Albee Stadium and adjacent Cooper Gulch facilities include artificial fill, Holocene age valley fill sediments (alluvium and colluvium), late Pleistocene age marine terrace deposits, a distinct “pre-terrace” mud (newly defined in this investigation), and Hookton Formation sediments. A stratigraphic cross-section extending from the main campus surface, across Cooper Gulch, is included as Figure 7. Relative to our previous interpretations, we found the Hookton Formation to be deeper than previously interpreted. Descriptions of these materials and their distributions within the study area are as follows, from youngest to oldest.

Artificial Fill

Albee Stadium and adjacent facilities north of Del Norte Street were created by filling the bottom of Cooper Gulch. Based on historic photography, this appears to have been completed in phases, with the development of Albee Stadium pre-dating the development of facilities north of Del Norte Street. Albee Stadium fills were presumably placed early in the 20th century, as it is present in historical photos (see front and back report covers). The area north of Del Norte Street was filled sometime in the 1960s by the U.S. Army Corps of Engineers, apparently using beach/dune sand removed from local beaches.

During our investigation, artificial fill soils were limited to flat-lying areas within the project area; significant fill soils were not encountered on the valley wall slopes surrounding Cooper Gulch in the areas under consideration herein. The margin of the substantial Cooper Gulch fill soils is shown on Figure 2.

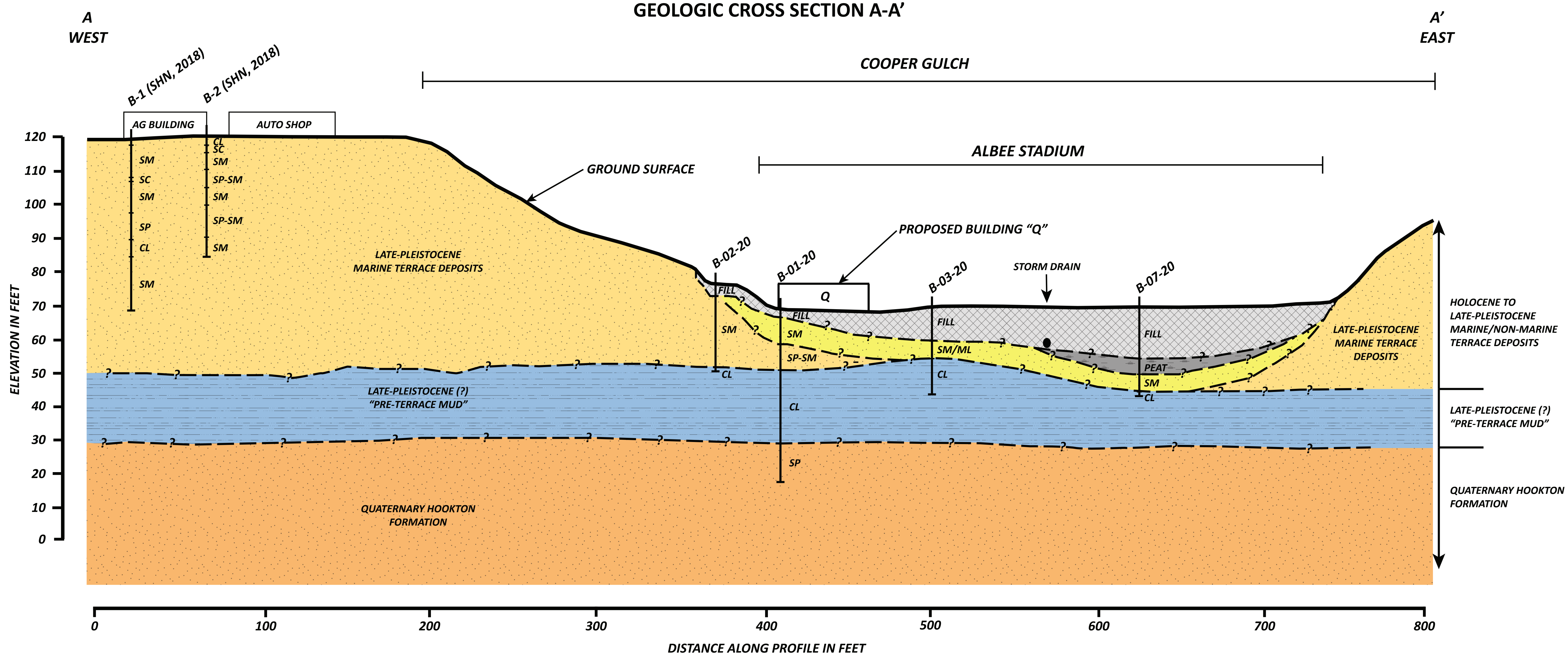
Artificial fill soils encountered during our investigation are relatively thin at the upstream (southern) end of Albee Stadium, and thicken toward the downstream (northern) end of the project area north of Del Norte Street, consistent with the natural gradient of the valley. At the southern end of Albee Stadium, where the storm drain inlet is visible just below grade, we observed fill thickness on the order of 6 or so feet (boring B-05-20). To the north, borings advanced near the storm drain alignment along the valley axis (B-03-20, B-07-20, B-08-20, from south to north), encountered fill thicknesses of 10, 16, and 21 feet, respectively. As would be expected, fill soils are thickest in the center of Cooper Gulch, and thin toward the valley margins.

Fill soils observed during the subsurface investigation are highly variable, consisting of mostly silty and clayey sands within Albee Stadium (these are the original early 20th century fills). North of Del Norte Street, near the downstream end of the storm drain (the existing baseball field), fill soils include large quantities of poorly graded sand, which we understand was imported to the site from local beaches. The fill soils throughout the project area were generally loose, with SPT blow counts typically less than 20. We assume that all the fill soils in the project area were placed without engineering control (that is, not placed with verified compaction). Minor amounts of wood and charcoal were noted within the fill soils.

Holocene Alluvium and Colluvium

Throughout the project area, it appears that the artificial fills were placed within Cooper Gulch on top of a thin, pre-existing organic-rich alluvial and colluvial layer that occurred on the valley floor. In our borings over the former valley floor, we observed this material as a distinct layer directly beneath the

EUREKA HIGH SCHOOL-ALBEE STADIUM
GEOLOGIC CROSS SECTION A-A'



EXPLANATION

- FILL
- PEAT
- HOLOCENE TO LATE-PLEISTOCENE VALLEY FILL SEDIMENTS (ALLUVIUM AND COLLUVIUM)
- LATE-PLEISTOCENE MARINE TERRACE DEPOSITS
- LATE-PLEISTOCENE (?) "PRE-TERRACE MUD"
- QUATERNARY HOOKTON FORMATION

SCALE:
HORIZONTAL: 1"=60'
VERTICAL: 1"=30'
V.E.=2X

0 1
INCH

NOTE: SURFACE ELEVATION INFORMATION FROM 2009-2011 CA COASTAL CONSERVANCY LIDAR: COASTAL CALIFORNIA

Path: \\eureka\projects\2020\020010-EHSvoidInvest\100-field-invest\GIS\FIGURES\Figure7_GeologicCrossSectionAA.ai User Name: psundberg DATE: 9/28/21, 1:15PM

SHN	Eureka City Schools Albee Stadium Geotechnical Investigation Eureka, California	Geologic Cross Section A-A' SHN 020010.100
September 2021	Figure7_GeologicCrossSectionAA	Figure 7

artificial fill, but above the underlying late Pleistocene age sediments. The material occurs as a wood-rich sand deposit or peat that increases in thickness from the southern end of the study area to the northern end. In boring B-4-20, we encountered about 5 feet of this material; at the northern end of the study area, in boring B-8-20, up to 9 feet was present (including a 4-foot-thick peat layer). In many areas, some of the material may have been displaced, as observed thicknesses were commonly on the order of 2 to 4 feet. The materials were generally loose (or soft where peats occur), with N-values less than 10 in all cases; poor sample recovery was common in these materials. We interpret this material to be Holocene in age, as it represents the organic-rich surface layer at the time Cooper Gulch was in-filled to create the stadium and adjacent facilities.

Quaternary Marine Terrace Deposits

Late Pleistocene age marine terrace deposits have been encountered beneath the entire main Eureka High campus surface, and they appear to extend into Cooper Gulch, where they are exposed along the valley side walls. Previous borings on the upland, main campus surface (see Figure 7) have encountered marine terrace deposits to the depth explored (about 50 feet). This is laterally equivalent to the upper part of the borings along the western valley wall that occurred during this investigation (B-2-20, for example), where we encountered similar marine terrace deposits. We take these materials to be laterally continuous and allows us to reinterpret the depth of the underlying Hookton Formation (which we had previously interpreted at a shallower depth).

The marine terrace deposits consist of predominantly silty sands of medium dense consistency. N-values ranging from 10 to 20 were observed in the limited exposures of this material in the boreholes.

The contact between the marine terrace deposits and underlying “pre-terrace” mud occurs near the base of the valley wall slope; therefore, terrace deposits were encountered on a limited basis in this investigation (although we note they have been the basis of all previous investigations at EHS). Terrace deposits are significant from a geotechnical standpoint, however, as they will support all or part of several of the improvements proposed herein (light standards, for example). Portions of Buildings Q and R, and all of their respective retaining walls will likely bear on terrace deposits.

Pre-terrace Mud

Throughout the project area, beneath the base of the marine terrace deposits, we encountered a distinct lean clay deposit. Based on geotechnical investigations throughout the Eureka area, we have come to recognize this previously unidentified deposit, as it occurs frequently at this same stratigraphic interval. For lack of a formal moniker, we refer to the deposit as the “pre-terrace mud” in this report.

The material occurs as a dark bluish-gray lean clay with sand, typically with low to medium plasticity. The massive (no apparent bedding) deposit is notably compact, and locally contains minor shell fragments. We interpret the material as an estuarine deposit. The pre-terrace mud is between 14 and 22 feet thick where we encountered both the top and bottom of the unit. The material has a medium stiff to stiff consistency, with N-values ranging from 10 to 28. Because of its stratigraphic position beneath late Pleistocene age marine terrace deposits, we infer the pre-terrace mud to be late Pleistocene in age as well. Through the project area, the buried alluvial/colluvial deposit within the bottom of Cooper Gulch sits directly on the pre-terrace mud.

Hookton Formation

The Hookton Formation occurs at depth beneath Cooper Gulch, beneath the "pre-terrace mud" described above. Throughout the project area, the Hookton Formation is a dark bluish-gray, very dense silty to clayey sand with occasional shells. The material is notable for the significant increase in penetration resistance at the upper contact, as N-values in the Hookton Formation are commonly in the 50 to 80 range. The upper contact of the Hookton Formation is present between 30 and 45 feet beneath the current ground surface within the project area (shallower in the south, deeper in the north).

Groundwater

Groundwater levels were observed to be highly variable across the project site at the time of our investigation (at the end of the dry season). We typically encountered the groundwater surface at depths between 10 and 20 feet, although borings near the valley margins encountered groundwater near the ground surface. One boring along the valley axis, near the main storm drain alignment (B-3-20), did not encounter groundwater in the upper 26.5 feet.

Groundwater levels in northern California are typically highly seasonally dependent and are expected to rise during the rainy season.

Spring sources are evident along the Cooper Gulch valley walls surrounding much of the project area, and wet areas are abundant along the margins of much of the site. Existing drainage facilities are extensive, some more effective than others (many older drainage facilities are present). Substantial flow was observed at the end of the dry season in many areas, and wet ground was observed in several areas relevant to proposed project improvements. Specifically, the areas of primary concern related to wet ground were observed adjacent to the west side of Building L, where a concrete access ramp is proposed, on the eastern slope of Albee Stadium, where stadium lighting is proposed, and in the southwestern corner of Albee Stadium, where a potential ramp is being considered.

Evaluation of Potential Geologic Hazards

Surface Fault Rupture

No known active fault crosses the Eureka High School campus. No evidence that a previously unrecognized active fault may be present at the site has been apparent in numerous geologic investigations, including the investigation described herein. The Eureka marine terrace, which occurs on either side of Cooper Gulch, 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 EHS 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 EHS campus. The risk of surface fault rupture at the EHS campus is negligible.

Seismic Ground Shaking

The project is located within a seismically active area and strong seismic ground shaking is the primary geologic risk at the site. Code-based seismic design parameters are provided below. It is our understanding, based on discussions with the project structural engineer, that site-specific ground motion assessment is not required for the current project. As described above, site-specific seismic evaluations have been completed previously at EHS, most recently in 2016, during the gymnasium investigation.

Soil Liquefaction Potential of Subsurface Materials

Liquefaction is a soil behavior phenomenon in which soil located below the groundwater table temporarily loses strength during and immediately after a seismic event because of strong earthquake ground motions. Recently deposited and geologically young Holocene age sediments consisting of relatively loose, saturated, non-cemented granular soil are most susceptible.

Liquefaction occurs as seismic shear stresses propagate through a saturated soil and distort the soil structure, causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure increases the porewater pressure between the soil grains. When porewater pressures increase to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid. As strength is lost, there is an increased risk of settlement. Liquefaction-induced settlement occurs as the elevated porewater pressures dissipate and the soil consolidates after the earthquake.

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) and from first person accounts in more recent earthquakes. Careful interpretation of the historical accounts indicates that liquefaction events in the area are entirely confined to recent alluvial sediments, primarily in the Eel River Valley, and late Holocene age bay margin sediments surrounding Humboldt Bay. There are no accounts of liquefaction in Pleistocene age sediments (marine terraces, for example) in the north coast region.

The potential for liquefaction to occur at the project site was calculated by comparing the cyclic shear stresses induced within the soil profile during an earthquake to the ability of the soils to resist these stresses. The cyclic shear stresses within the soil profile are estimated by computing the seismic response of horizontally layered soil deposits in response to the peak horizontal ground acceleration. The equivalent uniform stress profile is normalized by the vertical effective stress to develop a cyclic stress ratio (CSR) profile. The ability of the soils to resist these stresses, known as the cyclic resistance ratio (CRR), is based on soil strength as characterized by SPT N-values normalized for overburden pressures and corrected for such factors as fines content in accordance with the recommendations of Boulanger and Idriss (2014). The factor of safety against liquefaction is then defined as the ratio of CRR to CSR.

The peak horizontal ground acceleration and earthquake magnitude chosen to represent the design earthquake hazard level for our liquefaction analysis was determined based on the USGS interactive deaggregations web application (Dynamic: Conterminous U.S. 2014 (update) v4.2.0). To evaluate the potential for liquefaction, we have conservatively assumed groundwater depths ranging from 5 feet to 20 feet below the ground surface based on groundwater depths encountered during drilling and anticipated seasonal variations.

Our subsurface investigation and analysis indicate that the 5- to 9-foot-thick layer of loose to medium dense valley bottom soils (Holocene alluvium and colluvium, described above) and much of the overlying artificial fill section, where encountered below the water table, has a moderate to high likelihood of liquefying during the design earthquake. Due to their age, texture, and/or consistence, the late Pleistocene and older sediments (marine terrace sediments, "pre-terrace mud," Hookton Formation) are considered to have little if any liquefaction potential; they are too old. The effects of liquefaction, including seismically induced settlement and a reduction in bearing capacity due to soil strength loss, as well as mitigation strategies are discussed below in this report.

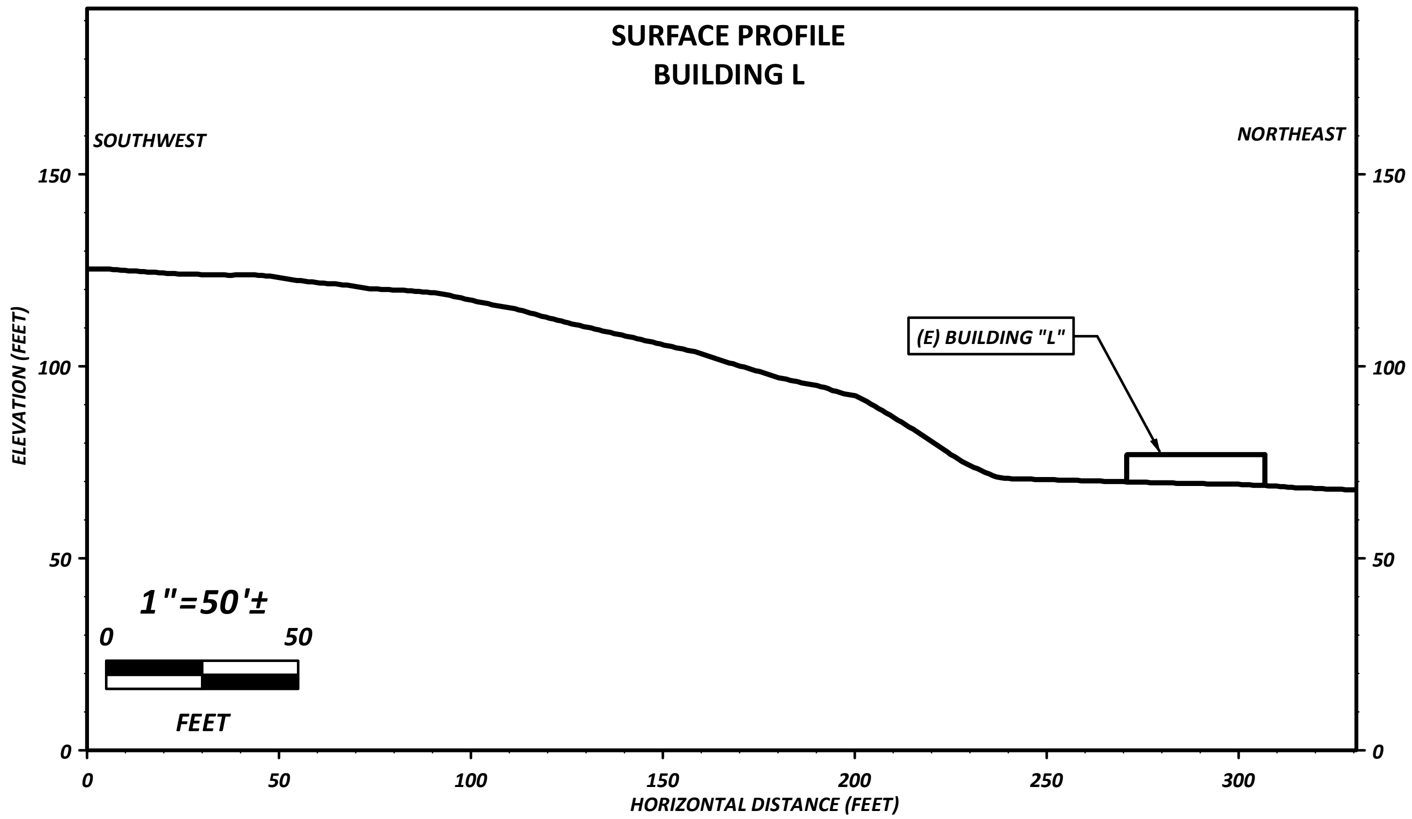
Slope Stability

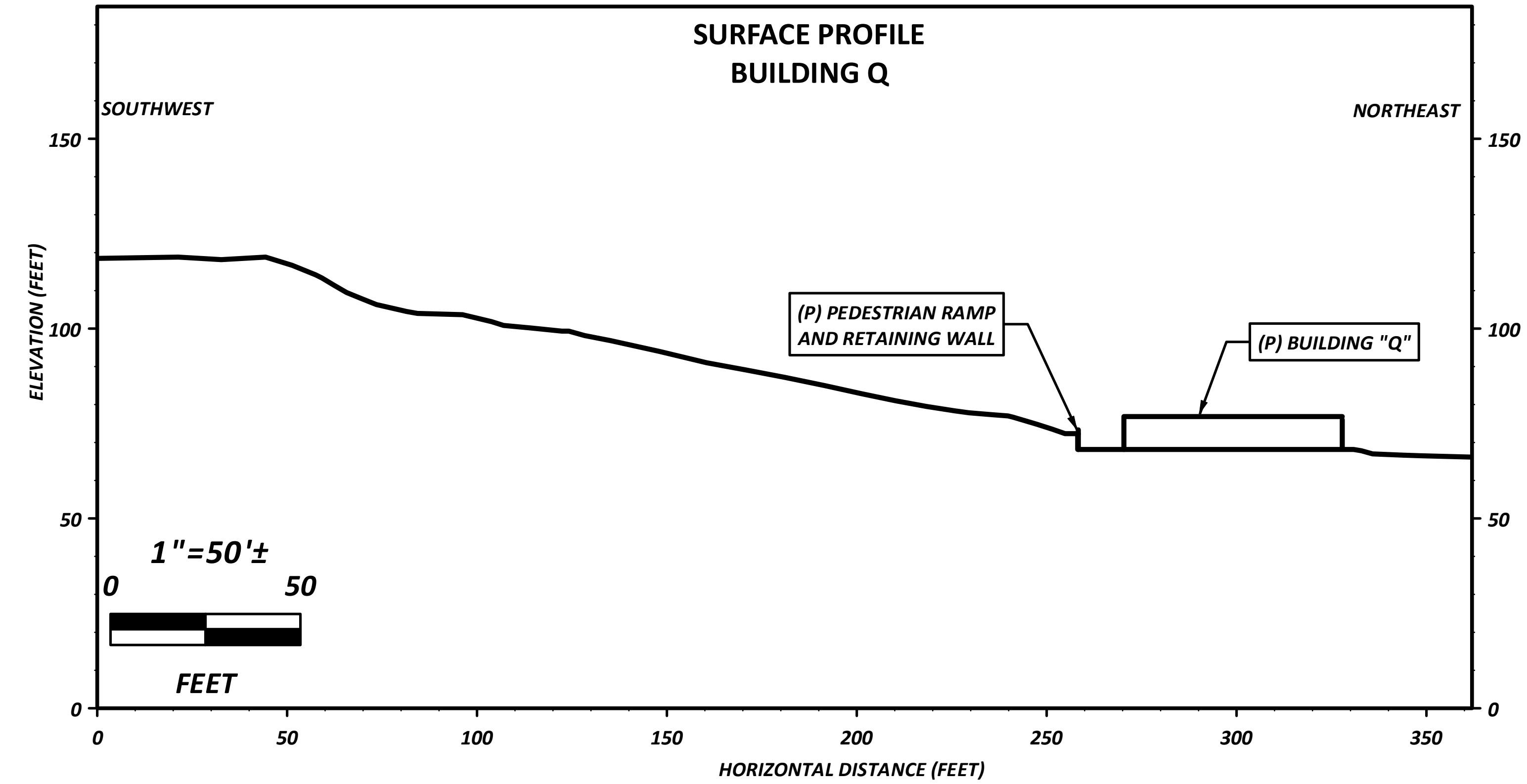
As is appropriate, the home field of the Eureka High "Loggers" in Albee Stadium is surrounded by slopes forested with second-growth redwood trees. Cooper Gulch slopes surrounding Albee Stadium and the adjacent facilities are low to moderate gradient slopes (generally 30 percent to 45 percent) formed in late Pleistocene age marine terrace sediments. Past slope grading within the area appears to have been primarily focused at the existing base of the slope (minor retreat appears to have occurred to enhance the flat area when the fields were initially graded) and in areas associated with the existing improvements; that is, in the cleared, non-forested areas. Existing improvements on the Albee Stadium slopes include the concrete bleacher complex on the western slope, three massive concrete stadium light foundations on the eastern slope, and various iterations of signs on the eastern slope above the field (see front report cover for an early example). A low (4-foot-high) retaining wall is present at the base of the slope along the eastern side of the track. The forested slopes appear to be natural slopes that have not experienced apparent grading.

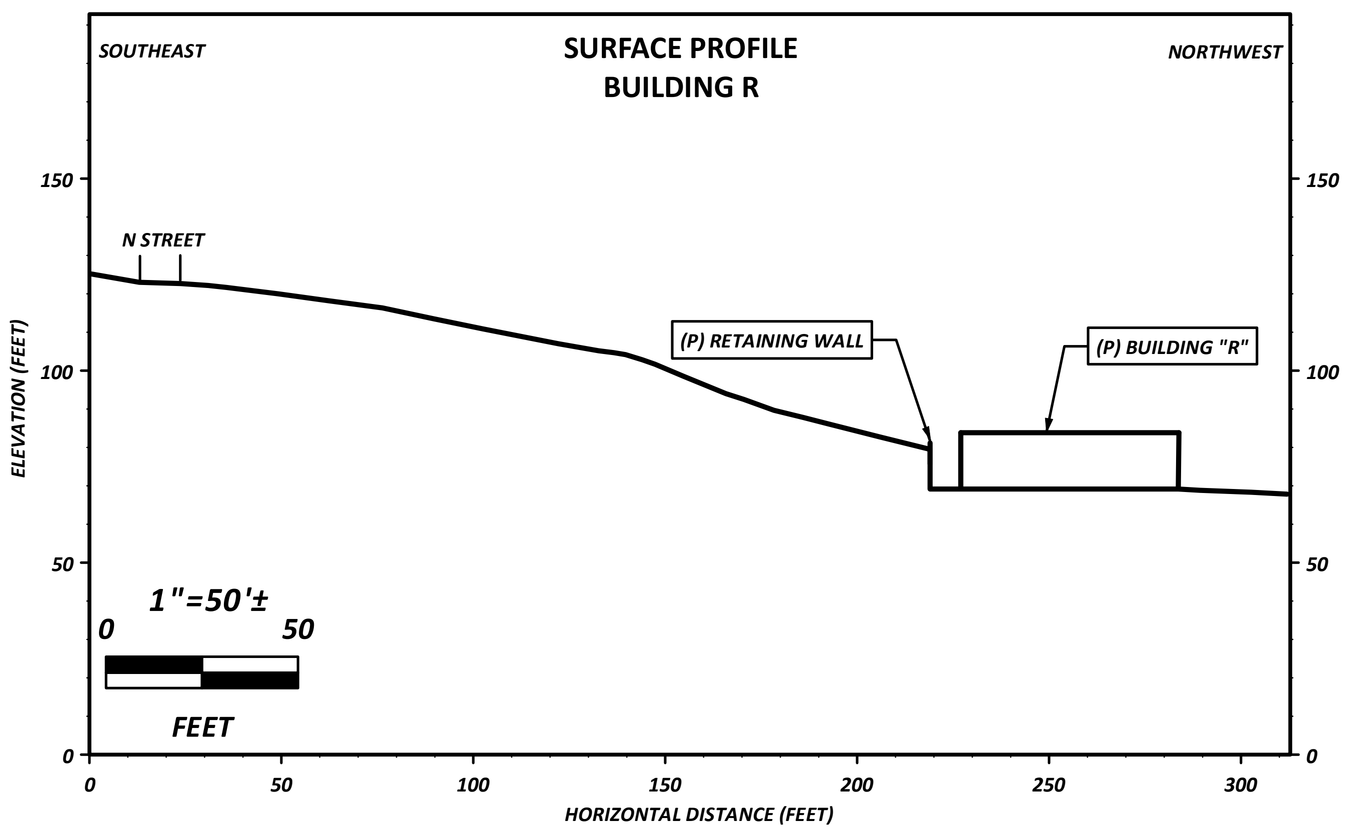
Slopes in the project area are not known to be unstable; there are no mapped landslides on Cooper Gulch slopes and no known historic accounts of unstable slopes at EHS or in the vicinity. The marine terrace deposits are not associated with distinct bedding that would be associated with bedding plane failures. From a geomorphic standpoint, the slopes are smooth and support straight-standing conifer trees; they exhibit no evidence of either shallow debris slides or deeper-seated rotational slides. Past grading at the site has not resulted in apparent unstable areas.

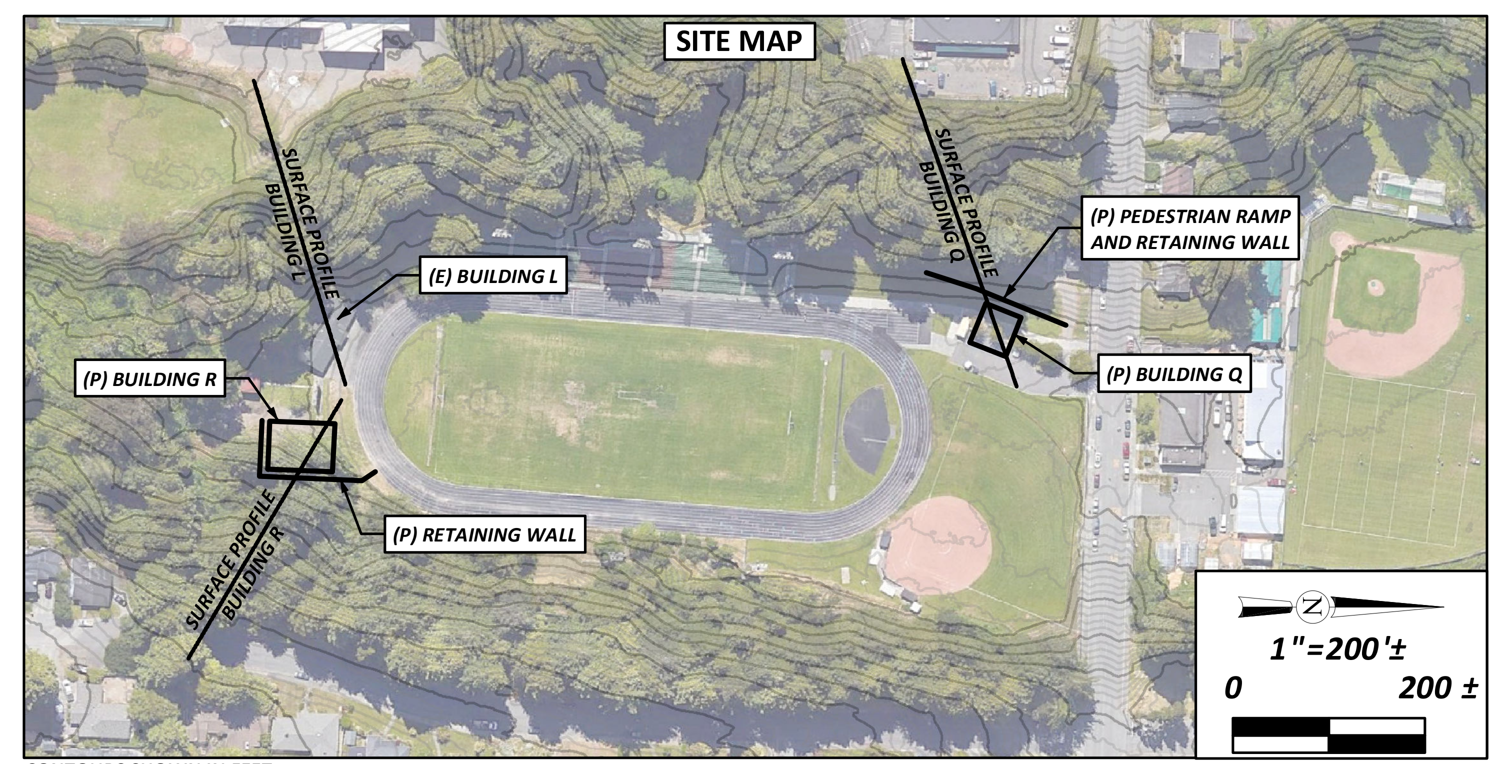
Proposed new buildings Q and R are located in low gradient areas removed from significant slopes. Slope profiles adjacent to these structures (without vertical exaggeration) are included in Figure 8 and indicate slopes on the order of 30 percent in these areas. The western end of the existing structure, Building L, is located at the toe of a moderate gradient slope that exhibits no geomorphic evidence of mass wasting. The slope toe above the building appears to have been graded in the past to control drainage, as there is considerable spring flow on the slope adjacent to Building L. A slope profile above Building L is included on Figure 8 as well.

In general, there appears to be a low potential for localized shallow instability on the slopes surrounding Albee Stadium and a negligible potential for a deep-seated landslide under all but the worst-case scenario (very large earthquake during the rainy season). The proposed new structures are not in areas









CONTOURS SHOWN IN FEET

NOTE: SURFACE ELEVATION INFORMATION FROM 2009-2011 CA COASTAL CONSERVANCY LIDAR: COASTAL CALIFORNIA

Eureka City Schools
Albee Stadium Geotechnical Investigation
Eureka, California

Surface Profiles

SHN 020010.100

September 2021 | Figure8_SurfaceProfiles

Figure 8

exposed to these low-level slope stability hazards, as they are planned in areas with low gradient slopes, and we conclude they are favorably located from a slope stability standpoint. In the absence of apparent exposure to landslide hazards, quantitative slope stability analysis does not appear warranted.

Storm Drainage Erosion-related Settlement

As described above, the existing storm drainage system is aged, deteriorated, and subject to pending upgrades. In the past, excessive cumulative erosion around drainage laterals has resulted in notable depressions (sinkholes). Past investigation of the storm drainage system as part of engineering studies has evaluated these depressions (SHN, 2018), and extensive inline camera work has been completed to evaluate the condition of the main storm drainpipe and several lateral pipes. The results of these studies indicate that the depressions have formed at damaged laterals or area drains and over the main storm drain near its downstream end.

Where damaged laterals or area drains are present, field evidence and inline photography shows sediment along the failed laterals has washed down the storm drain, resulting in localized voids. Erosion was noted in the inline camera work surrounding several failed laterals near their intersections with the main storm drain. Where a void propagates to the ground surface, it has (on two occasions) formed a depression. An 8-foot-wide depression on the softball field in 2014 was noted to be related to failing perforated area drains that allowed sediment to erode into the storm drain system.

North of Del Norte Street, where the fill transitions to clean, loose sand (imported dune sand), the storm drainpipe transitions from concrete pipe to corrugated metal pipe (CMP). In-line camera work showed the CMP portion of the drainpipe to be significantly degraded and increasingly filled with sand toward the downstream end of the pipe. In 2016, a 2-foot-diameter, 3-foot-deep sinkhole formed within a larger 12-foot-diameter depression directly over the transition from concrete pipe to CMP. Excavations at the time noted that the banding clamp at the junction had failed and that the CMP was badly corroded with several visible holes. The area was repaired and backfilled, but subsequent inline photography has shown additional areas of corrosion. Small sinkholes were noted in the baseball outfield as recently as this spring, again, directly atop the buried storm drain; the baseball field is currently closed until storm drain repairs can be completed.

We understand that plans for upgrades to the storm drainage system are being developed and will be implemented as part of the overall upgrades to Albee Stadium. It appears the main storm drain line will be rehabilitated via trenchless methods within Albee Stadium and that new drainage laterals will be constructed. The existing drainage laterals beneath the track/field in Albee Stadium will be abandoned (removed or grouted); the locations of most of these are believed to be known, although we anticipate the potential for encountering previously unidentified laterals during site preparation. Where the storm drain line is significantly degraded north of Del Norte Street (the CMP beneath the baseball field), it will be removed and replaced.

As part of the storm drain system rehabilitation, existing areas of subsurface erosion (voids) will need to be located and treated. In general, the potential for existing voids appears significant (and significantly higher) in areas north of Del Norte Street (baseball field) where loose sandy fill is present. The existing storm drain is extensively degraded (and sand-filled), and recent settlement has been observed. The potential appears lower in Albee Stadium, where older, more cohesive fill soils are present and historic settlement has been more limited.

Geotechnical Discussion and Conclusions

General

Based on the results of our geotechnical investigation, SHN concludes that the site can be developed as planned for the proposed construction, provided the recommendations presented in this report are followed. The main geotechnical considerations affecting the design and construction of the project are loose, unengineered fill soils underlying portions of proposed Buildings Q and R, the modest potential for liquefaction-induced settlement in the area of Buildings Q and R, and the need to provide uniform foundation support under the entire proposed structures. In addition, the apparent year-round standing water in the western area of existing Building L will make preparation of a firm subgrade for construction of the concrete ramps difficult without controlling the source of the standing water.

All geotechnical-related work should be performed in accordance with the recommendations of the Geotechnical Engineer-of-Record during construction. Where the recommendations of this report and the cited sections of Title 24 are in conflict, the Owner and Architect should request clarification from the Geotechnical Engineer-of-Record. The recommendations in this report should not be waived without the consent of the Geotechnical Engineer-of-Record for the project. The following sections present recommendations for the geotechnical-related work.

Seismically Induced Settlements

The potential for liquefaction was evaluated for the maximum considered earthquake (MCE) using a peak ground acceleration of 0.76g and a magnitude M_w 8.7 event, which is interpreted as the largest "modal" magnitude for the 475-year return period event as presented on the "Probabilistic Seismic Hazard Deaggregation" report provided by the USGS on its web application for this location (USGS, 2014). Liquefaction-induced settlement is estimated based largely on the empirical methodologies developed by Tokimatsu and Seed (1987), and Ishihara and Yoshimine (1992) for saturated soils. SHN's analysis was performed using the SPT-based liquefaction analysis software LiqSVs published by GeoLogismiki. Plots of the factor of safety against liquefaction versus depth with the cumulative settlement amounts determined for the proposed building locations are shown graphically on the liquefaction reports provided in Appendix 3.

SHN's liquefaction analysis indicates that 0.4 to 3 inches of post-liquefaction settlement may occur below the groundwater table following the design earthquake, although most of this modeled settlement results in late Pleistocene age marine terrace deposits that from a qualitative standpoint have a low liquefaction potential. We infer that areas likely to experience the most significant liquefaction (and settlement) are along the valley axis, where groundwater is highest and uncontrolled fills are thickest, but these areas support only parking and athletic fields, which are suitably low exposure improvements. The two proposed new buildings, Q and R, are to be located at the valley margins where fills are thin and likely taper out beneath the footprint of each structure.

At the location of the proposed Building Q, a thin veneer of uncontrolled fill soils is present overlying marine terrace sediments. It appears that most or all of the fill material will be removed during the site preparation process and replaced with a geogrid-reinforced engineered fill mat that is described below. At the location of Building R, up to about 8 feet of uncontrolled fill and buried alluvial sediments are present, tapering out over a short distance at the valley margin. Most of the unsuitable soils will be removed beneath Building R, but likely not all; in order to mitigate any remaining hazard, development of a suitable geogrid-reinforced fill mat beneath the footprint will be appropriate to mitigate differential settlement potential. To ensure uniform bearing support beneath Buildings Q and R, SHN recommends they be supported on a triaxial geogrid–reinforced engineered-fill mat. Ground improvements and foundation design construction recommendations intended to mitigate the potential for structural distress because of both static and earthquake-induced settlements are provided below in the “Site Preparation and Grading” section of this report.

At Building L, about 8 feet of uncontrolled fill, buried topsoil, and alluvium overlies a few feet of loose marine terrace material. The “pre-terrace mud” underlies the east end of Building L (closest to the valley center) at a depth of about 11 feet. Fill soils beneath Building L thin to about 3 feet at the western end, approaching the valley margin. Based on the settlement values described above, we assume a modest differential settlement potential, on the order of 1.5 inches across Building L, exists during strong seismic shaking. Inspection of the ±80-year-old building indicates it remains sound and shows no significant effects from past differential seismic settlement despite experiencing several significant seismic events. We find the potential for substantial liquefaction-related damage to Building L to be low.

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 2019 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 gravity forces, to minimize fire hazards and 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 service building as designed and constructed shall be capable of providing essential service to the public after a disaster.”

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. The recurrence interval for very strong earthquake events originating on the CSZ is 300 to 500 years. As discussed above, under “Cascadia Subduction Zone,” evidence suggests the last major subduction zone quake occurred on January 27, 1700.

Corrosivity

The results of soil corrosivity tests on a composite sample of soil collected from several borings are presented in Appendix 2. Buried metal and reinforced concrete should be designed to resist corrosion based on the test results, and cement types should be specified based on the test results. Corrosion testing should be performed on imported fill that will be in contact with buried metal and concrete.

Recommendations

Seismic Design Parameters

Based on the subsurface conditions encountered at our exploration locations, laboratory test results, and our interpretation of soil conditions within 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 American Society of Civil Engineers (ASCE) 7-16. On this basis, the mapped and design spectral response accelerations were determined using the Structural Engineers Association of California (SEAOC) and California Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps (Accessed October 8, 2020) in conjunction with the site class and the site coordinates (40.789037° N, -124.155442° W). Calculated values for ASCE 7-16 are presented in Table 1.

Table 1. ASCE 7-16 Spectral Acceleration Parameters

Parameter	0.2 Second	1 Second
Maximum Considered Earthquake Spectral Acceleration (MCE_R)	S_S = 2.812	S_1 = 1.085
Site Class	D	
Site amplification factor	F_a = 1	F_v = null – See Section 11.4.8
Site-modified spectral acceleration	S_{MS} = 2.812	S_{M1} = null – See Section 11.4.8
Numeric seismic design value	S_{DS} = 1.875	S_{D1} = null – See Section 11.4.8
Seismic Design Category (SDC)	E	
MCE_G peak ground acceleration (PGA)	1.147	
Site amplification factor at PGA (F_{PGA})	1.1	
Site modified peak ground acceleration (PGA_M)	1.261	

The 2019 CBC incorporates procedures outlined in ASCE 7-16. Section 11.4.8 of ASCE 7-16 and other referenced sections provide options for either developing a ground motion hazard analysis or taking exceptions. The applicable exception for this project is Exception 2 because the design Site Class is D and because S_1 is greater than 0.2 g (acceleration of gravity). Exception 2 requires using a seismic response coefficient C_S determined by Eq. 12.8.2 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8.3 for $T_L \geq T > 1.5T_S$ or Eq. 12.8.4 for $T > T_L$. The intent of the code is to increase the design seismic base shear for longer periods unless a detailed ground motion hazard analysis is performed allowing for lower design base shears for the longer periods.

Section 11.4.8 permits a site response analysis in accordance with Section 21.1 of ASCE 7-16 and/or a ground motion hazard analysis in accordance with Section 21.2 to determine ground motions for any structure. SHN can, however, provide the ground motion hazard analysis if the structural engineer/owner determines that developing one will significantly reduce construction and design costs.

Site Preparation and Grading

Site preparation includes demolition of existing surface and subsurface improvements and removal of debris, organics, organic topsoil, loose soil, and any other unsuitable material. Site preparation operations should extend at least 5 feet beyond the limits of improvements. We anticipate that stripping to a depth of about 2 to 4 inches will be required to remove the organics and topsoil. Deeper stripping may be locally required to remove concentrations of vegetation, such as brush and tree roots. Where the removal of large trees is required, it will be necessary to remove all major root systems, then fill the excavations with properly placed engineered fill compacted to at least 90 percent relative compaction[1]. The cleared vegetation and debris should be removed from the site, but the strippings can be stockpiled for reuse in landscape areas.

Any vegetation and organic topsoil with more than 2 percent organic material by dry weight should be removed. The Geotechnical Engineer should observe and approve the prepared site prior to any excavation, subgrade preparation, and placement of fill or improvements.

We expect that the site soils will be excavatable with conventional grading and trenching equipment. If grading commences in the winter or spring, or after a period of excessive rainfall, it is likely that the surficial soils will become saturated due to the presence of fine-grained material. Further, perennial wet areas appear to be present at the site (see the discussion above regarding groundwater). Wet or saturated soil may cause difficulties in access with grading and trenching equipment and difficulties in loading, spreading, and compaction of fill material. Moisture conditioning and/or aerating of the site soils may be required. The time required for drying can be reduced by disking, ripping, or otherwise aerating the soil.

The contractor shall be responsible for the stability of all temporary excavations and should comply with applicable Occupational Safety and Health Administration (OSHA) regulations (California Construction Safety Orders, Title 8). The Contractor should periodically monitor all open cuts for evidence of incipient stability failures.

Buildings Q and R

As previously discussed, there are loose unengineered fill soils underlying the proposed Buildings Q and R, a modest potential for liquefaction-induced settlement in the area of Buildings Q and R, and the need to provide uniform foundation support under the entire proposed structures. The area to contain the proposed Buildings Q and R and for a horizontal distance of at least 5 feet beyond each building, should be over-excavated to a minimum depth of 5 feet below proposed subgrade elevation. This will allow for the removal of any loose surficial soils and a majority of the unengineered fill soils. The over-excavated subgrade should be scarified to a depth of 6 inches, moisture conditioned or aerated and recompacted to 90 percent relative compaction. The Geotechnical Engineer or qualified representative should observe and approve the over-excavation, and prior to subgrade preparation and placement of engineered fill or improvements.

---

1 Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same soil, as determined by the ASTM D1557 compaction test procedure. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Following recompaction of the over-excavated subgrade, we recommend that a layer of triaxial geogrid (Tensar® TX160 or equivalent) be placed on the exposed subgrade (prior to backfilling the over-excavated area with engineered fill) and extending up the sides of the excavation with 5 feet of excess geogrid. A conceptual drawing showing the preferred method of geogrid placement is included as Figure 9. With the basal layer of geogrid in place, place and compact 24 inches of engineered fill in 8-inch-thick loose layers to a minimum of 90 percent relative compaction. A second layer of triaxial geogrid should be placed on the 24-inch-thick layer of engineered fill. Place and compact an additional 24 inches of engineered fill in 8-inch-thick loose layers to a minimum of 90 percent relative compaction. Extend the layer of triaxial geogrid that had been placed up the sides of the excavation 5 feet over the last layer of properly compacted engineered fill on both sides (a 5-foot "return" per Figure 9). Place the final 12 inches of engineered fill compacted to a minimum of 90 percent relative compaction. We, therefore, anticipate that approximately 5 feet of engineered fill, with 2 layers of triaxial geogrid and a 5-foot "return," will be placed below the proposed building footprints.

The placement of a triaxial geogrid–reinforced engineered-fill mat below the proposed Buildings Q and R is intended to minimize the estimated differential settlements caused by any settlement of the underlying liquefaction-susceptible soils that undergo volumetric strain due to post-liquefaction reconsolidation. In addition, the high tensile strength of the triaxial geogrid reinforcement is expected to reduce the potentially damaging effects associated with liquefaction-induced ground surface deformation, if they occur.

Storm Drainage System Rehabilitation

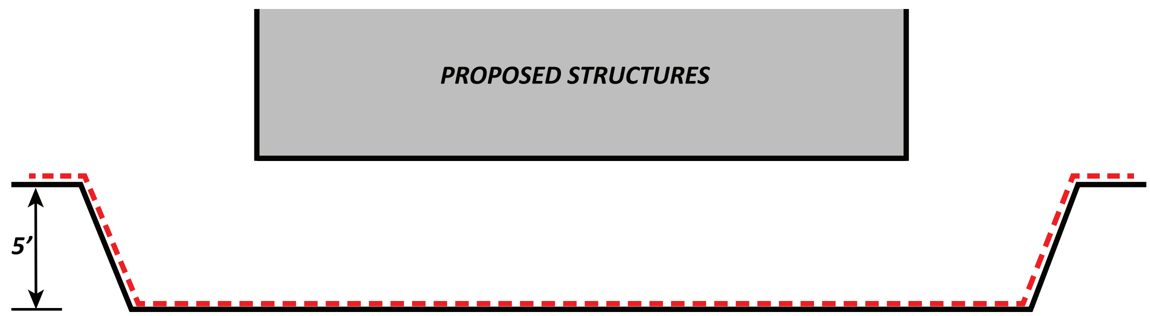
In order to mitigate the potential for existing soil voids following the rehabilitation of the storm drainage system, we recommend the following:

- North of Del Norte Street, on the baseball field, replace the existing storm drainpipe by exposing it in an excavation with 2:1 side slopes. We understand the pipe to be 16 to 20 feet deep, so this will be a substantial excavation. We assume this will allow removal and mitigation of existing soil voids or eroded areas during the excavation and backfilling process. If additional voids or soft areas are identified, dig them out as appropriate, under the supervision of the project Geotechnical Engineer or qualified representative. Fill the excavation, as prescribed below, with properly compacted engineered fill benched into firm soils along the bottom and sides to provide a firm level surface on which to place new engineered fill.

  Special consideration will need to be made at the south end of the excavation, near the existing Wood Shop structure. The excavation should not undermine or compromise the existing structure; therefore, underpinning, or temporary retaining structures may be required. Any unreinforced excavation should be at least 10 feet from the existing structure.

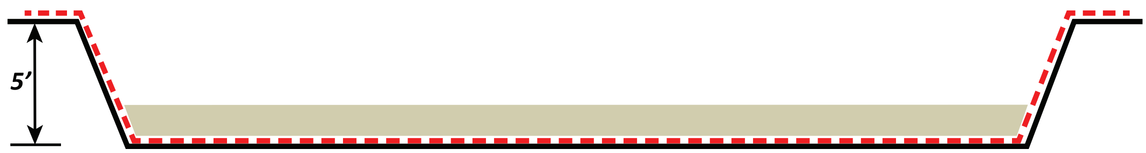
- South of Del Norte Street, in Albee Stadium, conduct a thorough subgrade review once the existing track and fields have been removed during the Site Preparation phase to locate all existing drainage laterals or other debris to be mitigated. Where existing laterals are encountered, remove them, or fill them with grout, as appropriate. Removal of existing deeper laterals should extend at least 8 feet below grade. For laterals extending below 8 feet, the Geotechnical Engineer will determine whether additional excavation is required or whether a geogrid-supported engineered fill section will adequately bridge the remaining lateral.

GEOTEXTILE REINFORCEMENT SCHEMATIC



5'

STEP 1: EXCAVATE TO 5 FEET DEPTH (MIN.) AND LAY A LAYER OF TENSAR TRIAX TX TO COVER FOUNDATION AREA AND EXTEND UP SIDES OF FOUNDATION.

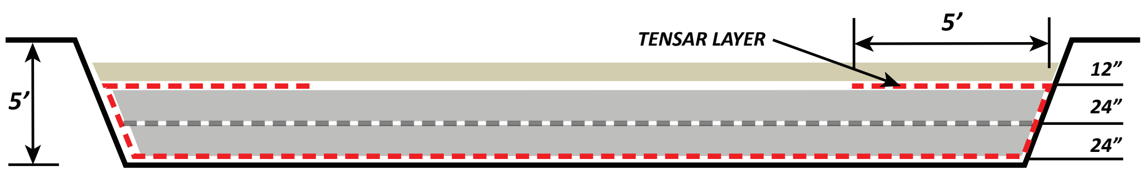


5'

STEP 2: PLACE AND COMPACT ENGINEERED FILL TO A COMPACTED DEPTH OF 24 INCHES IN 8-INCH LIFTS.

5'

STEP 3: EXTEND A LAYER OF TENSAR TRIAX TX TO THE EDGE OF THE EXCAVATION FILL AND PLACE A 24-INCH LAYER OF ENGINEERED FILL ON TENSAR TRIAX TX.



5'

STEP 4: FOLD THE LAYER OF TENSAR TX THAT WAS EXTENDED UP THE SIDE OF THE EXCAVATION BACK OVER PRIOR LAYER OF ENGINEERED FILL (5 FEET EACH SIDE) AND PLACE A 12 INCH LAYER OF ENGINEERED FILL ON TENSAR TRIAX TX AND COMPACT THE REMAINING FILL IN 8-INCH LIFTS TO SUBRADE ELEVATION.

Existing drainage laterals are expected only in areas to be developed with athletic fields. Existing drainage laterals are not known to exist beneath the areas proposed for Buildings Q and R.

Albee Stadium Track and Field

Current plans indicate the existing track and football field will be replaced. We have not, to date, received development plans for these improvements. Based on recent experience, however, we can anticipate several likely complications to the development of a new track and field. As described above, existing fill soils in Albee Stadium are highly variable, and locally very wet. Soils beneath the existing track are likely well above optimum moisture and will require significant drying and/or conditioning. Management of run-on seepage from adjacent slopes will require specific planning. We expect that site preparation and grading will encounter soft subgrade areas that will require some form of stabilization. Removal and replacement of these areas with stabilization rock, or placement of geogrid reinforcement, are options. Once specific design plans become available, additional geotechnical input may be required. General recommendations for replacement of the running track and construction of an artificial turf football field are presented below.

Synthetic Turf Football Field

If synthetic turf is used to upgrade the football field, the existing fill soils may need to be over-excavated. If unstable or pumping subgrade soils are encountered, the area may need to be over-excavated and backfilled with 1.5-inch minus crushed rock and overlain with a woven stabilizing fabric, such as, Mirafi 500X or equivalent; the Geotechnical Engineer should provide recommendations based on exposed conditions.

The compaction and material requirements, as well as the drainage section, including subdrainage systems, are typically proprietary and should be determined by the synthetic turf manufacturer. However, the recommendations below are considered typical for synthetic turf installation.

The new synthetic turf should be underlain by a minimum 6-inch drainage section of permeable material, such as ¾-inch crushed rock or California Department of Transportation (Caltrans) Class 2 Permeable Material. The drainage section should meet the minimum requirements of the synthetic turf manufacturer for compaction, gradation, permeability, and stability. The compacted subgrade below the synthetic turf should be overlain with a nonwoven filter fabric such as Mirafi 170N or equivalent prior to placement of the drainage section. The nonwoven filter fabric should be installed to protect the aggregate drainage material from contamination by the underlying subgrade.

Running Track Replacement

We anticipate that following the removal of the existing running track surface, that large areas of pumping subgrade soils will be encountered (commonly found underneath old asphalt pavement areas) with heavy, rubber-tired construction vehicles driving on the exposed wet subgrade soils. In order to construct a new pavement section of adequate strength under the proposed running track replacement, stabilization of the subgrade soils will probably be necessary.

Depending on the severity of pumping subgrade soils, the following is a description of subgrade stabilization options that we have observed to be successful in the past:

1) **Geogrid Placement:** For subgrade areas that exhibit mild pumping, install a layer of geogrid (TenCate Mirafi® BXG120 or equivalent) on the exposed subgrade soils. Follow all geogrid manufacturer's installation procedures (for example, subgrade preparation, geogrid overlap, and so on). Extend geogrid at least 18 inches beyond the perimeter of soft soil areas. Continue backfilling with the specified Class 2 Aggregate Base material. Assessment of whether or not the subgrade has been adequately stabilized should be based on compaction testing (ASTM D6938) of the aggregate base material and proof-rolling of final aggregate base grade.
2) **Over-Excavation and Replacement**: For subgrade areas exhibiting moderate to severe pumping, over-excavate an additional 12 to 18 of subgrade material from soft areas. Install a geotextile stabilization fabric (Mirafi® 600X or equivalent) over the exposed subgrade. Place 3-inch-minus crushed, durable rock over the stabilization fabric, for a total rock thickness of approximately 12 to 18 inches. Install another layer of geotextile stabilization fabric over the layer of crushed rock, with fabric extending at least 2 feet beyond the perimeter of rocked area. Follow all geotextile manufacturer's installation procedures (for example, subgrade preparation, geotextile positioning and overlap, and so on). In particular, hold the fabric tightly in place at its edges (using pins/staples/soil/and so on) so that fabric is taut when the specified Class 2 Aggregate Base material is installed to final grade. Do not allow vehicles to drive directly on geotextile. Assessment of whether or not the subgrade has been adequately stabilized should be based on compaction testing (ASTM D6938) of the aggregate base material and proof-rolling of final aggregate base grade.

Depending on the option selected, we recommend using the technique in a small "test area." This will ensure that the technique yields adequate results, before attempting the technique in all areas. Where mentioned, "proof-rolling" means an inspection during which the Geotechnical Engineer or qualified representative observes a fully loaded dump truck driving over the improved areas, to ensure that "pumping" and other signs of failure are not evident.

The new running track at Albee Stadium will serve as an emergency vehicle access and will need to accommodate loads from a fire truck. Recommendations for an asphalt pavement section to be placed below the running track that can accommodate loads from fire trucks are presented below in "Asphalt and Concrete Pavements."

Engineered Fills

Engineered fill should have less than 2 percent by dry weight of vegetation and deleterious material and should meet the gradation requirements presented in Table 2.

Table 2. Engineered Fill Gradation Criteria

Sieve Designation	Percent Passing by Dry Weight
3-inch (50 mm)[a]	100
2½-inch (37.5 mm)	85 minimum
¾-inch (19 mm)	70 minimum
No. 4 (4.75 mm)	60 minimum
No. 200 (75 μm) [b]	5 minimum, 30 maximum

[a] mm: millimeters
[b] μm: micrometers

We anticipate that onsite soils will be suitable for reuse as engineered fill following removal of debris, organics, and any other unsuitable material. Fine-grained soil with a liquid limit greater than 40 and a plasticity index greater than 15 should not be used as engineered fill. If clayey soils do not meet the plasticity requirements, mixing of the clayey soils with sandier soils may be required. Crushing and/or removal of rock particles greater than 3 inches in size, should they be encountered, will be required. Select engineered fill should have a low corrosion potential, which is defined as a minimum resistivity of 2,000 ohms-centimeter (ohms-cm) and maximum sulfate and chloride concentrations of 250 parts per million (ppm). In addition, we do not recommend using river-run material as engineered fill; crushed, angular material is preferred with at least 50 percent of the material (as determined by the material's dry weight) containing a minimum of two fractured faces.

Engineered fill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to a minimum of 90 percent relative compaction. The Geotechnical Engineer or qualified representative should approve all fill prior to placement.

A qualified field technician should be present to observe fill placement and perform field density tests in accordance with ASTM D 6938 at random locations throughout each lift to verify that the specified compaction is being achieved.

Samples of proposed import fill materials should be submitted to SHN for approval at least three business days prior to use at the site.

Excavations

Excavations should be made in accordance with OSHA specifications and conditions. Excavations deeper than 4 feet BGS (or shallower if excavations appear unsafe) should be laid back to a safe slope inclination or supported by an appropriate shoring system. Slopes for excavations deeper than 20 feet are required to be designed by a licensed engineer. It should be noted that the Contractor is solely responsible for site safety and safe working conditions during construction. A temporary or permanent shoring system should be installed in a configuration that will allow vertical side slopes for deep excavations where laying back the excavation is impractical.

Excavated soils should be placed a minimum of 10 feet away from the edge of the below-grade excavation to reduce surcharge loads on the temporary cut slopes. If shoring systems are used, the effects of the soil stockpile on the shoring system should be considered during design if the soils are placed in the area between the top of the excavation and a 1H:1V (horizontal to vertical) projection from the toe of the excavation, to reduce the potential of a shoring failure.

Similarly, heavy equipment should be operated in a safe manner and should be kept an adequate distance from unshored excavation sidewalls to prevent a cut slope stability hazard. If shoring is used, surcharge loads from heavy equipment should be considered in the design calculations to prevent a surcharge failure during construction. For an unshored excavation, a heavy equipment exclusionary zone should be established based on soil type, depth of excavation, presence of groundwater, and configuration of the open cut. As a general guideline, heavy equipment should be excluded from a zone located between the top of the excavation and a 1H:1V projection from the bottom toe of the adjacent excavation sidewall. This may be modified in the field for specific geotechnical conditions.

Cut and Fill Slopes

All permanent cut slopes up to 5 feet in height should be no steeper than 1.5 horizontal to 1 vertical (H:V). Higher or steeper slopes should be reviewed by the Geotechnical Engineer for stability. The Geotechnical Engineer or qualified representative should be notified by the Contractor prior to beginning slope excavations and should review the cut slopes during excavation.

All permanent fill slopes up to 5 feet in height should be no steeper than 2H:1V. Higher or steeper fill slopes should be reviewed by the Geotechnical Engineer for stability. All material used to construct fill slopes should meet the select engineered fills specifications and compaction requirements outlined above. Areas to receive select engineered fill should be prepared following the steps outlined above in "Site Preparation and Grading." Where fill is placed on existing slopes steeper than 5H:1V, the fill should be keyed and benched into competent native soil.

Wet Weather Subgrade Protection

The near-surface soils consist of loose, non-cohesive, fine-grained granular materials and/or fine-grained silts. We expect that both light and heavy construction equipment will have difficulty operating on the near-surface soils if grading commences during and/or immediately following the wet season. Contactors should expect high soil moisture conditions in the near-surface soils throughout the wet season and into the late spring months following a typical winter wet season, and in the common perennially wet areas at the site. The wet season in coastal northern California generally begins in the month of November and continues through May. Heavy rains are also not uncommon during the months of October and June. Beginning construction activities and earthwork immediately prior to the onset of the wet season is not advised and will likely lead to delays if measures are not taken to stabilize and protect the exposed subgrade.

Soils that have been disturbed during site preparation activities, or unsuitable areas identified during proof-rolling or probing, should be removed to firm ground and replaced with stabilization material and compacted structural fill.

Protection of the subgrade is the responsibility of the Contractor. Track-mounted excavating equipment may be required during and following wet weather. The Contractor will be responsible for constructing an all-weather access road and staging area. The thickness of the haul road to access the currently undeveloped portions of the site for construction and staging areas will depend on the amount and type of construction traffic. The materials used for haul roads or site access drives should be stabilization material consisting of pit or quarry run rock that is well-graded, angular, crushed rock consisting of 4- to 6-inch minus material with less than 5 percent passing the US Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material. A minimum 6- to 12-inch-thick mat of stabilization material should be used for light staging areas. The stabilization material for haul roads and areas with repeated heavy construction traffic will likely need to be increased to between 12- to 18-inches. The actual thickness of haul roads and staging areas should be based on the Contractor's approach to site work and the amount and type of construction traffic and is the Contractor's responsibility. The stabilization material should be placed in one lift over the prepared, undisturbed subgrade and compacted using a smooth-drum, non-vibratory roller. Additionally, a geotextile fabric should be placed as a barrier between the subgrade and stabilization material. The geotextile should meet specifications for soil separation and stabilization, such as Mirafi 600X or equivalent.

Surface and Subsurface Drainage Control

Surface drainage should be planned to prevent ponding and enable water to drain away from foundations, slabs-on-grade, edges of pavements and tops of slopes and retaining walls, and towards suitable collection or discharge facilities. Surface drainage should be designed to capture runoff from upslope of proposed structures. A positive surface drainage of at least 4 percent is recommended within 10 feet of all building foundations and retaining walls in unpaved areas. In paved areas, a positive surface drainage of at least 1.5 percent is recommended to allow for rapid removal of surface water. Roof drainage systems should be planned to direct rainwater away from building foundations and retaining walls.

A concrete-lined interceptor drainage ditch (that is, V-ditch) at least 30 inches wide and 12 inches deep should be used along the top of retaining walls and cuts where the tributary drainage area above slopes towards the wall or cut. Slope drainage and terracing should also be provided for cut and fill slopes steeper than 3H:1V (horizontal to vertical), as indicated in CBC Appendix Chapter 13.

Concentrated water should not be discharged onto bare ground or slopes but should be carried in pipes or lined channels to suitable disposal points. Because onsite soils generally have a moderate potential for erosion, we recommend that approved temporary and permanent erosion control measures be implemented to limit erosion and comply with applicable City of Eureka regulations.

Soils on graded slopes should be fertilized, mulched, and planted as soon as possible after grading with erosion resistant vegetation. These plants should be watered lightly at appropriate intervals until growth is established; drip irrigation systems are recommended.

The use of water-intensive landscaping around the perimeter of structures should be avoided to reduce the amount of water introduced to the subgrade. Irrigation of landscaping around structures should be limited to drip or bubbler-type systems. Trees with large roots should also be avoided since they can dry out the soil beneath foundations and cause settlement. The purpose of these recommendations is to avoid large differential moisture changes adjacent to foundations, which have been known to cause large differential movement over short horizontal distances in expansive soils, resulting in cracking of slabs and architectural damage.

In addition, surface drainage should adhere to the setbacks for low-impact development (LID) features as shown in Table 3.

Table 3. Recommended Setbacks for LID[a] Features

Type of LID Feature	Setback from Building Foundations	Setback from Pavement Sections and Exterior Slabs-on-Grade
Designed to infiltrate collected and concentrated stormwater (that is, dry wells, vegetated swales, bioretention facilities)	10 feet	3 feet[b]
Alternative engineered hardscaping (that is, porous asphalt, permeable pavers) subject only to incidental rainfall (*not* subject to re-routed, concentrated stormwater)	5 feet	3 feet[b]

[a] LID: low-impact development

[b] Setback is not required *only if* an effective barrier is installed (such as a concrete-filled cutoff trench that prevents moisture from traveling from the LID feature to below the pavement section/slab-on-grade).

Spring sources are evident along the Cooper Gulch valley walls surrounding much of the project area, and wet areas are abundant along the margins of much of the site. Where wet areas are observed, such as, along the base of the descending slopes and at the base of the bleachers, we recommend the installation of a cut-off trench. The cut-off trench should consist of a trench (approximately 1-foot-wide and 4-feet-deep) and sloped to drain at a minimum 1 percent gradient toward project storm drains. Subdrains should consist of 4- to 6-inch-diameter perforated pipes surrounded by drainage aggregate that connects to solid discharge pipes. The pipes should be standard dimension ratio (SDR)-35, Schedule 30, or stronger. The drainage aggregate should be Class 2 Permeable Materials (Caltrans Specification) or open-graded rock with less than 2 percent fines by dry weight. If open-graded rock is used, then the contractor should wrap it in filter fabric (Mirafi 140N or equal). Subdrain pipes should lie over at least 3 inches of drainage aggregate, and perforations should face downward. Aggregate width should be at least 12 inches. The drainage material should be capped with a minimum of 12-inches of compacted fine-grained soil, soil-cement, or other relatively impermeable material or barrier.

The contractor should add cleanouts to upstream ends of subdrains. Junction angles in pipes should be no sharper than 45 degrees, and the angles should sweep from the upstream to downstream direction to allow for access of cleanout equipment to the entire pipe system from the cleanouts.

Utility Trench Backfill

New utility trenches excavated parallel to spread footing foundations should be set back from the footings such that the trench bottoms lie outside a projected hypothetical 1.5H:1V line extending downward from the footing bottom.

Unless concrete bedding is required around utilities, bedding should consist of sand having a sand equivalent (SE) of at least 30. The bedding should extend from 6 inches below to 1 foot above the conduit or pipe. Sand bedding should not be jetted or ponded into place and should be mechanically compacted to a minimum of 90 percent relative compaction.

In areas to support improvements (such as adjacent-to-structure foundations), backfill placed above the bedding in utility trenches (including culvert and sprinkler lines) should be properly placed and adequately compacted to minimize settlement and provide a stable subgrade. If possible, the trench backfill should be compacted following rough grading, but prior to final grading and compaction. Onsite inorganic soils meeting the requirements for engineered fill may be used as trench backfill. Backfill consisting of onsite soils should be placed in layers not exceeding 8 inches in loose thickness, moisture-conditioned, and compacted to at least 90 percent relative compaction as described for engineered fill. Trench backfill need only be compacted to 85 percent relative compaction in landscape areas or in areas more than 5 feet beyond the limits of building foundations.

Where utility trenches cross underneath buildings, we recommend that a plug be placed within the trench backfill to minimize the normally granular backfill from acting as a conduit for water to enter beneath the building. The plug should be constructed using sand cement slurry (minimum 28-day compressive strength of 500 pounds per square inch [psi]) or relatively impermeable native soil for pipe bedding or backfill. We recommend that the plug extend a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

Foundations

Buildings Q and R

The proposed Buildings Q an R structures may be supported on conventional reinforced concrete spread footing foundations bearing on a level pad underlain by a triaxial geogrid-reinforced engineered fill pad. Spread footing foundations for structures supported by engineered fill soils may be designed to support dead loads plus normal duration live loads using an allowable bearing capacity of 2,500 pounds per square foot (psf) provided the footings are embedded at least 18 inches below lowest adjacent subgrade elevation. Footing widths should meet the minimum values given in the 2019 CBC. The allowable bearing capacities given above may be increased by one-third when considering short-term wind and seismic loads.

The maximum total settlement of foundations designed as described above and using the allowable bearing values given above is not expected to exceed 1 inch. The maximum differential settlement between adjacent wall and/or column footings is not expected to exceed ½ inch.

A horizontal friction coefficient of 0.30 times the net vertical dead load may be used for the footing/soil contact. 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 loading, such as lateral foundation response to wind or earthquake loadings. Passive pressure should be neglected in the upper 12 inches unless confined by concrete slabs or asphalt pavement.

All foundation excavations should be observed and approved by the Geotechnical Engineer prior to placement of forms and reinforcing steel. The excavations should be trimmed neat, level and square. All loose, sloughed, and moisture-softened materials should be removed prior to setting reinforcing steel and placement of concrete.

Stadium Lighting, Backstops, and Scoreboard

The planned stadium lighting, backstops, and scoreboards should be supported on drilled, reinforced, cast-in-place, concrete friction pier foundations. The piers for the stadium lighting should be at least 18 inches in diameter and bottomed at least 10 feet below the depth where a minimum of 10 feet of lateral confinement is obtained. The piers should be designed using an allowable skin friction of 700 psf of surface area per foot of depth for dead plus long-term live loads, starting below the depth where a minimum of 10 feet of lateral confinement is obtained. This value may be increased by one-third when evaluating total loads including wind and seismic forces. Drilled piers for the proposed backstops and scoreboards should be at least 18 inches in diameter and be bottomed a minimum of 10 feet deep. These drilled piers may be designed using an allowable friction capacity in axial compression of 600 psf per foot of depth for dead plus long-term live loads. This value may be increased by one-third when evaluating total loads including wind and seismic forces. Eighty percent of these values may be used in determining uplift resistance. The upper three feet should be neglected in determining axial capacities.

Please note that the above values are recommended minimum pier dimensions and that other structural criterion, such as the need to resist lateral forces, may force the pier design diameters and depths to be greater.

Resistance to lateral loads can be obtained from passive earth pressure acting on pier faces. A passive earth pressure of 300 pcf (triangular distribution), starting at a depth where at least 10 feet of lateral confinement is obtained, should be used. The upper 3 feet of scoreboard piers should be neglected in determining resistance to lateral loads. Passive pressure can be assumed to act on a width equal to 1.50 times the pier diameter, to take advantage of edge effects.

The bottoms of the pier excavations should be substantially free of loose cuttings and soil slough, and tamped, prior to the installation of reinforcing steel and the placement of concrete. In addition, any significant amounts of accumulated water in the pier excavations should be pumped out prior to placing concrete or displaced using the tremie method when placing concrete. The Geotechnical Engineer, or approved representative, should observe the pier excavations to evaluate whether the piers are founded in the supportive material and whether the pier excavations are properly prepared. The pier depths recommended above may require adjustment, if differing conditions are encountered during excavation. Pier excavations should be filled with concrete as soon as practical after drilling to minimize the potential for caving. If temporary casing is used, we recommend its removal from the hole as concrete is being placed. The bottom of the casing should be maintained below the top of the concrete during casing withdrawal and concrete placement. The casing should not be withdrawn until sufficient quantities of concrete have been placed into the excavation to balance the groundwater head outside the casing if groundwater is encountered.

Proposed Elevator

We understand an exterior elevator is proposed east of the bleachers and stadium press box. Based on observations of exposed soil conditions in the storage areas below the press box, we anticipate that the elevator pit will be bottomed in medium dense silty sands (Marine Terrace Deposits). The excavation should be checked by a representative of our firm to ensure that the elevator pit is bottomed in the firm native silty sands. Following excavation to the desired depth, scarify and recompact the upper 6 inches of exposed subgrade soils to a minimum of 90 percent relative compaction. At least 6 inches of granular structural fill, such as crushed aggregate base, should be placed and compacted in the floor slab area to a minimum of 90 percent relative compaction.

Footings for the perimeter walls and corner columns of the elevator pit should be sized, embedded, and reinforced to at least the minimums presented in the 2019 CBC. These footings should be designed using an allowable soil bearing pressure of 2,500 psf for dead loads plus live loads. This allowable load may be increased by one-third to account for the short-term effects of wind and/or seismic loading.

In the vicinity of the elevator pit, we assume that the floor of the pit will be several feet below current grade, and the exposed subgrade before placement of base rock will be medium dense silty sand. Assuming scarification and recompaction of the subgrade to 90 percent relative compaction, the design modulus of subgrade reaction is estimated to be 200 pounds per cubic inch (pci).

The resistance to lateral loadings may be calculated using the sum of a friction factor of 0.40 between the bottom of the elevator pit and the granular structural fill, and a passive resistance of 300 pound per cubic foot (pcf) equivalent fluid weight developed between the elevator pit and the adjacent soil.

At-rest earth pressures against the elevator pit perimeter walls can be calculated using an equivalent fluid pressure of 60 pcf. This assumes the walls are back drained to avoid potential hydrostatic pressure build-up.

To control moisture inside the elevator pit, the base of the floor should be waterproofed and wall back-drainage should be installed. The floor slab of the elevator pit should be underlain by a moisture/vapor barrier manufactured for the purpose, at least 10 mils in thickness. The membrane should be taped at joints. Back drainage can be achieved by placing a perforated pipe/drain rock back-drain system behind the wall, with the drainpipe at the bottom of the wall, and with the drain rock extending up to within 18 inches of finished grade. This back-drain system should be encased in filter fabric and have a gravity drainage outlet. If gravity drainage is not feasible, then a sump pump should be installed. Drain rock for the elevator pit walls should be free-draining, durable, granular material, with 100 percent passing the 1½-inch sieve, and not over 3 percent passing the No. 10 sieve. Caltrans Class 2 permeable material is acceptable. To avoid excess pressure against the wall, drain rock close to the wall should not be over compacted. For back-drain filter fabric, use 6-ounce per square yard minimum weight, non-woven, geotextile fabric by a reputable manufacturer, specifically designed for allowing water passage while retaining soil materials. Perforated pipe should be durable, and at least 4 inches in minimum diameter. Holes or slots should be matched to surrounding permeable material such that the finer particles do not enter the pipe during or after installation. Backfill consisting of relatively “impermeable” soil, at least 1.5 feet thick should be placed above the permeable drain rock to prevent infiltration of surface water. Alternatively, asphalt or concrete pavement may be substituted for the “impermeable” backfill.

Concrete Slabs-on-Grade

Concrete slabs-on-grade should be supported by firm native soil/rock or engineered fill prepared in accordance with our recommendations for earthwork.

To reduce water vapor transmission upward through floor slabs, concrete slabs-on-grade should be constructed on a minimum 4-inch-thick layer of capillary break material covered with a vapor retarder. The capillary break material should be free-draining, clean gravel or rock, such as, No. 4 by ¾-inch pea gravel or permeable aggregate complying with Caltrans Standard Specification, Section 68, Class 1, Type B Permeable Material. The vapor retarder should be at least 10 mils in thickness and meet the material requirements for Class C vapor retarders presented in ASTM E1745, and should be installed according to ASTM E1643. These installation requirements include overlapping seams by 6 inches, taping seams, and sealing penetrations in the vapor retarder.

The field of moisture vapor transmission is a specialty field, and we suggest that qualified experts be contacted to assist in the design and construction of measures related to moisture transmission through slabs-on-grade.

The American Concrete Institute (ACI) Committee document "Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials" (ACI 302.2R-06) provides guidelines for reducing moisture migration through slabs-on-grade. This document advises that concrete slabs be cast directly on the vapor retarder (ACI 302.2R-06, Section 9.3) and provides guidelines for selecting vapor permeance, tensile strength, and puncture resistance. When casting the slab directly on the vapor retarder, a reduced joint spacing, low shrinkage mix design, or other appropriate measures should be used to control slab curl. The ACI guide also notes that a maximum water-cement ratio of 0.5 has yielded satisfactory performance on many slab-on-grade projects. Water-reducing admixtures may be useful in achieving workability at low water-cement ratios. Control joints should be provided at appropriate intervals to control the location of shrinkage cracks. After proper curing, the slab should be allowed to dry and then should be tested to check that the moisture transmission rate is appropriate for the intended floor covering.

For exterior flatwork and other slabs-on-grade where water vapor transmission through slabs is not a concern, the vapor barrier and capillary break material described in this section may be omitted. However, a minimum of 4 inches of Class 2 Aggregate Base rock, compacted to a minimum of 90 percent relative compaction, should be provided beneath exterior flatwork and other slabs-on-grade where vapor transmission is not a concern.

It is important that the subgrade be moist and free of desiccation cracks at the time the slab is cast. Recommendations for slab reinforcement, strength, thickness, control, and construction joints, and so on, should be provided by others.

Although cracks in concrete slabs are common and should be expected, the following measures may help to reduce cracking of slabs.

- Slabs should be cast using concrete with a maximum slump of 4 inches or less.
- Add a water reducing agent or plasticizer to the concrete to increase slump while maintaining a low water-cement ratio to reduce concrete shrinkage. (Concrete having a high water-cement ratio is a major cause of concrete cracking.)
- Control joints should be provided at appropriate intervals to control the location of shrinkage cracks.

Retaining Walls

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 “Site Preparation and Grading,” and drainage should be provided behind walls according to the recommendations that follow. Retaining wall foundations should be designed according to the spread footing recommendations above in “Foundations.”

This discussion assumes the proposed retaining walls will be founded in native marine terrace deposits. We recognize the tapered western end of the southern wall at Building R may terminate in fill soils. We recommend the subgrade in this area be evaluated in the field by the Geotechnical Engineer; over-excavation or additional reinforcement may be appropriate.

Conventional Retaining Walls

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 4.

Table 4. Equivalent Fluid Unit Weight (pcf)[a]

Backfill Slope	At-Rest Conditions	Active Conditions
Level	60	40
3H:1V[b]	72	48
2H:1V	97	65

[a] pcf: pounds per cubic foot

[b] H:V: horizontal to vertical

Lateral earth pressures for backfill slopes other than those given above can be estimated by interpolation; backfill slopes should be inclined no steeper than 2H:1V. 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 the backfill 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 (equivalent to an additional 2 feet of backfill) 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.

Surcharge loads on retaining walls resulting from proposed adjacent building foundations parallel to the proposed retaining wall can be approximated by the following expression:

$$\Delta p_h = (4p/\pi)(x^2z/R^4)$$

Where:
Δp_h = the lateral stress on the wall at depth z
p = magnitude of the footing load (lbs/ft)
x = centerline distance from the footing load to the wall
z = depth below surface
$R^4 = x^4 + z^4$ = the radius from the location on the wall where Δp is, measured to the footing load on the surface

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 19 pcf for level backfill, 27 pcf for back slopes of 3H:1V, and 55 pcf for back slopes of 2H: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) with an 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.

Tied-Back Walls

Permanent tied-back shotcrete or concrete walls may also be considered as an alternative to conventional retaining walls. Tiebacks should extend into the slope at approximately 15 degrees from horizontal. The tiebacks should consist of high strength, prestressed strands or threaded bars, grouted in 4-inch minimum diameter holes drilled into firm soils. Where permanent tiebacks are required, the tiebacks should be double corrosion protected. The downward component of the tiebacks should be supported by the concrete retaining wall. The portion of the tiebacks extending into firm soil beyond the no-load (unbonded) zone should be preliminarily designed using an allowable bond stress of 10 psi for static loading conditions. This value includes an estimated factor of safety of 2. For seismic loading conditions, increase the allowable bond stress value by one-third from those for static loading conditions. The no-load zone is defined by a projection originating a distance of H/5 from the base of the wall and extending up at 60 degrees from horizontal. Allowable capacities of the tiebacks will depend on the drilling method, hole diameter, grout pressure, and work quality. The load-carrying capacity of tiebacks installed in cohesionless soils may be adversely affected if the overburden over the bond length is less than 15 feet.

The contractor should confirm the required bond length by performing load tests under the Geotechnical Engineer’s oversight. We recommend the tiebacks be performance tested to 150 percent of the design load and locked off at 90 percent of the design load. Testing should conform to the latest Post-Tensioning Institute’s (PTI) procedures.

During testing, the contractor should measure the deflection of each tieback with a free-standing, tripod-mounted dial gauge. Load increments should be as follows (DL = design load from the Structural Engineer): 0.1 DL, 0.25 DL, 0.5 DL, 0.75 DL, 1.00 DL, and 1.33 DL, while measuring the deflection at each increment. The contractor should hold the maximum test load for at least 10 minutes and make measurements at 1, 2, 3, 4, 5, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is more than 0.04 inches, then the Contractor should hold the load for an additional 50 minutes with measurements made at 20, 30, 40, 50, and 60 minutes. The readings should be plotted on a load versus elongation chart to aid in evaluating the adequacy of the tiebacks. If the creep rate at a load of 1.33 DL exceeds 0.08 inches between the 6- and 60-minute reading, the tieback design loading should be re-evaluated by the project Geotechnical Engineer. Testing should be completed by the Contractor under the observation of the project Geotechnical Engineer.

The back of the wall face should be fully drained using a prefabricated wall drain system. Tiebacks should be designed and constructed in accordance with the latest edition of the PTI specifications.

Asphalt and Concrete Pavements

Pavement construction should conform to the requirements of the Caltrans Standard Specifications, latest edition. Recommendations for both asphalt concrete and Portland cement concrete pavements are given in this section.

Our recommendations for flexible asphalt pavement sections in the proposed parking lot and track replacement is based on an R-Value of 40, assumed Traffic Indices (T.I.) of 6.0 for the heavily traveled driveway and truck service areas in the small parking lot and track replacement, 4.5 for automobile parking areas, and the California Standard Procedure for Flexible Asphalt Pavement. Based on a T.I. of 6.0 for driveway and truck service areas, and the running track replacement, which will serve as an emergency vehicle access for fire trucks, we recommend an asphalt pavement section consisting of 0.25 feet of asphalt concrete overlying 0.50 feet of Class 2 Aggregate base. Using a T.I. of 4.5 for automobile parking areas, we recommend an asphalt pavement section of 0.2 feet of asphalt concrete overlying 0.35 feet of Class 2 Aggregate Base. Concrete aprons should be considered adjacent to debris boxes. The additional strength would significantly reduce future maintenance.

Aggregate used for asphalt concrete surfacing should conform to the grading specified in Section 39 for 9.5 millimeters (mm) or 12.5 mm (⅜ inch or ½ inch) maximum, medium grading. Asphalt concrete surfacing should be placed in a single lift. Base rock aggregate should comply with the minimum requirements for Class 2 Aggregate Base rock specified in Caltrans Standard Specifications Section 26 and should be compacted to at least 95 percent relative compaction; crushed, angular material should have at least 50 percent of the material (as determined by the material's dry weight) containing a minimum of two fractured faces.

We recommend that exterior concrete pavements consist of at least 6 inches of aggregate base rock beneath at least 6 inches of concrete. For durability and wear resistance, all Portland cement concrete pavements should have a minimum compressive strength of 4,000 psi. A modulus of subgrade reaction, k_v (30-inch circular plate) of 200 psi may be used for design of Portland cement concrete pavements.

Paved areas should be sloped and adequately drained to prevent surface water or subsurface seepage from saturating the pavement subgrade soil. All curbs surrounding landscape areas should be embedded at least 6 inches into the soil subgrade to minimize the migration of water beneath pavements.

Additional Services

We suggest that communications be maintained during the design phase between the design team and SHN to optimize compatibility between the design and soil conditions. For this reason, we recommend that SHN be given the opportunity to review the geotechnical elements of project grading, and foundation plans and specifications to check that the intent of our recommendations have been incorporated into these project documents. If SHN does not review the geotechnical elements of the plans and specifications, the reviewing Geotechnical Engineer should thoroughly review this report and should agree with its conclusions and recommendations or otherwise provide alternative recommendations. Furthermore, if another geotechnical consultant is retained for follow-up services to this report, SHN will at that time cease to be the Geotechnical Engineer-of-Record. SHN cannot assume responsibility or liability for the adequacy of our geotechnical recommendations unless SHN is retained to observe the soil-related portions of the construction.

We recommend that SHN be retained during the construction phase to verify the implementation of our recommendations related to earthwork and to perform the following tasks:

1. Monitor site clearing, including removal of loose fill material, and any other unsuitable material if it is determined that this is required.
2. Monitor over-excavation and subgrade preparation.
3. Observe and test placement of engineered fill, including geogrid-reinforced engineered fill and backfill.
4. Observe foundation excavations.
5. Observe back-drainage construction for retaining walls.
6. Observe and test subgrade and placement and compaction of aggregate base in pavement areas.

This construction phase monitoring is important, because it provides the stakeholders 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.

Limitations

The recommendations provided in this report are based on the assumption that SHN will be retained to provide the construction monitoring described above in order to evaluate compliance with our recommendations. The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man, on this or adjacent properties. In addition, changes in applicable standards of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years. In addition, this report should not be used and is not applicable for any property other than that evaluated.

References

American Society of Civil Engineers. (2017). "ASCE 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Structures." Reston, VA:ASCE.

ASTM-International. (NR). Annual Books of ASTM Standards (Revision issued annually). Conshohocken, PA:ASTM-International.

Atwater, B.F., Musmi-Rokkaku, S. Satake, K., Tsuji, Y. Ueda, K. and Tamaguchi, K.K. (2005). The orphan tsunami of 1700 – Japanese clues to a parent earthquake in North America (Second Edition). University of Washington Press and U.S. Geological Survey Professional Paper 1707.

Blake, MR., A.S. Jayko, , and R.J. McLaughlin. (1985). "Tectonostratigraphic Terranes of the Northern Coast Ranges, California," *in* D.G. Howell (editor), Tectonostratigraphic Terranes of the Circum-

Pacific Region: Circum-Pacific Council for Energy and Mineral Resources Earth Science Series 1. P. 159-171. Reston, VA Circum-Pacific Council.

Boulanger, R.W. and Idriss, I.M. (2014). “CPT and SPT Based Liquefaction Triggering Procedures.” Davis, CA:Civil and Environmental Engineering College of Engineering, University of California at Davis.

California Building Standards Commission. (2020). “2019 California Building Code.” California Code of Regulations, Title 24, Part 2, Volume 2 of 2, based on the 2018 International Building Code. Sacramento, CA:CBSC.

California Department of Conservation, Division of Mines and Geology/United States Geological Survey (CDMG/USGS). (1996). “Probabilistic Seismic Hazard Assessment for the State of California.” DMG Open-File Report 96-08, USGS Open-File Report 96-706. Sacramento, CA:CDMG.

California State Transportation Agency, Department of Transportation. (2018). “Standard Specifications.” Sacramento, CA:Caltrans.

Carver, G.A., and R.M. Burke. (1992). “Late Cenozoic Deformation on the Cascadia Subduction Zone in the Region of the Mendocino Triple Junction”: *in* Burke, R.M. and Carver, G.A. (editors), 1992 Friends of the Pleistocene Guidebook, Pacific Cell, p. 31-63. NR:NR.

Clarke, Samuel H., Jr. (1992). “Geology of the Eel River Basin and Adjacent Region: Implications for Late Cenozoic Tectonics of the Southern Cascadia Subduction Zone and Mendocino Triple Junction.” *The American Association of Petroleum Geologists Bulletin. 76:2,* pp. 199-224. Alexandria, VA:AAPG.

Dengler, L., R. McPherson, and G.A. Carver. (1992). “Historic Seismicity and Potential Source Areas of Large Earthquakes In North Coast California,” in Burke, R.M. and G.A. Carver, (Eds), *Guidebook for the 1992 Friends of the Pleistocene Field Trip, Pacific Cell,* p. 112-118. NR:NR.

Earth Sciences Associates. (1975). “Geology of the Humboldt Bay Region with Special Reference to the Geology of the Humboldt Bay Power Plant Site and Vicinity:” Unpublished report prepared for the Pacific Gas and Electric Company.

Federal Highway Administration. (2011). LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Report No. FHWA-NHI-11-032.

Geomatrix Consultants. (1994). “Seismic Ground Motion Study for Humboldt Bay Bridges on Route 255.” Unpublished consultants report for the California Department of Transportation. NR:Geomatrix.

Hart, E.W. and W.A. Bryant. (1997). *Fault-Rupture Zones in California, Alquist-Priolo Earthquake Fault Zoning Act* (with Index to Earthquake Fault Zone Maps, California Division of Mines and Geology Special Publication 42, with Supplements 1 and 2 added in 1999, 38). NR:NR.

Ishihara, K. and Yoshimine, M. (1992). “Evaluation of settlements in sand deposits following liquefaction during earthquakes.” Soils and Foundations, 32, 173-88. NR:Japanese Geotechnical Society.

Jennings, C.W. (1994). “Fault Activity Map of California and Adjacent Areas, with Locations and Ages of Recent Volcanic Eruptions,” Scale 1:750,000, California Division of Mines and Geology Geologic Data Map No. 6. Sacramento, CA:CDMG.

Kramer, Steven L. (1996) Geotechnical Earthquake Engineering, (Prentice-Hall International Series in Civil Engineering and Engineering Mechanics). Upper Saddle River, NJ:Prentice Hall.

McCulloch, D.S., A.T. Long, and P.A. Utter. (1977). *Acoustic Profiles Offshore Humboldt Bay, California. United States Geological Survey Open-File Report 77-667*. NR:USGS.

McLaughlin, R.J., and 7 others (2000). "Geology of the Cape Mendocino, Eureka, Garberville, and Southwestern part of the hayfork 30 x 60 Minute Quadrangles and Adjacent Offshore Area, Northern California," *U.S. Geological Survey Miscellaneous Field Studies MF-2336*. NR: USGS.

McPherson, R.C. (1992). "Style of Faulting at the Southern End of the Cascadia Subduction Zone": *in* Burke, R.M. and Carver, G.A. (editors), *1992 Friends of the Pleistocene Guidebook, Pacific Cell,* p. 97-111. NR:NR.

Nelson, A.R. and 9 others (2021). "A maximum rupture model for the central and southern Cascadia subduction zone-reassessing ages for coastal evidence of megathrust earthquakes and tsunamis." Quaternary Science Reviews 261 (2021) 106922.

Ogle, B. A. (1953). "Geology of the Eel River Valley Area, Humboldt County, California," *California Department of Natural Resources, Division of Mines, Bulletin 164*. 128p Sacramento, CA:CDMG.

SHN. (2004). "Geologic Hazard and Geotechnical Report: Planned Woodshop Building, Concession Stand and Restroom, Gymnasium, and Single-Story Classroom, Eureka High School, Eureka, CA." 31 p. with Appendices.

---. (2016). "Geologic Hazard and Geotechnical Report: Jay Willard Gymnasium Seismic Retrofit or Replacement, Eureka High School, Eureka, CA." 23 p. with Appendices.

---. (2018). Geologic Hazard and Geotechnical Report: Proposed New Agricultural Building, Eureka High School, Eureka, CA." 22 p. with Appendices.

Structural Engineers Association of California. (NR). "Seismic Design Maps." NR:SEAOC. Accessed November 2020 at: https://seismicmaps.org

Tokimatsu, K. and H.B. Seed. (1987). "Evaluation of settlements in sand due to earthquake shaking." Journal of Geotechnical Engineering, Vol. 113, No. 8.Reston, VA:ASCE.

United States Geological Survey. (2014). interactive deaggregations web application "Dynamic: Conterminous U.S. 2014 (update) v4.2.0." NR:USGS.

---. (2008). United States Quaternary Faults and Fold Database. NR:USGS.

---. (NR). "Preliminary Determinations of Epicenters Catalog." Accessed September 2021 at http://earthquake.usgs.gov/earthquakes/eqarchives/epic/.

Wills, C. J. (1990). "Little Salmon and Related Faults." California Division of Mines and Geology Fault Evaluation Report FER-215 pp 16. Sacramento, CA:CDMG.

Woodward-Clyde Consultants. (1980). "Evaluation of the Potential for Resolving the Geologic and Seismic Issues at the Humboldt Bay Power Plant Unit No. 3." Unpublished consultants report for Pacific Gas and Electric. NR:Woodward-Clyde.

Youd, T.L., and S.N. Hoose. (1978). *Geological Survey Professional Paper Number 993: Historic Ground Failures in Northern California Triggered by Earthquakes*. Washington, D.C.: United States Government Printing Office.

United States Occupational Health and Safety Administration. (NR). "OSHA Excavation and Trench Safety Standards 29 CFR Part 1926, Title 29—Labor, Subtitle B--Regulations Relating to Labor, Chapter XVII--Occupational Safety and Health Administration, Department of Labor, Part 1926--Safety and Health Regulations for Construction." Washington, D.C.:OSHA.

Boring Logs

1

BORING NUMBER B-01-20

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/28/20 **COMPLETED** 7/28/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers/ Mud Rotary
AT TIME OF DRILLING 6.50 ft
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with cement grout
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SILTY SAND (SM), moist, dark brown, fine to medium sand, few fine subrounded gravel, slightly cohesive (FILL).										
		SILTY SAND (SM), loose, moist to wet, dark brown, fine to medium sand, organic rich (wood and roots) (FILL).	MC		7-5-5 (10)		107	15				
			SPT		1-4-6 (10)							
5		Grades brownish-gray to gray.										
		SILTY SAND (SM), medium dense, wet, bluish-gray, fine to medium sand, non-cemented, some iron staining (MARINE TERRACE DEPOSITS).	MC		6-9-13 (22)		116	15				
			SPT		7-7-5 (12)							16
10												
		Reddish-brown.	MC		9-13-17 (30)							17
		Grades grayish-brown.	SPT		4-8-10 (18)							
15												
			MC		3-5-7 (12)							
		SANDY LEAN CLAY (CL), very stiff, moist, dark bluish-gray, low to medium plasticity, very fine sand (PRE-TERRACE MUD).	MC		6-7-10 (17)							
20			SPT		6				31	22	9	
			SPT		7-8	1.75						95
25												
		LEAN CLAY WITH SAND (CL), stiff, moist, dark bluish-gray, medium plasticity, fine sand (PRE-TERRACE MUD).	MC		6-8-10 (18)	2.25						
30												

(Continued Next Page)

CLIENT Eureka City Schools District **PROJECT NAME** Albee Stadium, Eureka High School

PROJECT NUMBER 020010.100 **PROJECT LOCATION** Eureka, California

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
30		LEAN CLAY WITH SAND (CL), stiff, moist, dark bluish-gray, medium plasticity, fine sand (PRE-TERRACE MUD). *(continued)* Contains shells.	SPT		4-4-5 (9)							
35			MC		6-7-12 (19)	2.0						
40		POORLY GRADED SAND WITH SILT (SP-SM), very dense, moist, dark bluish-gray, fine to medium sand, non-cemented (HOOKTON FORMATION).	SPT		33-34-33 (67)							
45			MC		40-50/5"							7
50		Abundant shells.	SPT		8-8-8 (16)							

Bottom of borehole at 51.5 feet.

BORING NUMBER B-02-20

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/28/20 **COMPLETED** 7/28/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers
▽ **AT TIME OF DRILLING** 5.00 ft
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
▼ **AT END OF DRILLING** ---
NOTES Backfilled with cement grout
▼ **AFTER DRILLING** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SILTY SAND (SM), medium dense, moist, brown, fine to medium sand, weak cementation (FILL).										
			MC		14-11-9 (20)		101	10				
		SILTY SAND (SM), loose, moist, dark brown (NATIVE).	SPT		3-3-3 (6)							47
5		▽ SILTY SAND (SM), loose to medium dense, mottled gray and brown, non-cemented, <25% fines (MARINE TERRACE DEPOSITS). Becomes wet.	MC		3-4-6 (10)		97	25				
			SPT		3-3-6 (9)							20
10		SILTY SAND (SM), medium dense, wet, reddish-brown, fine to medium sand, weak cementation (MARINE TERRACE DEPOSITS).	MC		7-12-14 (26)		105	21				
			SPT		5-6-6 (12)							17
15			MC		6-7-16 (23)							11
20			SPT		8-13-12 (25)							17
25			SPT		4-7-10 (17)	2.25						10
		LEAN CLAY (CL), stiff, moist, dark bluish-gray, medium plasticity (PRE-TERRACE MUD).										

Bottom of borehole at 26.5 feet.

BORING NUMBER B-03-20

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/28/20 **COMPLETED** 7/28/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers
AT TIME OF DRILLING --- Not Encountered
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with cement grout
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	POORLY GRADED SAND (SP), loose, gray, non-cemented (FILL).										
	SILTY SAND (SM), loose, moist to wet, mottled brown to bluish-gray, non-cemented to weak cementation (FILL).	MC		5-6-5 (11)		106	15				
		SPT		2-4-3 (7)							22
5	Wet.	MC		4-5-4 (9)		106	17				
	Bluish-gray.	SPT		3-2-2 (4)							28
10	SANDY SILT (ML) medium stiff, moist to wet, very dark brown, fine sand, organic rich, roots (COLLUVIUM?).	SPT		2-3-4 (7)							22
	SILTY SAND (SM) loose, wet, brownish-gray, fine to medium sand, non-cemented (MARINE TERRACE DEPOSITS).										
15	LEAN CLAY WITH SAND (CL), medium stiff to stiff, moist, bluish-gray, low to medium plasticity, fine sand, rootlets (PRE-TERRACE MUD).	SPT		2-3-5 (8)							
20	No organics.	SPT		4-5-7 (12)							
25	LEAN CLAY WITH SAND (CL), stiff, moist, dark bluish-gray, low to medium plasticity (PRE-TERRACE MUD).	SPT		5-5-7 (12)							

Bottom of borehole at 26.5 feet.

BORING NUMBER B-04-20

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/29/20 **COMPLETED** 7/29/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers/ Mud Rotary
AT TIME OF DRILLING 11.50 ft
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with cement grout
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SANDY SILT (ML), moist, very dark brown (FILL).										
		SILTY SAND (SM), loose to medium dense, moist, mottled yellowish-brown, weak cementation, fine to medium sand (FILL).	MC		15-17-11 (28)		103	9				
			SPT		5-4-3 (7)							
5			MC		3-3-3 (6)		80	10				
		SILTY SAND (SM), loose, moist, gray, fine sand, non-cemented, 25% fines (ALLUVIUM).	SPT		2-1-2 (3)							
		SILTY SAND (SM), very loose, moist, brownish-gray, non-cemented, 40% fines, charcoal, organics (MARINE TERRACE DEPOSITS).										
10		Saturated, coarse sand, wood fragment.	MC		7-10-10 (20)		89	20				31
			SPT		2-1-2 (3)							
		LEAN CLAY WITH SAND (CL), soft, moist, bluish-gray, medium plasticity (PRE-TERRACE MUD).										
15		Becomes stiff, increase in fine sand.	MC		5-7-13 (20)	>4.5						
20			SPT		4-4-6 (10)							
25		Becomes stiff.	MC		4-5-9 (14)	3.25						
30												

(Continued Next Page)

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTIC LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
30		CLAYEY SAND (SC), very dense, moist to wet, dark bluish-gray, weak cementation, abundant shells (HOOKTON FORMATION).	SPT		7-23-30 (53)							
35			MC		50/5"							
		POORLY GRADED SAND WITH SILT (SP-SM), very dense, moist, dark bluish-gray, fine sand, weak cementation, abundant shells.	SPT		19-35-38 (73)							
40			MC		40-50/5"							
45			SPT		23-40-43 (83)							
50		SILTY SAND (SM), very dense, wet, dark bluish-gray, fine sand, weak cementation, abundant shells.	SPT		29-28-33 (61)							

Bottom of borehole at 51.5 feet.

BORING NUMBER B-05-20

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/29/20 **COMPLETED** 7/29/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers
AT TIME OF DRILLING 20.00 ft
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with cement grout
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SILTY SAND (SM), loose to medium dense, moist, brown, fine sand, non-cemented (FILL).										
			MC		5-6-6 (12)		89	25				
			SPT		3-3-5 (8)							
5			MC		6-8-11 (19)		90	13				
		POORLY GRADED SAND WITH SILT (SP-SM), loose, moist to wet, gray, fine to medium sand, non-cemented, some wood fragments (FILL).	SPT		5-5-5 (10)	1.0						
		LEAN CLAY (CL), medium stiff, moist, bluish-gray, low plasticity (PRE-TERRACE MUD).										
10		LEAN CLAY WITH SAND (CL), stiff, moist, bluish-gray, low to medium plasticity, fine sand (PRE-TERRACE MUD).	MC		2-5-9 (14)	3.25						
			SPT		3-4-6 (10)							
15		Very Stiff.	MC		7-11-14 (25)	4			26	21	5	80
20			SPT		5-6-9 (15)							
25		Very Stiff.	SPT		6-9-12 (21)							
		SANDY LEAN CLAY (CL), stiff, moist, dark bluish-gray, low plasticity, fine sand, shells (PRE-TERRACE MUD).										

Bottom of borehole at 26.5 feet.

BORING NUMBER B-06-20

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/29/20 **COMPLETED** 7/29/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers/ Hollow Stem Augers
AT TIME OF DRILLING 1.00 ft Perched water at 1 ft, dry below 6.5 ft.
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with cement grout
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); loose, 3/4-inch diameter subrounded gravel, wet (FILL).										
	WELL GRADED SAND WITH SILT AND GRAVEL (SW-SM), medium dense, wet, gray, non-cemented (FILL).	SPT		5-8-10 (18)							9
	SILTY SAND (SM), medium dense, wet, bluish-gray, fine sand, non-cemented (MARINE TERRACE DEPOSITS).										
5	Some coarse rounded sand.	MC		10-13-18 (31)		112	17				
	SANDY LEAN CLAY (CL), stiff, moist, bluish-gray, low to medium plasticity, fine sand (PRE-TERRACE MUD).	SPT		2-3-6 (9)							6
	LEAN CLAY (CL), stiff, moist, bluish-gray, low to medium plasticity (PRE-TERRACE MUD).	MC		3-6-8 (14)	2	97	25				
10		SPT		4-6-10 (16)							
15	SANDY LEAN CLAY (CL), stiff, moist, bluish-gray, fine sand, low to medium plasticity (PRE-TERRACE MUD).	SPT		4-6-8 (14)	3.25						
20		MC		5-6-12 (18)	3.5						
25	Increase in sand content (40%).	SPT		5-6-9 (15)							

Bottom of borehole at 26.5 feet.

BORING NUMBER B-07-20

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/30/20 **COMPLETED** 7/30/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers
∇ **AT TIME OF DRILLING** 11.00 ft
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
▼ **AT END OF DRILLING** ---
NOTES Backfilled with cement grout
▼ **AFTER DRILLING** ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	SILTY SAND (SM), moist, brown (FILL).										
	CLAYEY SAND (SC), loose, moist, yellowish-brown, non-cemented, fine to medium sand (FILL).	MC		4-4-3 (7)		98	19				
		SPT		1-0-1 (1)							
5	SILTY SAND (SM), very loose, moist, brown, fine to medium sand, non-cemented (FILL).	MC		2-1-1 (2)		90	15				
		SPT		1-1-1 (2)							24
10	CLAYEY SAND (SC), very loose, moist to wet, gray, cohesive, fine to medium sand (FILL). ∇	MC		1-1-2 (3)		104	20				35
		SPT		1-1-1 (2)							42
15	SILTY SAND (SM), very loose, wet, grayish-brown.	MC		3-1-1 (2)							
	PEAT (PT), very soft, wet, dark brown, organic rich SANDY SILT.										
20	SILTY SAND (SM), loose, moist to wet, dark gray, fine to medium sand, few organics (ALLUVIUM/MARINE TERRACE DEPOSITS?).	SPT		3-3-3 (6)							28
25	CLAYEY SAND (SC), loose to medium dense, wet, grayish-brown, slightly cohesive, fine to medium sand (MARINE TERRACE DEPOSITS).	SPT		5-3-7 (10)							23

Bottom of borehole at 26.5 feet.

BORING NUMBER B-08-20

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/30/20 **COMPLETED** 7/30/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers/ Mud Rotary
AT TIME OF DRILLING 7.50 ft
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with cement grout
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0	POORLY GRADED SAND (SP), loose, brownish-gray (FILL).										
	Medium dense, moist, grayish-brown, non-cemented.	MC		7-10-11 (21)		99	5				
		SPT		3-2-2 (4)							
5		MC		4-4-4 (8)		95	5				
	Becomes wet.	SPT		1-3-4 (7)							4
	SILTY SAND (SM), loose, moist, dark brown, weak cementation (FILL).										
10	Gray and brown (mixed), strongly cemented, red nodules.	MC		5-5-7 (12)							
	POORLY GRADED SAND WITH SILT (SP-SM), medium dense, moist to wet, dark gray, charcoal fragments (FILL).	SPT		6-9-5 (14)							7
	CLAYEY SAND WITH GRAVEL (SC), medium dense, moist, dark gray, fractured rock broken by sampler (FILL).										
15	WELL GRADED SAND WITH SILT (SW-SM), loose, moist, dark brown, fine to medium sand, some wood fragments (FILL).	SPT		2-3-4 (7)							11
	POORLY GRADED SAND WITH SILT (SP-SM), loose, moist to wet, gray, non-cemented (FILL).										
20	SILTY SAND (SM), loose, wet, gray, non-cemented (FILL).	MC		2-3-7 (10)							12
	PEAT (PT), soft to medium stiff, wet, dark brown, organic rich SANDY SILT (NATIVE).										
25	WELL GRADED SAND WITH SILT (SW-SM), loose, wet, gray, non-cemented, rootlets and organics (MARINE TERRACE DEPOSITS).	SPT		4-3-2 (5)							8
30											

(Continued Next Page)

BORING NUMBER B-08-20

CLIENT Eureka City Schools District **PROJECT NAME** Albee Stadium, Eureka High School

PROJECT NUMBER 020010.100 **PROJECT LOCATION** Eureka, California

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
30		CLAYEY SAND (SC), medium dense, wet, gray, cohesive (FILL).	MC		11-17-11 (28)							47
		Increase in fines.										
35		SANDY LEAN CLAY (CL), stiff, moist, bluish-gray, fine to medium sand, low plasticity, organics (PRE-TERRACE MUD).	SPT		3-4-7 (11)				34	19	15	78
40		Very stiff.	MC		6-10-17 (27)	3.0						
45		SILTY SAND (SM), dense, moist to wet, bluish-gray, non-cemented, fine to medium sand, 15% fines (HOOKTON FORMATION).	SPT		11-15-19 (34)							
50		Very dense.	SPT		23-27-36 (63)							

Bottom of borehole at 51.5 feet.

BORING NUMBER B-09-20

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/30/20 **COMPLETED** 7/30/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Taber Drilling
GROUNDWATER DEPTH
DRILLING METHOD Solid Flight Augers
AT TIME OF DRILLING 10.00 ft
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with cement grout
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	POORLY GRADED SAND (SP), loose, gray (FILL).										
	Medium dense, wet, gray, non-cemented, fine to medium sand.	MC		5-8-10 (18)		102	19				
	SILTY SAND (SM), loose, moist, dark brown, fine sand, weak cementation, 40% fines.	SPT		3-2-3 (5)							
5		MC		2-2-2 (4)		90	26				
		SPT		1-1-2 (3)							
10		MC		2-1-3 (4)							
	POORLY GRADED SAND WITH SILT (SP-SM), medium dense, wet, yellowish-brown, non-cemented, 15% fines (MARINE TERRACE DEPOSITS).	SPT		3-4-6 (10)							9
15	LEAN CLAY WITH SAND (CL), stiff, moist, brown, low plasticity, fine sand (PRE-TERRACE MUD). Grades to bluish-gray.	SPT		5-4-5 (9)	1.0						
20		SPT		5-5-7 (12)							
25	Very stiff.	SPT		6-8-11 (19)							

Bottom of borehole at 26.5 feet.

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 8/13/20 **COMPLETED** 8/13/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR
GROUNDWATER DEPTH
DRILLING METHOD Hand Auger
AT TIME OF DRILLING --- Not Encountered
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with soil cuttings
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0		SANDY SILT (ML), medium stiff, wet, brown, non-cemented, medium sand, organic rich, 45% sand (FILL). Grades gray.										
		SILTY SAND (SM), loose, wet, gluish-gray, non-cemented, fine to medium sand, few organics (FILL). Becomes mottled, iron-oxide nodules.										
		SILT WITH SAND (ML), medium stiff, very dark brown, fine sand, organic rich, wood fragments.										
2.5		PEAT (PT), very soft, moist, very dark brown, mostly organics/wood fragments/wood waste.										
		CLAYEY SAND (SC), medium dense, moist, bluish-gray, weakly cemented, slightly cohesive (MARINE TERRACE DEPOSITS).	GB					18				27
5.0		Becomes wet.										
		SANDY LEAN CLAY (CL), medium stiff to stiff, bluish-gray (MARINE TERRACE DEPOSITS).	GB					24				37
		POORLY GRADED SAND WITH SILT (SP-SM), medium dense, wet, bluish-gray, non-cemented, medium sand (MARINE TERRACE DEPOSITS).	GB					19				13
7.5												

Bottom of borehole at 8.5 feet.

BORING NUMBER HB-02-20

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 8/13/20 **COMPLETED** 8/13/20
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR
GROUNDWATER DEPTH
DRILLING METHOD Hand Auger
AT TIME OF DRILLING --- Saturated conditions at 5'
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with soil cuttings
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0	SANDY SILT (ML), soft, wet, dark brown, organic rich (FILL).										
	SANDY LEAN CLAY/SILTY SAND (CL/SM), medium stiff/loose, wet, dark gray, non-cemented, fine to coarse subrounded gravel, organics (FILL).										
	SILTY SAND WITH GRAVEL(SM), very loose, wet, dark gray, medium sand, non-cemented, fine to coarse subrounded gravel, organic rich (FILL).										
2.5											
	SILTY GRAVEL WITH SAND (GM), very loose, wet, brown, fine to coarse subrounded gravel, organic rich (FILL).										
	CLAYEY SAND (SC), medium dense, moist to wet, bluish-gray, weakly cemented, slightly cohesive (MARINE TERRACE DEPOSITS).	GB									
5.0	POORLY GRADED SAND WITH SILT (SP-SM), loose to medium dense, wet, bluish-gray, non-cemented, medium sand (MARINE TERRACE DEPOSITS).										
		GB					25				8
7.5											

Bottom of borehole at 8.0 feet.

BORING NUMBER B-01-21

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/12/21 **COMPLETED** 7/12/21
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Clear Heart Drilling
GROUNDWATER DEPTH
DRILLING METHOD Rotary Hollow Stem Auger
AT TIME OF DRILLING 18.50 ft
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with cement grout
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SILTY SAND (SM), loose to medium dense, moist, dark brown, fine to medium sand, weak cementation, slightly cohesive, organics (FILL).										
		SILTY SAND (SM), medium dense, moist, brownish-gray, fine to medium sand, weak cementation, slightly cohesive (MARINE TERRACE DEPOSITS). Grades bluish-gray at 3.5 feet; becomes saturated.										
5		TXUU @ 6-6.5 feet; See Appendix 2.	MCS		9-10-14 (24)		97	32				9
			SPT		5-8-12 (20)							
10		LEAN CLAY with SAND (CL), medium stiff to stiff, moist, bluish-gray, fine sand, low plasticity, silty (PRE-TERRACE MUD). Unconfined Compression Test @ 11-11.5 feet = 2503 psf; See Appendix 2.	MCS		10-8-12 (20)	3.0	98	26				
			SPT		4-3-7 (10)							
15			SPT		4-6-7 (13)	2.0						
20			SPT		5-8-11 (19)	1.5						

Bottom of borehole at 21.5 feet.

BORING NUMBER B-02-21

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/13/21 **COMPLETED** 7/13/21
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Clear Heart Drilling
GROUNDWATER DEPTH
DRILLING METHOD Rotary Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with cement grout
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SILTY SAND (SM), loose to medium dense, dry to moist, brownish-gray, fine to medium sand, non-cemented (FILL).										
		SANDY SILT (ML), soft, moist, dark brown, fine sand, non-plastic, organics (TOP SOIL).										
5		SILTY SAND (SM), loose, moist, dark brown, fine sand, non-cemented, few organics (ALLUVIUM).	SPT		1-2-5 (7)							
		SILTY SAND (SM), medium dense, moist, brownish-gray, fine to medium sand, weak cementation, slightly cohesive (MARINE TERRACE DEPOSITS). Grades bluish-gray @ 8.5 feet.	SPT		2-4-7 (11)							
10		LEAN CLAY with SAND (CL), medium stiff, moist, bluish-gray, fine sand, low plasticity, cohesive, silty (ALLUVIUM). Increase in sand content at 11.25 feet (PRE-TERRACE MUD).	SPT		5-2-4 (6)	1.5						
15			SPT		2-3-5 (8)	1.5						

Bottom of borehole at 16.5 feet.

BORING NUMBER B-03-21

CLIENT Eureka City Schools District
PROJECT NAME Albee Stadium, Eureka High School
PROJECT NUMBER 020010.100
PROJECT LOCATION Eureka, California
DATE STARTED 7/13/21 **COMPLETED** 7/13/21
GROUND ELEVATION **HOLE SIZE**
DRILLING CONTRACTOR Clear Heart Drilling
GROUNDWATER DEPTH
DRILLING METHOD Rotary Hollow Stem Auger
AT TIME OF DRILLING 3.00 ft
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
AT END OF DRILLING ---
NOTES Backfilled with cement grout
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SILTY SAND (SM), loose to medium dense, moist to wet, bluish-gray to reddish-brown, fine to medium sand, non-cemented (FILL).										
3		LEAN CLAY with SAND (CL), soft to medium stiff, moist, bluish-gray, fine sand, low plasticity, cohesive (PRE-TERRACE MUD).										
5			SPT		3-5-7 (12)	2.5						
			SPT		3-4-5 (9)							
10			SPT		3-3-4 (7)	1.25						

Bottom of borehole at 11.5 feet.

Laboratory Test Data 2

COOPER TESTING LABORATORY

Corrosivity Tests Summary

CTL # 054-182 **Date:** 11/9/2020 **Tested By:** JC **Checked:** PJ

Client: SHN **Project:** Eureka High School Albee Stadium **Proj. No:** 020010.100

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	
Bulk	-	-	-	-	9,684	14	158	0.0158	5.3	-	-	-	16.3	Dark Grayish Brown Silty SAND

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	EHS Albee Stadium	Project Number:	020010.100
Performed By:	ESP	Date:	9/11/2020
Checked By:	NAN	Date:	9/11/2020
Project Manager:	GSW		

Lab Sample Number	20-812	20-815	20-818	20-832	20-835
Boring Label	B01-20	B01-20	B02-20	B02-20	B02-20
Sample Depth (ft)	2.5-3	6-6.5	11-11.5	3-3.5	6-6.5
Diameter of Cylinder, in	2.42	2.42	2.42	2.42	2.42
Total Length of Cylinder, in.	5.96	6.00	6.00	6.00	6.00
Length of Empty Cylinder A, in.	0.00	0.00	0.25	0.55	0.00
Length of Empty Cylinder B, in.	1.35	0.41	0.28	0.00	0.78
Length of Cylinder Filled, in	4.61	5.59	5.47	5.45	5.22
Volume of Sample, in^3	21.20	25.71	25.16	25.07	24.01
Volume of Sample, cc.	347.47	421.34	412.29	410.79	393.45

Pan #	S8	S26	S22	S27	S25
Weight of Wet Soil and Pan	841.5	1062.1	985.7	883.0	904.4
Weight of Dry Soil and Pan	753.0	948.1	841.3	818.2	755.0
Weight of Water	88.5	114.0	144.4	64.8	149.4
Weight of Pan	159.8	164.1	150.9	152.2	145.1
Weight of Dry Soil	593.2	784.0	690.4	666.0	609.9
Percent Moisture	14.9	14.5	20.9	9.7	24.5
Dry Density, g/cc	1.71	1.86	1.67	1.62	1.55
Dry Density, lb/ft^3	106.6	116.2	104.5	101.2	96.8

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	EHS Albee Stadium	Project Number:	020010.100
Performed By:	ESP	Date:	9/11/2020
Checked By:	NAN	Date:	9/11/2020
Project Manager:	GSW		

Lab Sample Number	20-846	20-849	20-858	20-861	20-863
Boring Label	B03-20	B03-20	B04-20	B04-20	B04-20
Sample Depth (ft)	3-3.5	6-6.5	3-3.5	6-6.5	11-11.5
Diameter of Cylinder, in	2.42	2.42	2.42	2.42	2.42
Total Length of Cylinder, in.	5.95	6.00	6.00	6.00	6.00
Length of Empty Cylinder A, in.	0.00	0.00	0.00	0.00	0.00
Length of Empty Cylinder B, in.	0.85	0.83	0.00	0.00	2.64
Length of Cylinder Filled, in	5.10	5.17	6.00	6.00	3.36
Volume of Sample, in^3	23.46	23.78	27.60	27.60	15.45
Volume of Sample, cc.	384.41	389.68	452.24	452.24	253.26

Pan #	S27	S29	S8	S26	S22
Weight of Wet Soil and Pan	904.9	920.7	965.6	798.7	583.0
Weight of Dry Soil and Pan	806.3	806.1	902.3	742.8	511.2
Weight of Water	98.6	114.6	63.3	55.9	71.8
Weight of Pan	152.1	147.1	159.8	164.4	151.3
Weight of Dry Soil	654.2	659.0	742.5	578.4	359.9
Percent Moisture	15.1	17.4	8.5	9.7	19.9
Dry Density, g/cc	1.70	1.69	1.64	1.28	1.42
Dry Density, lb/ft^3	106.2	105.6	102.5	79.8	88.7

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	EHS Albee Stadium	Project Number:	020010.100
Performed By:	ESP	Date:	9/11/2020
Checked By:	NAN	Date:	9/11/2020
Project Manager:	GSW		

Lab Sample Number	20-877	20-880	20-892	20-894	20-902
Boring Label	B05-20	B05-20	B06-20	B06-20	B07-20
Sample Depth (ft)	3-3.5	6-6.5	5-5.5	8.5-9	3-3.5
Diameter of Cylinder, in	2.42	2.42	2.42	2.42	2.42
Total Length of Cylinder, in.	6.00	6.00	6.00	6.00	6.00
Length of Empty Cylinder A, in.	0.00	0.00	0.00	0.00	0.00
Length of Empty Cylinder B, in.	0.57	1.60	0.63	2.25	0.68
Length of Cylinder Filled, in	5.43	4.40	5.37	3.75	5.32
Volume of Sample, in^3	24.98	20.24	24.70	17.25	24.47
Volume of Sample, cc.	409.28	331.64	404.76	282.65	400.99

Pan #	S25	A7	A6	A12	A5
Weight of Wet Soil and Pan	869.3	623.7	937.6	639.2	831.2
Weight of Dry Soil and Pan	726.8	564.2	813.2	527.9	714.6
Weight of Water	142.5	59.5	124.4	111.3	116.6
Weight of Pan	145.3	86.7	87.5	87.5	86.8
Weight of Dry Soil	581.5	477.5	725.7	440.4	627.8
Percent Moisture	24.5	12.5	17.1	25.3	18.6
Dry Density, g/cc	1.42	1.44	1.79	1.56	1.57
Dry Density, lb/ft^3	88.7	89.9	111.9	97.3	97.7

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	EHS Albee Stadium	Project Number:	020010.100
Performed By:	ESP	Date:	9/11/2020
Checked By:	NAN	Date:	9/11/2020
Project Manager:	GSW		

Lab Sample Number	20-904	20-908	20-917	20-920	20-938
Boring Label	B07-20	B07-20	B08-20	B08-20	B09-20
Sample Depth (ft)	5.5-6	11-11.5	3-3.5	6-6.5	3-3.5
Diameter of Cylinder, in	2.42	2.42	2.42	2.42	2.42
Total Length of Cylinder, in.	5.98	5.98	6.00	6.00	6.00
Length of Empty Cylinder A, in.	0.00	0.00	0.00	0.00	0.00
Length of Empty Cylinder B, in.	2.03	0.97	1.65	0.47	0.82
Length of Cylinder Filled, in	3.95	5.01	4.35	5.53	5.18
Volume of Sample, in^3	18.17	23.04	20.01	25.44	23.83
Volume of Sample, cc.	297.73	377.62	327.88	416.82	390.44

Pan #	A3	SS9	S26	S29	S27
Weight of Wet Soil and Pan	578.6	952.1	707.0	812.1	912.1
Weight of Dry Soil and Pan	515.9	826.8	682.7	782.3	791.5
Weight of Water	62.7	125.3	24.3	29.8	120.6
Weight of Pan	85.3	196.4	164.3	147.1	152.0
Weight of Dry Soil	430.6	630.4	518.4	635.2	639.5
Percent Moisture	14.6	19.9	4.7	4.7	18.9
Dry Density, g/cc	1.45	1.67	1.58	1.52	1.64
Dry Density, lb/ft^3	90.3	104.2	98.7	95.1	102.3

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	EHS Albee Stadium	Project Number:	020010.100
Performed By:	ESP	Date:	9/11/2020
Checked By:	NAN	Date:	9/11/2020
Project Manager:	GSW		

Lab Sample Number	20-941	20-947	20-948	20-949	20-950
Boring Label	B09-20	HB1-20	HB1-20	HB1-20	HB2-20
Sample Depth (ft)	6-6.5	4.25-5	6-6.5	6.5-7	6-6.5
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.	1.22				
Length of Cylinder Filled, in	4.78				
Volume of Sample, in^3	21.99				
Volume of Sample, cc.	360.29				

Pan #	S22	SS12	SS2	SS6	SS14
Weight of Wet Soil and Pan	807.6	732.9	684.6	972.6	654.6
Weight of Dry Soil and Pan	670.3	650.6	590.1	848.4	562.4
Weight of Water	137.3	82.3	94.5	124.2	92.2
Weight of Pan	151.1	194.2	193.4	196.1	192.7
Weight of Dry Soil	519.2	456.4	396.7	652.3	369.7
Percent Moisture	26.4	18.0	23.8	19.0	24.9
Dry Density, g/cc	1.44				
Dry Density, lb/ft^3	90.0				

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name:	EHS Albee Stadium	Project Number:	020010.100
Performed By:	ESP	Date:	9/11/2020
Checked By:	NAN	Date:	9/11/2020
Project Manager:	GSW		

Lab Sample Number	20-816	20-819	20-822	20-829	20-830
Boring Label	B01-20	B01-20	B01-20	B01-20	B01-20
Sample Depth	6.5-8	11.5-13	19.5-21	45-46	50-51.5
Pan Number	SS9	SS14	SS11	SS3	missing
Dry Weight of Soil & Pan	469.5	402.0	390.1	422.9	
Pan Weight	196.6	192.7	192.6	197.1	
Weight of Dry Soil	272.9	209.3	197.5	225.8	
Soil Weight Retained on #200&Pan	425.0	365.9	203.1	406.4	
Soil Weight Passing #200	44.5	36.1	187.0	16.5	
Percent Passing #200	**16.3**	**17.2**	**94.7**	**7.3**	

Lab Sample Number	20-833	20-836	20-839	20-841	20-842
Boring Label	B02-20	B02-20	B02-20	B02-20	B02-20
Sample Depth	3.5-5	6.5-8	11.5-13	16-16.5	20-21.5
Pan Number	SS15	SS7	SS8	SS10	SS1
Dry Weight of Soil & Pan	327.7	366.4	404.3	413.2	430.9
Pan Weight	193.3	193.1	192.8	195.5	194.8
Weight of Dry Soil	134.4	173.3	211.5	217.7	236.1
Soil Weight Retained on #200&Pan	264.6	331.2	369.5	389.0	390.0
Soil Weight Passing #200	63.1	35.2	34.8	24.2	40.9
Percent Passing #200	**46.9**	**20.3**	**16.5**	**11.1**	**17.3**

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name:	EHS Albee Stadium	Project Number:	020010.100
Performed By:	ESP	Date:	9/11/2020
Checked By:	NAN	Date:	9/11/2020
Project Manager:	GSW		

Lab Sample Number	20-843	20-847	20-850	20-853	20-863
Boring Label	B02-20	B03-20	B03-20	B03-20	B04-20
Sample Depth	25-26	3.5-5	6.5-8	11-11.5	11-11.5
Pan Number	SS12	SS6	SS5	SS2	SS5
Dry Weight of Soil & Pan	378.5	394.4	371.5	330.0	416.0
Pan Weight	194.1	196.0	195.5	193.5	195.4
Weight of Dry Soil	184.4	198.4	176.0	136.5	220.6
Soil Weight Retained on #200&Pan	361.0	351.7	321.6	300.0	347.7
Soil Weight Passing #200	17.5	42.7	49.9	30.0	68.3
Percent Passing #200	**9.5**	**21.5**	**28.4**	**22.0**	**31.0**

Lab Sample Number	20-887	20-890	20-893	20-906	20-908
Boring Label	B05-20	B06-20	B06-20	B07-20	B07-20
Sample Depth	16-16.5	2-3.5	5.5-6.5	6.5-8	11-11.5
Pan Number	SS15	SS8	SS7	SS10	SS9
Dry Weight of Soil & Pan	414.8	604.5	378.7	433.1	414.6
Pan Weight	194.5	192.9	193.1	195.4	196.4
Weight of Dry Soil	220.3	411.6	185.6	237.7	218.2
Soil Weight Retained on #200&Pan	238.9	567.5	368.5	376.3	337.8
Soil Weight Passing #200	175.9	37.0	10.2	56.8	76.8
Percent Passing #200	**79.8**	**9.0**	**5.5**	**23.9**	**35.2**

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name:	EHS Albee Stadium	Project Number:	020010.100
Performed By:	ESP	Date:	9/11/2020
Checked By:	NAN	Date:	9/11/2020
Project Manager:	GSW		

Lab Sample Number	20-909	20-914	20-915	20-921	20-924
Boring Label	B07-20	B07-20	B07-20	B08-20	B08-20
Sample Depth	11.5-13	21-21.5	25-26.5	6.5-8	11.5-13
Pan Number	SS3	SS11	SS1	SS12	SS6
Dry Weight of Soil & Pan	411.9	316.9	487.5	427.7	558.7
Pan Weight	197.0	192.6	194.8	195.2	196.0
Weight of Dry Soil	214.9	124.3	292.7	232.5	362.7
Soil Weight Retained on #200&Pan	321.0	281.9	421.5	419.6	534.5
Soil Weight Passing #200	90.9	35.0	66.0	8.1	24.2
Percent Passing #200	**42.3**	**28.2**	**22.5**	**3.5**	**6.7**

Lab Sample Number	20-925	20-927	20-929	20-931	20-932
Boring Label	B08-20	B08-20	B08-20	B08-20	B08-20
Sample Depth	15-16.5	20.5-21	25-26.5	31-31.5	35-36.5
Pan Number	SS2	SS14	SS3	SS11	SS15
Dry Weight of Soil & Pan	481.8	903.1	308.8	427.8	399.0
Pan Weight	193.4	192.7	197.0	192.6	194.3
Weight of Dry Soil	288.4	710.4	111.8	235.2	204.7
Soil Weight Retained on #200&Pan	449.6	816.3	299.8	317.6	239.4
Soil Weight Passing #200	32.2	86.8	9.0	110.2	159.6
Percent Passing #200	**11.2**	**12.2**	**8.1**	**46.9**	**78.0**

Phone: (707) 441-8855 Email: info@shn-engr.com Web: shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name:	EHS Albee Stadium	Project Number:	020010.100
Performed By:	ESP	Date:	9/11/2020
Checked By:	NAN	Date:	9/11/2020
Project Manager:	GSW		

Lab Sample Number	20-943	20-947	20-948	20-949	20-950
Boring Label	B09-20	HB1-20	HB1-20	HB1-20	HB2-20
Sample Depth	11.5-13	4.25-4.5	6-6.5	6.5-7	6-6.5
Pan Number	SS8	SS12	SS2	SS6	SS14
Dry Weight of Soil & Pan	440.5	650.8	590.2	848.5	562.6
Pan Weight	192.9	194.2	193.4	196.1	192.7
Weight of Dry Soil	247.6	456.6	396.8	652.4	369.9
Soil Weight Retained on #200&Pan	419.1	528.2	441.7	764.5	534.2
Soil Weight Passing #200	21.4	122.6	148.5	84.0	28.4
Percent Passing #200	**8.6**	**26.9**	**37.4**	**12.9**	**7.7**

Lab Sample Number					
Boring Label					
Sample Depth					
Pan Number					
Dry Weight of Soil & Pan					
Pan Weight					
Weight of Dry Soil					
Soil Weight Retained on #200&Pan					
Soil Weight Passing #200					
Percent Passing #200					

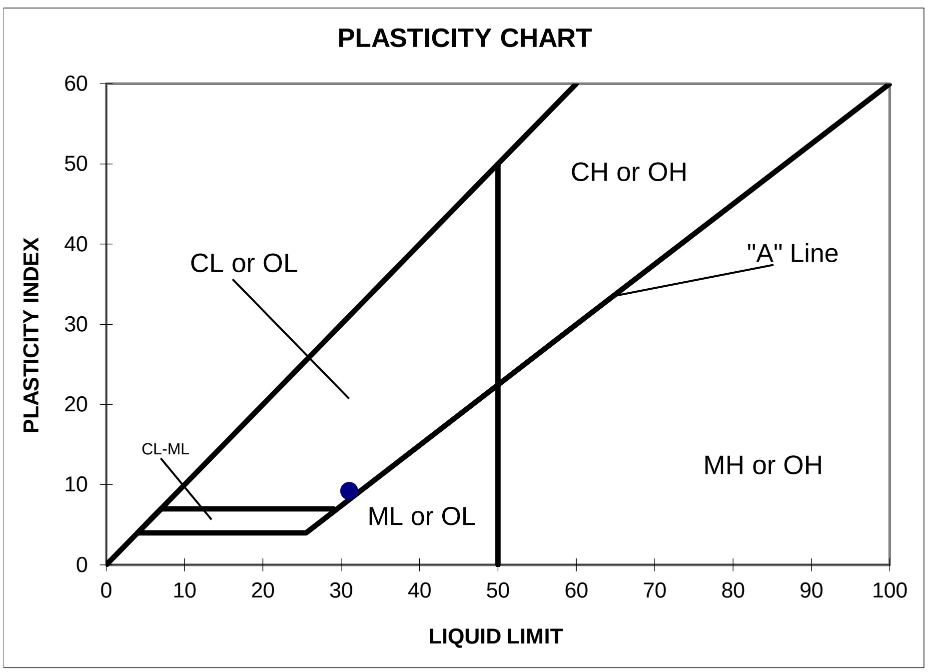
ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

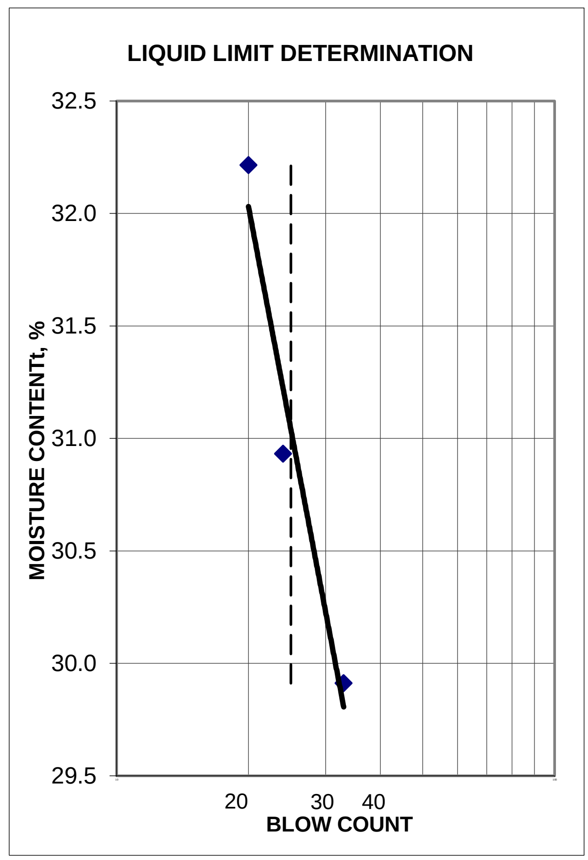
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME:	**EHS- Albee Stadium**	JOB #:	**020010.100**	LAB SAMPLE #:	**20-821**
SAMPLE ID:	**B-01-20 @ 19-19.5**	PERFORMED BY:	**ESP**	DATE:	**9/8/2020**
PROJECT MANAGER:	**GSW**	CHECKED BY:	**NAN**	DATE:	**9/18/2020**

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)	20.600	21.000	29.340	28.880	29.650
C	WT. WET SOIL & PAN (g)	27.640	27.990	36.680	36.330	35.560
D	WT. DRY SOIL & PAN (g)	26.360	26.760	34.990	34.570	34.120
E	WT. WATER (C-D)	1.280	1.230	1.690	1.760	1.440
F	WT. DRY SOIL (D-B)	5.760	5.760	5.650	5.690	4.470
G	BLOW COUNT	--	--	33	24	20
H	MOISTURE CONTENT (E/F*100)	22.2	21.4	29.9	30.9	32.2

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
31	**9**	22





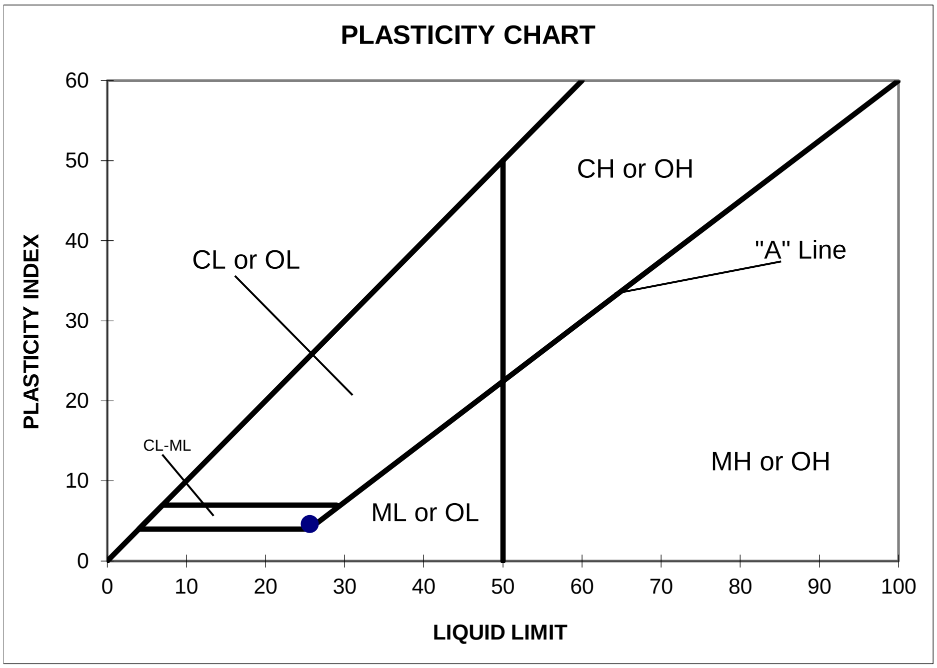
ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

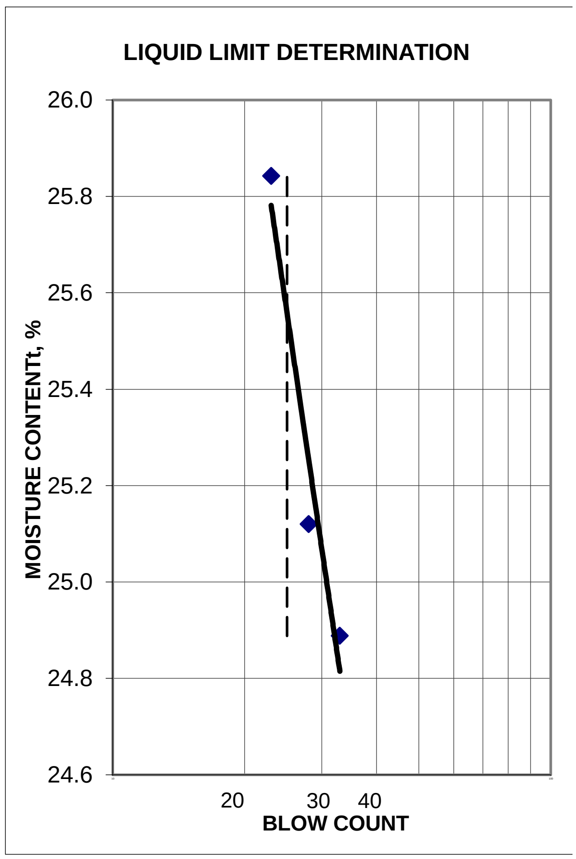
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME:	**EHS-Albee Stadium**	JOB #:	**020010.100**	LAB SAMPLE #:	**20-887**
SAMPLE ID:	**B-05-02 @ 16-16.5'**	PERFORMED BY:	**ESP**	DATE:	**9/8/2020**
PROJECT MANAGER:	**GSW**	CHECKED BY:	**NAN**	DATE:	**9/18/2020**

LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	13	14	1	2	3
B	PAN WT. (g)	22.040	20.160	29.650	28.980	28.980
C	WT. WET SOIL & PAN (g)	29.500	27.340	38.030	36.800	35.700
D	WT. DRY SOIL & PAN (g)	28.200	26.100	36.360	35.230	34.320
E	WT. WATER (C-D)	1.300	1.240	1.670	1.570	1.380
F	WT. DRY SOIL (D-B)	6.160	5.940	6.710	6.250	5.340
G	BLOW COUNT	--	--	33	28	23
H	MOISTURE CONTENT (E/F*100)	21.1	20.9	24.9	25.1	25.8

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
26	**5**	21





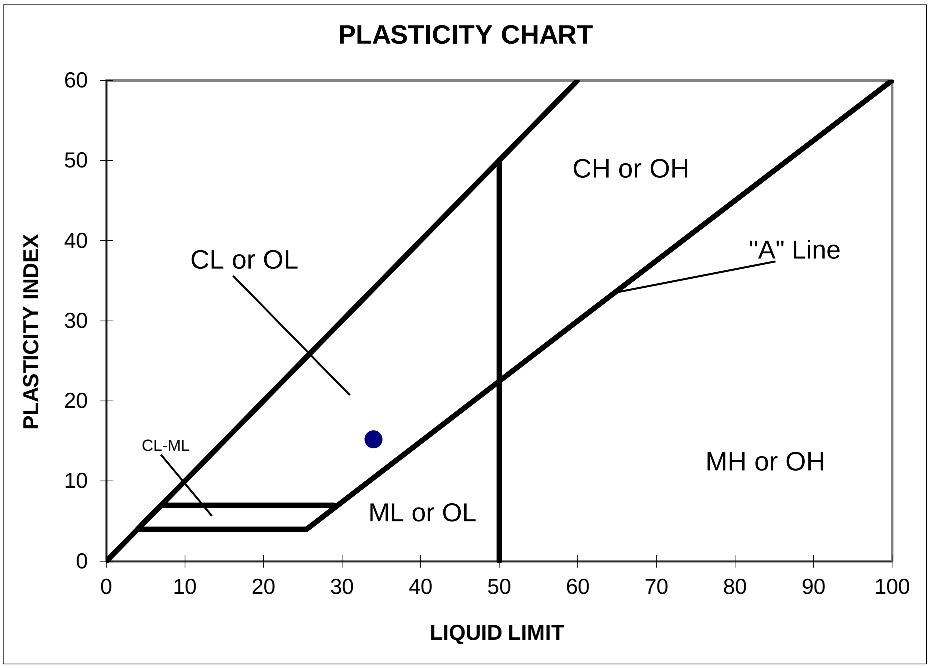
ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

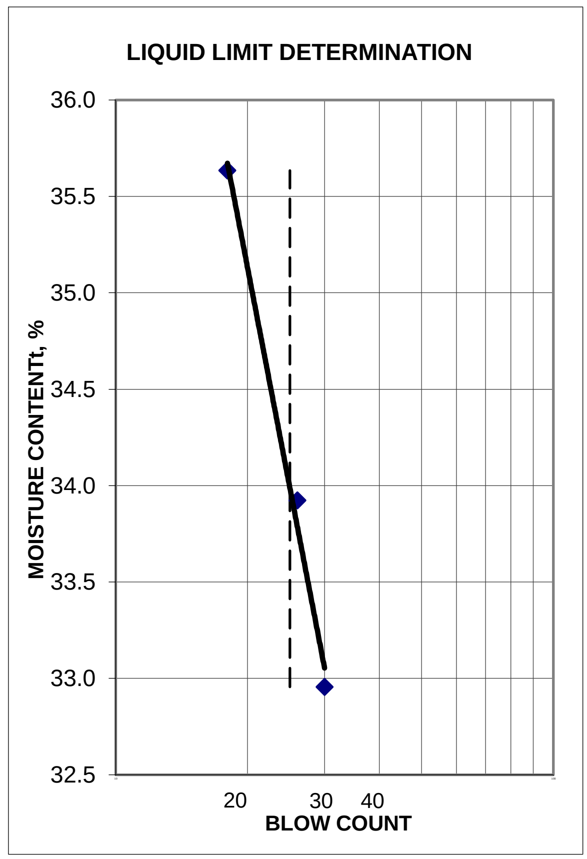
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME:	EHS- Albee Stdium	JOB #:	020010.100	LAB SAMPLE #:	20-932
SAMPLE ID:	B-08-20 - 35-36.5	PERFORMED BY:	JMA	DATE:	9/16/2020
PROJECT MANAGER:	GSW	CHECKED BY:	NAN	DATE:	9/18/2020

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)	20.600	21.000	29.320	28.830	29.610
C	WT. WET SOIL & PAN (g)	27.370	27.440	38.720	37.160	38.060
D	WT. DRY SOIL & PAN (g)	26.310	26.410	36.390	35.050	35.840
E	WT. WATER (C-D)	1.060	1.030	2.330	2.110	2.220
F	WT. DRY SOIL (D-B)	5.710	5.410	7.070	6.220	6.230
G	BLOW COUNT	--	--	30	26	18
H	MOISTURE CONTENT (E/F*100)	18.6	19.0	33.0	33.9	35.6

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
34	**15**	19





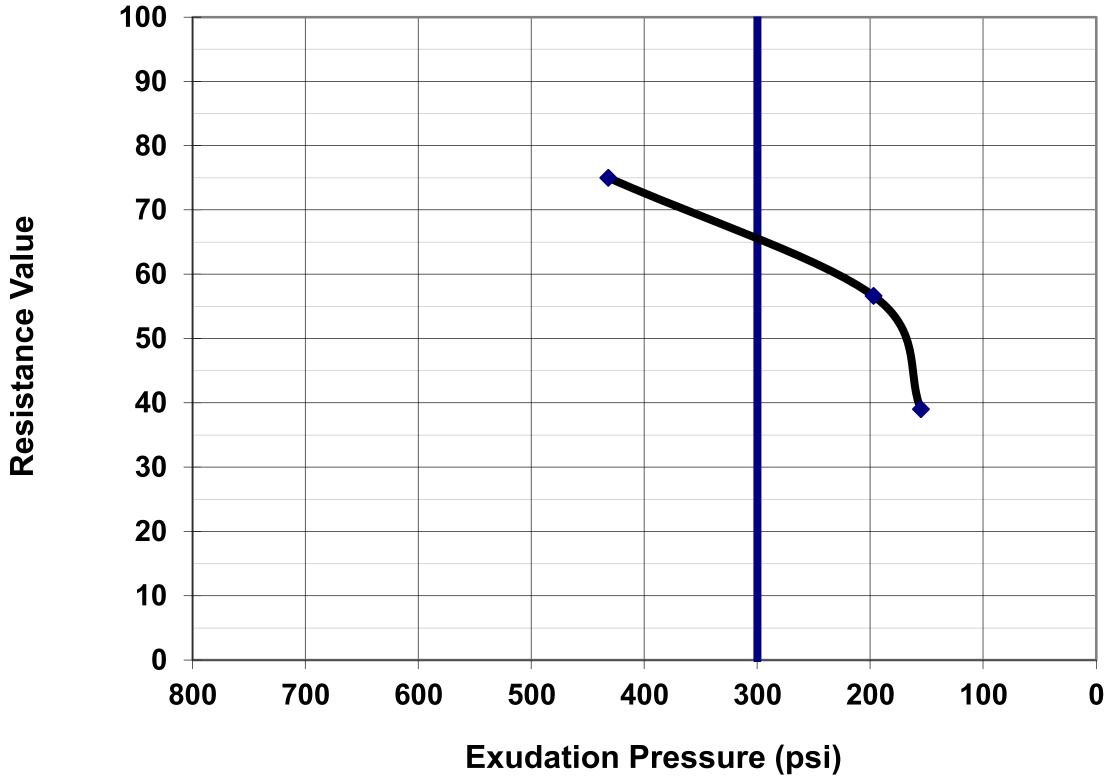
Resistance, R-Value

Caltrans Method 301

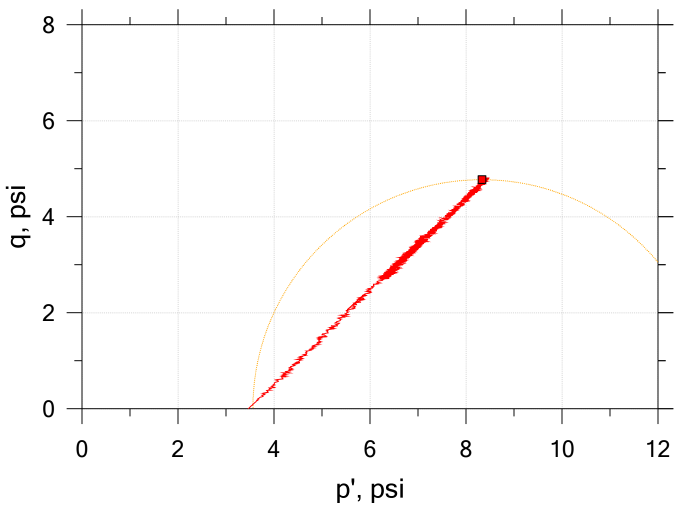
Project :	Albee Stadium	Project No. :	020010
Client :	Eureka City Schools	Sampled By :	JMA
Sample Location :	0-12"	Test Date :	1/0/1900
Sample Description :	Silty SAND	Sample Number :	20-1188

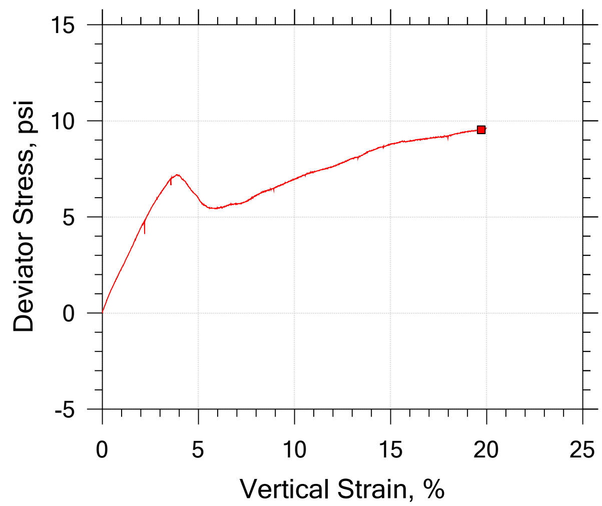
Test Specimen	1	2	3
Moisture Content (%)	14.9	16.5	18.1
Dry Density (pcf)	108.2	109.2	106.0
Expansion Pressure (psf)	277.1	112.6	60.6
Exudation Pressure (psi)	432	197	155
Resistance Value	75	57	39

R Value at 300 psi Exudation Pressure: 65



UNCONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D2850



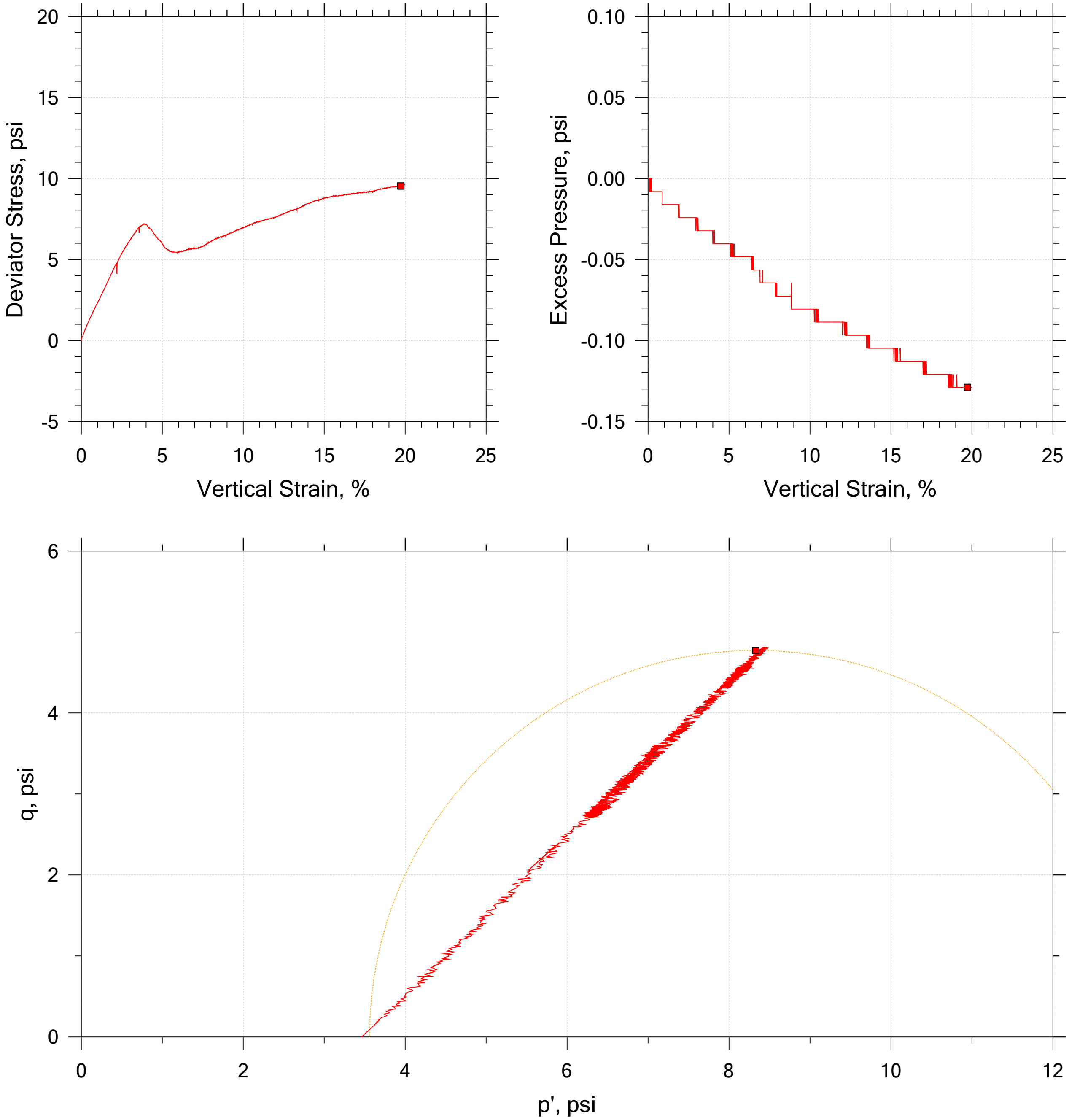


Symbol		■			
Sample ID		1			
Depth, ft		6-6.5			
Test Number		21-674			
Initial	Height, in	4.650			
	Diameter, in	2.420			
	Moisture Content (from Cuttings), %	32.5			
	Dry Density, pcf	97.0			
	Saturation (Wet Method), %	120.4			
	Void Ratio	0.724			
Final	Moisture Content, %	27.0			
	Dry Density, pcf	97.1			
	Cross-Sectional Area (Method A), in²	4.597			
	Saturation, %	100.0			
	Void Ratio	0.723			
	Back Pressure, %	0.0000			
Vertical Effective Consolidation Stress, psi		3.450			
Horizontal Effective Consolidation Stress, psi		3.462			
Vertical Strain after Consolidation, %		0.0000			
Volumetric Strain after Consolidation, %		0.0000			
Time to 50% Consolidation, min		0.0000			
Shear Strength, psi		4.773			
Strain at Failure, %		19.7			
Strain Rate, %/min		1.000			
Deviator Stress at Failure, psi		9.545			
Effective Minor Principal Stress at Failure, psi		3.559			
Effective Major Principal Stress at Failure, psi		13.10			
B-Value		---			

Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

Project: EHS Albee Stadium	Location: Eureka	Project No.: 020010.100
Boring No.: B-01-21	Tested By: JMA	Checked By:
Sample No.: 1	Test Date: 7/16/21	Depth: 6-6.5
Test No.: 21-674	Sample Type: Undisturbed	Elevation:
Description: Brown SAND		
Remarks:		

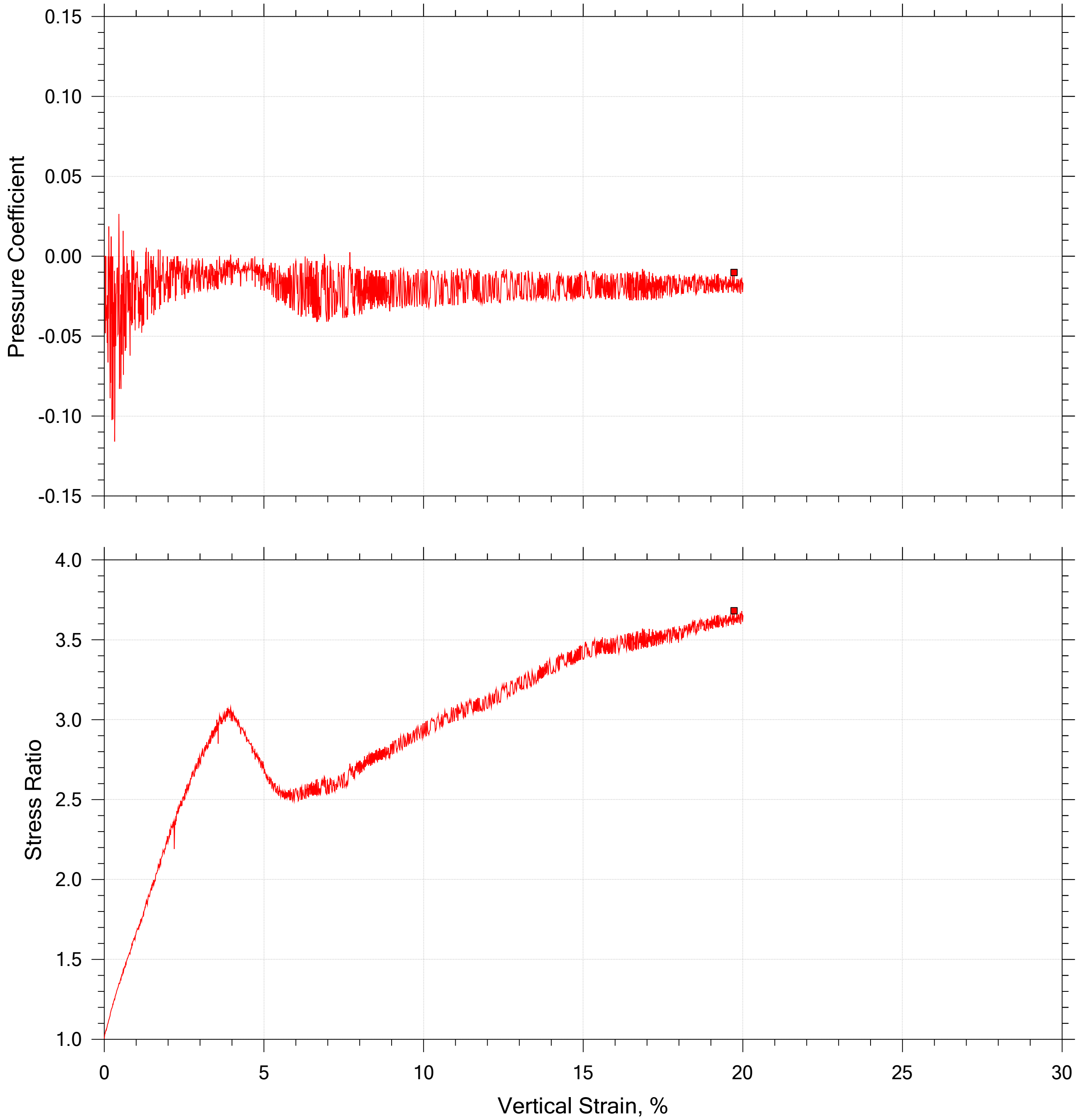
UNCONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D2850



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	1	21-674	6-6.5	JMA	7/16/21			TXUU 21-674 albee.dat

Project: EHS Albee Stadium	Location: Eureka	Project No.: 020010.100
Boring No.: B-01-21	Tested By: JMA	Checked By:
Sample No.: 1	Test Date: 7/16/21	Depth: 6-6.5
Test No.: 21-674	Sample Type: Undisturbed	Elevation:
Description: Brown SAND		
Remarks:		

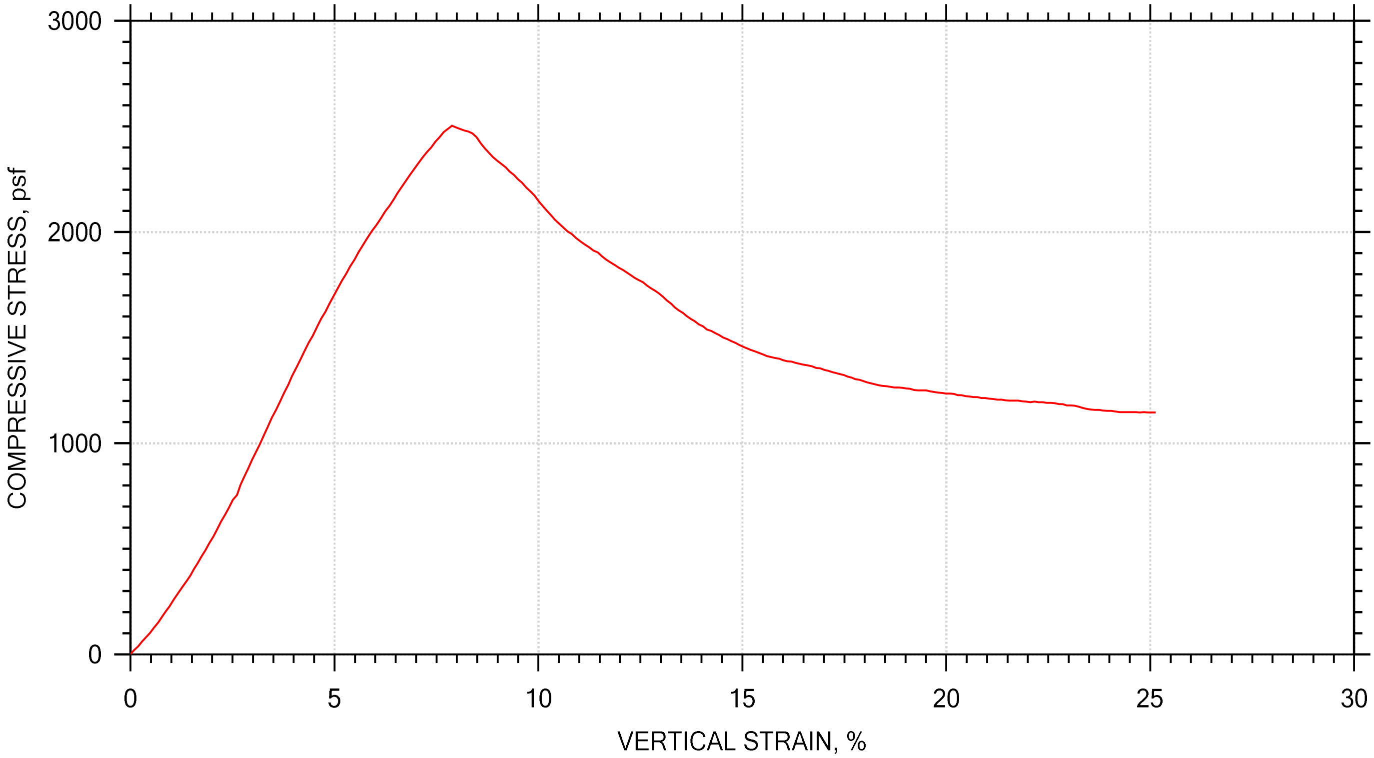
UNCONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D2850



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	1	21-674	6-6.5	JMA	7/16/21			TXUU 21-674 albee.dat

Project: EHS Albee Stadium	Location: Eureka	Project No.: 020010.100
Boring No.: B-01-21	Tested By: JMA	Checked By:
Sample No.: 1	Test Date: 7/16/21	Depth: 6-6.5
Test No.: 21-674	Sample Type: Undisturbed	Elevation:
Description: Brown SAND		
Remarks:		

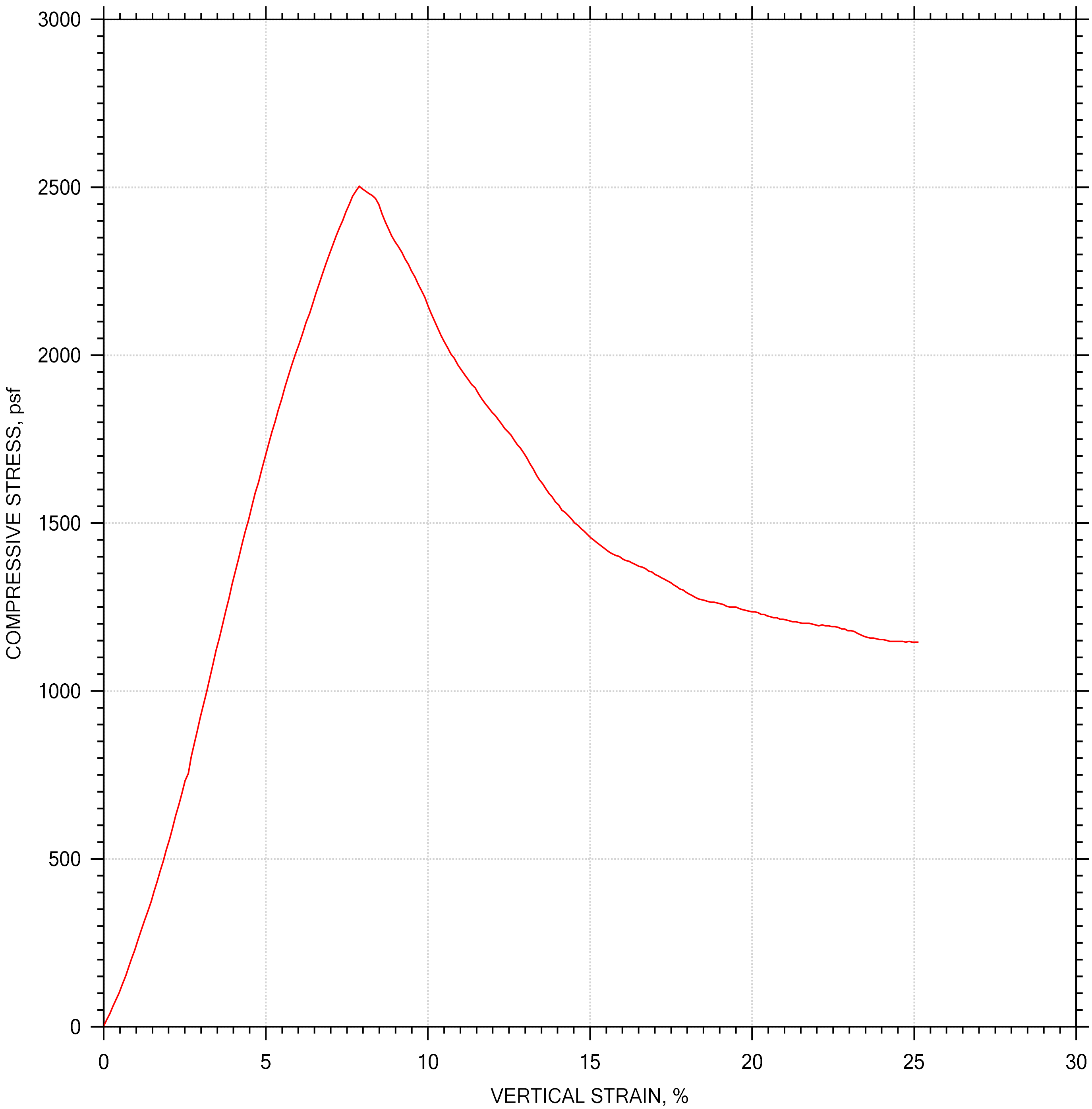
UNCONFINED COMPRESSION TEST REPORT



Symbol					
Test No.		21-677			
Initial	Diameter, in	2.42			
	Height, in	3.9			
	Water Content, %	26.43			
	Dry Density, pcf	98.03			
	Saturation, %	101.86			
	Void Ratio	0.688			
Unconfined Compressive Strength, psf		2503			
Undrained Shear Strength, psf		1252			
Time to Failure, min		7.9009			
Strain Rate, %/min		0.01			
Estimated Specific Gravity		2.65			
Liquid Limit		---			
Plastic Limit		---			
Plasticity Index		---			
Failure Sketch					

SHN	Project: EHS Albee Stadium	Location: Eureka	Project No.: 020010.100
	Boring No.: B-01-21	Tested By: JMA	Checked By:
	Sample No.: 4	Test Date: 7/16/21	Elevation:
	Test No.: 21-677	Preparation: Undisturbed	Depth: 11-11.5
	Description: Gray SILT		
	Remarks: Specimen Length/Diameter Ratio does not meet ASTM D2166		

UNCONFINED COMPRESSION TEST REPORT



Project: EHS Albee Stadium	Location: Eureka	Project No.: 020010.100
Boring No.: B-01-21	Tested By: JMA	Checked By:
Sample No.: 4	Test Date: 7/16/21	Elevation:
Test No.: 21-677	Preparation: Undisturbed	Depth: 11-11.5
Description: Gray SILT		
Remarks: Specimen Length/Diameter Ratio does not meet ASTM D2166		

Liquefaction Analysis Results

3

SHN Consulting Engineers and Geologists

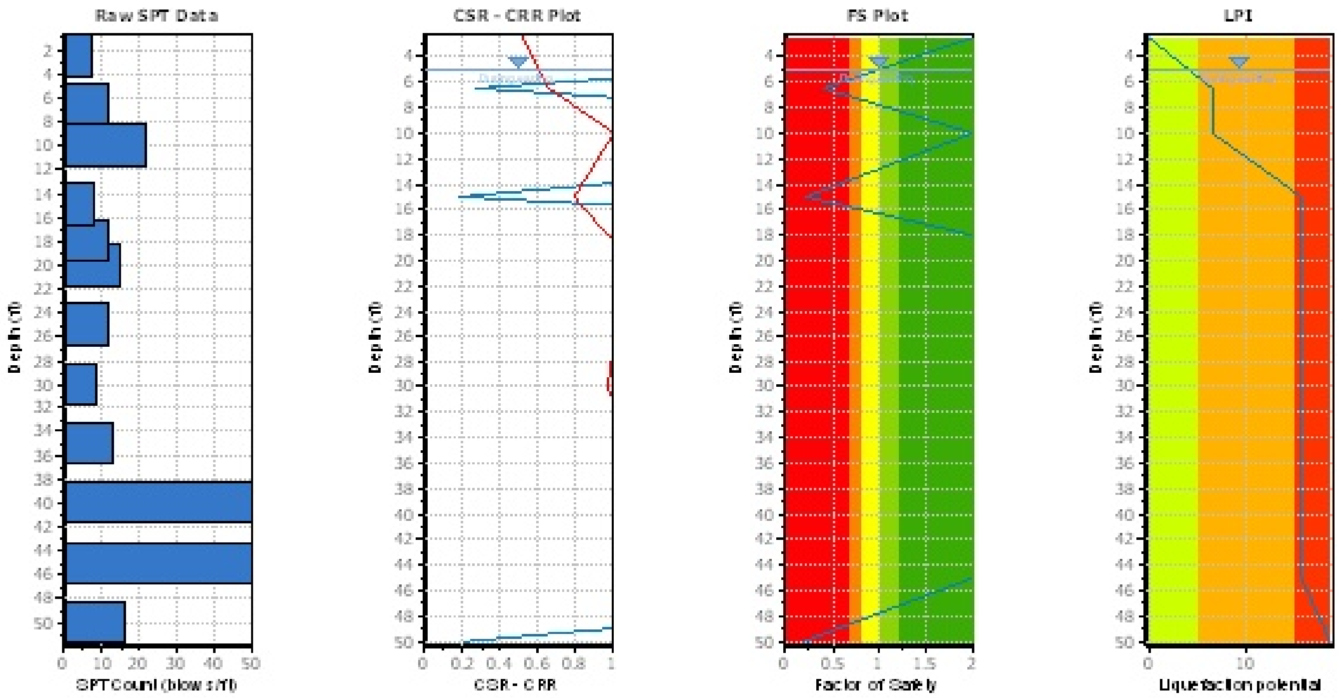
SPT BASED LIQUEFACTION ANALYSIS REPORT

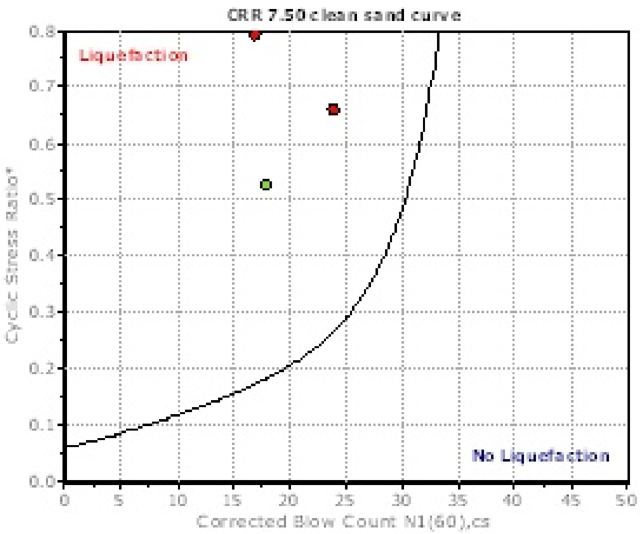
Project title : Eureka High School-Albee Stadium **SPT Name: B-01-20**

Location : Lat: 40.789 N., Lng:-124.1555 W.

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	6.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	5.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	8.7
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.76 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		





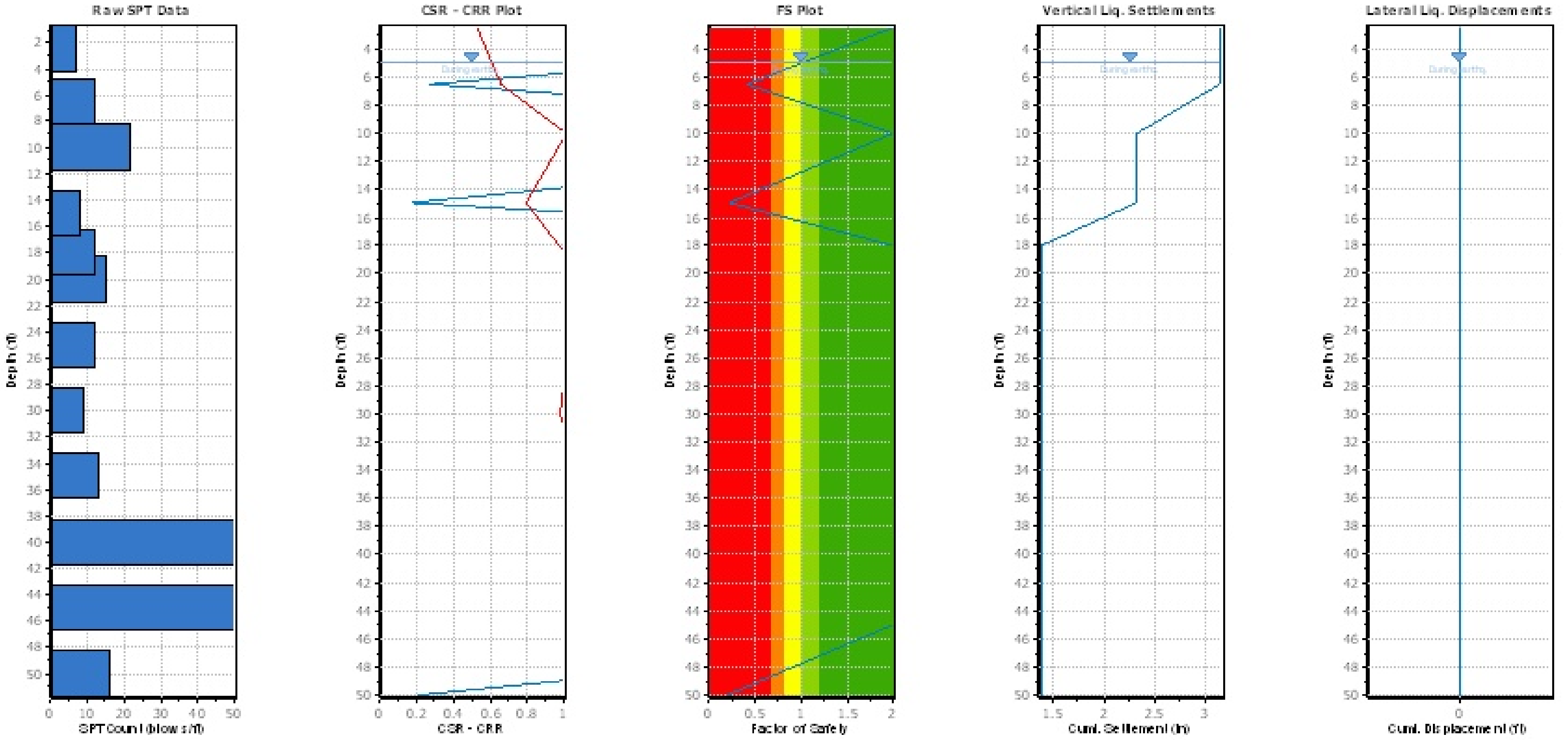
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
2.50	7	15.00	120.00	4.00	Yes
6.50	12	16.00	130.00	3.50	Yes
10.00	22	17.00	130.00	5.00	Yes
15.00	8	17.00	130.00	3.00	Yes
18.00	12	95.00	130.00	2.00	No
20.00	15	95.00	130.00	5.00	No
25.00	12	95.00	130.00	5.00	No
30.00	9	95.00	130.00	5.00	No
35.00	13	95.00	130.00	5.00	No
40.00	50	7.00	130.00	5.00	Yes
45.00	50	7.00	130.00	5.00	Yes
50.00	16	7.00	130.00	5.00	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
2.50	7	120.00	0.15	0.00	0.15	0.45	1.70	1.25	1.00	0.75	1.20	14	15.00	3.26	18	4.000
6.50	12	130.00	0.41	0.00	0.41	0.40	1.46	1.25	1.00	0.75	1.20	20	16.00	3.58	24	0.268
10.00	22	130.00	0.64	0.11	0.53	0.30	1.23	1.25	1.00	0.85	1.20	35	17.00	3.85	39	4.000
15.00	8	130.00	0.96	0.27	0.70	0.47	1.21	1.25	1.00	0.85	1.20	13	17.00	3.85	17	0.174
18.00	12	130.00	1.16	0.36	0.80	0.40	1.12	1.25	1.00	0.95	1.20	20	95.00	5.50	26	4.000
20.00	15	130.00	1.29	0.42	0.87	0.37	1.08	1.25	1.00	0.95	1.20	24	95.00	5.50	30	4.000
25.00	12	130.00	1.61	0.58	1.04	0.42	1.01	1.25	1.00	0.95	1.20	18	95.00	5.50	24	4.000
30.00	9	130.00	1.94	0.73	1.20	0.46	0.94	1.25	1.00	1.00	1.20	13	95.00	5.50	19	4.000
35.00	13	130.00	2.26	0.89	1.37	0.42	0.90	1.25	1.00	1.00	1.20	18	95.00	5.50	24	4.000
40.00	50	130.00	2.59	1.05	1.54	0.13	0.95	1.25	1.00	1.00	1.20	72	7.00	0.14	73	4.000
45.00	50	130.00	2.91	1.20	1.71	0.14	0.93	1.25	1.00	1.00	1.20	71	7.00	0.14	72	4.000
50.00	16	130.00	3.24	1.36	1.88	0.45	0.77	1.25	1.00	1.00	1.20	19	7.00	0.14	20	0.206

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
m:	Stress exponent normalization factor
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} to a 60% energy ratio
$\Delta(N_1)_{60}$	Equivalent clean sand adjustment
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cyclic resistance ratio for M=7.5

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

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
2.50	120.00	0.15	0.00	0.15	1.00	0.496	1.42	18	0.86	0.579	1.10	0.526	2.000
6.50	130.00	0.41	0.05	0.36	1.00	0.559	1.67	24	0.77	0.726	1.10	0.660	0.406
10.00	130.00	0.64	0.16	0.48	1.00	0.654	2.20	39	0.59	1.113	1.10	1.012	2.000
15.00	130.00	0.96	0.31	0.65	1.00	0.729	1.38	17	0.87	0.839	1.06	0.793	0.219
18.00	130.00	1.16	0.41	0.75	1.00	0.757	1.77	26	0.74	1.030	1.06	0.973	2.000
20.00	130.00	1.29	0.47	0.82	0.99	0.771	2.00	30	0.66	1.173	1.05	1.115	2.000
25.00	130.00	1.61	0.62	0.99	0.99	0.798	1.67	24	0.77	1.036	1.01	1.025	2.000
30.00	130.00	1.94	0.78	1.16	0.98	0.814	1.45	19	0.84	0.965	0.99	0.976	2.000
35.00	130.00	2.26	0.94	1.33	0.98	0.825	1.67	24	0.77	1.072	0.96	1.111	2.000
40.00	130.00	2.59	1.09	1.50	0.97	0.832	2.20	73	0.59	1.415	0.90	1.576	2.000
45.00	130.00	2.91	1.25	1.66	0.97	0.835	2.20	72	0.59	1.420	0.87	1.639	2.000
50.00	130.00	3.24	1.40	1.83	0.96	0.835	1.49	20	0.83	1.006	0.93	1.085	0.190

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	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
FS:	Calculated factor of safety against soil liquefaction

GEOLOGISMIKI Geotechnical Software

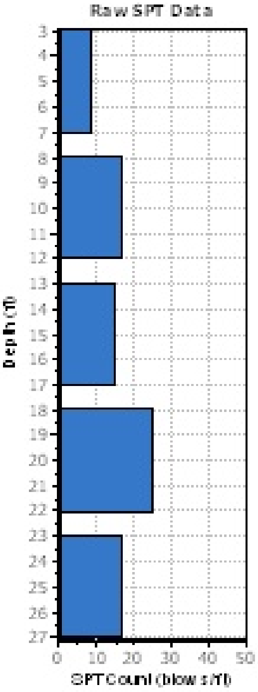
SPT BASED LIQUEFACTION ANALYSIS REPORT

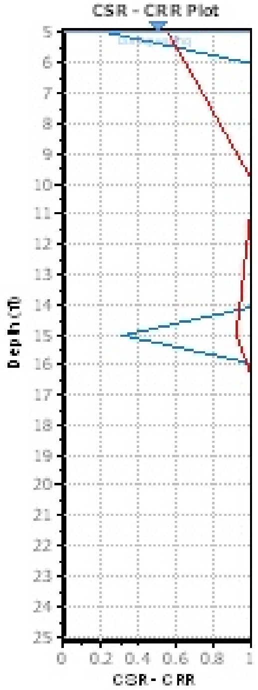
Project title : Eureka High School-Albee Stadium **SPT Name: B-02-20**

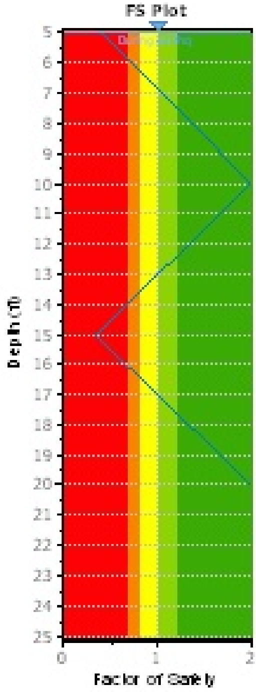
Location : Lat: 40.789 N., Lng:-124.1555 W.

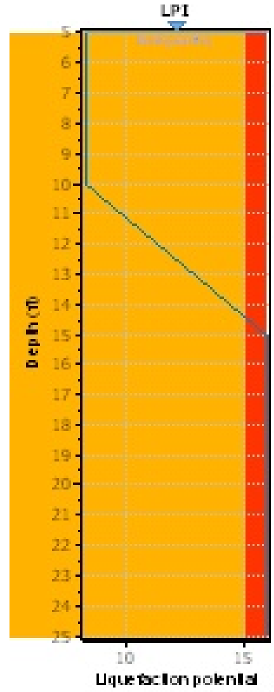
:: Input parameters and analysis properties ::

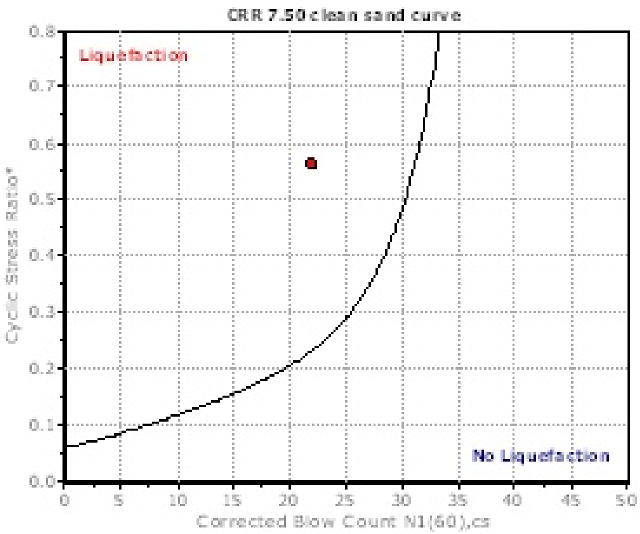
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	5.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	5.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	8.7
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.76 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		











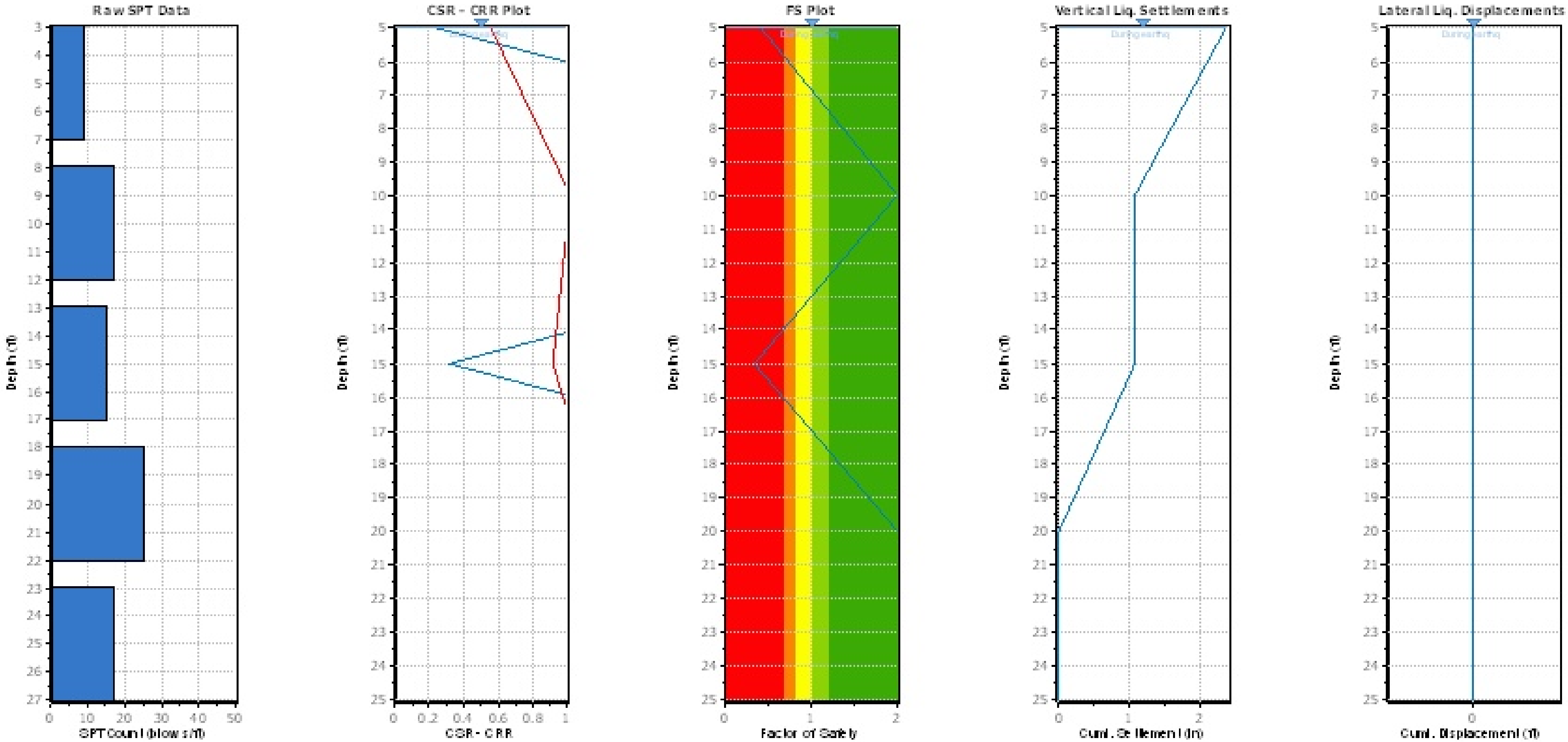
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	9	20.00	122.00	5.00	Yes
10.00	17	21.00	127.00	5.00	Yes
15.00	15	11.00	127.00	5.00	Yes
20.00	25	17.00	127.00	5.00	Yes
25.00	17	10.00	127.00	5.00	No

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	9	122.00	0.30	0.00	0.30	0.41	1.66	1.25	1.00	0.75	1.20	17	20.00	4.48	22	0.233
10.00	17	127.00	0.62	0.16	0.47	0.33	1.31	1.25	1.00	0.85	1.20	29	21.00	4.63	34	4.000
15.00	15	127.00	0.94	0.31	0.63	0.40	1.23	1.25	1.00	0.85	1.20	24	11.00	1.61	26	0.316
20.00	25	127.00	1.26	0.47	0.79	0.28	1.09	1.25	1.00	0.95	1.20	39	17.00	3.85	43	4.000
25.00	17	127.00	1.57	0.62	0.95	0.40	1.04	1.25	1.00	0.95	1.20	26	10.00	1.15	28	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
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} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

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

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
5.00	122.00	0.30	0.00	0.30	1.00	0.495	1.58	22	0.80	0.618	1.10	0.562	0.415
10.00	127.00	0.62	0.16	0.47	1.00	0.659	2.20	34	0.59	1.121	1.10	1.019	2.000
15.00	127.00	0.94	0.31	0.63	1.00	0.737	1.77	26	0.74	1.003	1.09	0.922	0.343
20.00	127.00	1.26	0.47	0.79	0.99	0.782	2.20	43	0.59	1.330	1.09	1.224	2.000
25.00	127.00	1.57	0.62	0.95	0.99	0.810	1.88	28	0.70	1.160	1.02	1.138	2.000

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

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	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
FS:	Calculated factor of safety against soil liquefaction

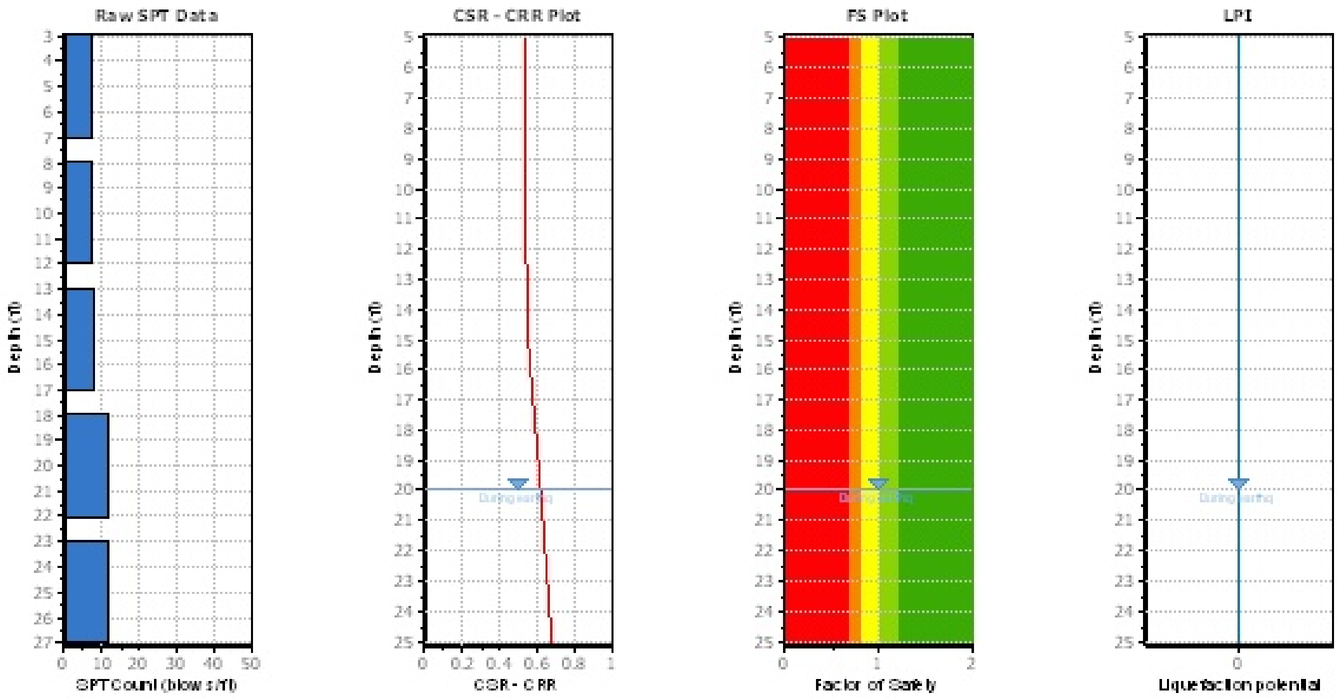
SPT BASED LIQUEFACTION ANALYSIS REPORT

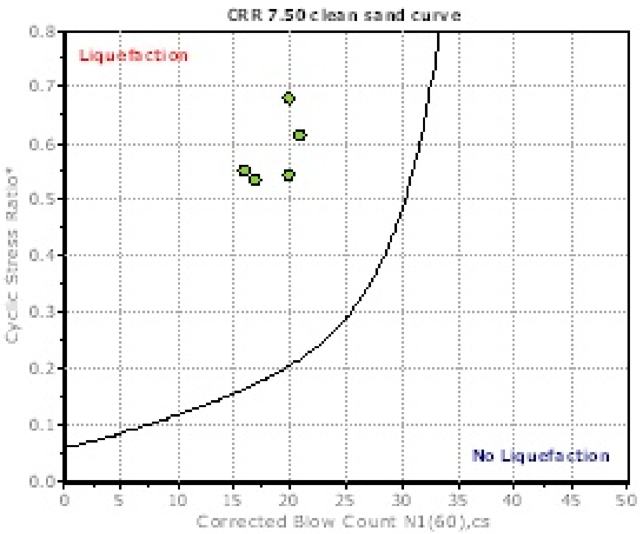
Project title : Eureka High School-Albee Stadium **SPT Name: B-03-20**

Location : Lat: 40.789 N., Lng:-124.1555 W.

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	26.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	20.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	8.7
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.76 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		





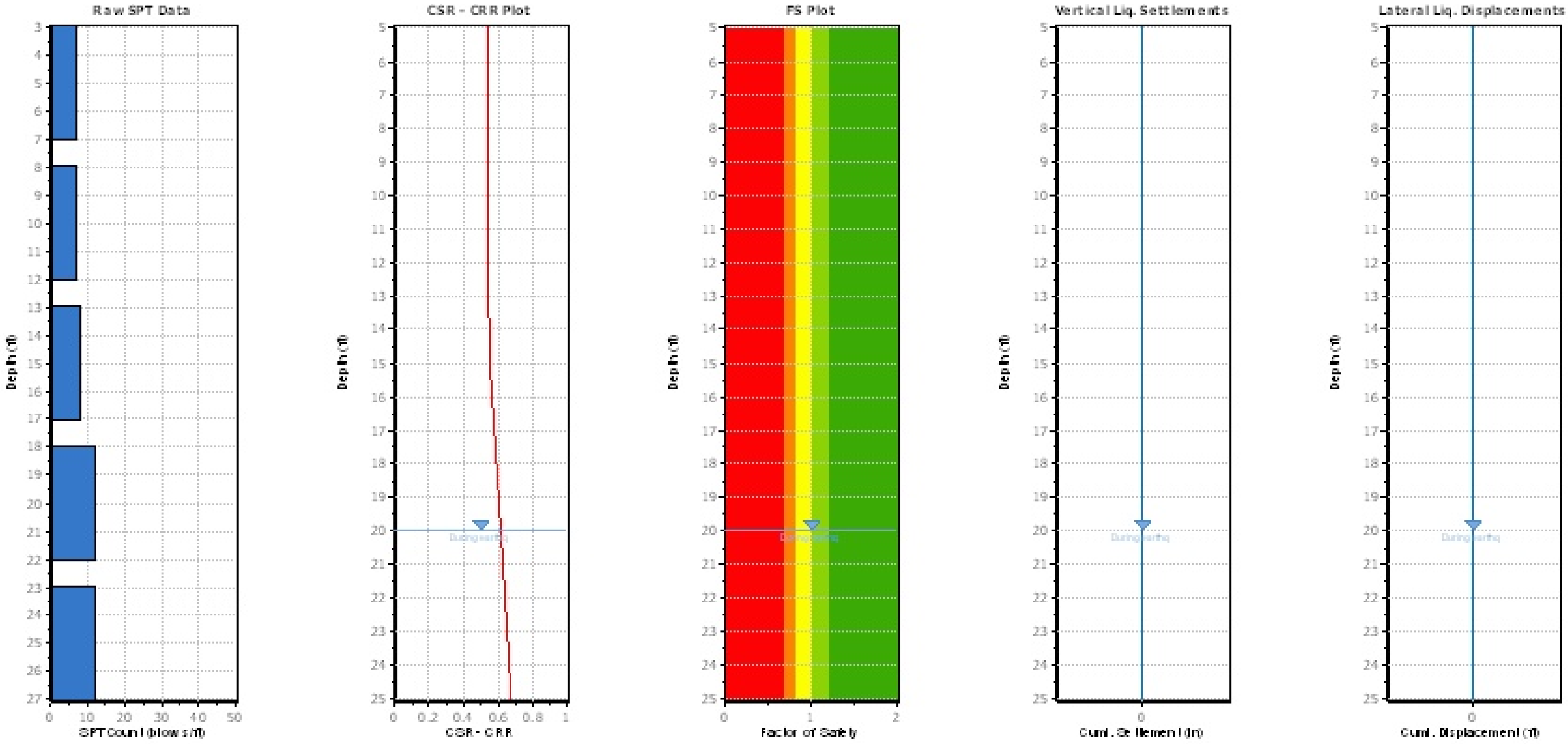
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	7	28.00	124.00	5.00	Yes
10.00	7	22.00	124.00	5.00	Yes
15.00	8	22.00	124.00	5.00	No
20.00	12	22.00	124.00	5.00	No
25.00	12	22.00	124.00	5.00	No

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	7	124.00	0.31	0.00	0.31	0.42	1.68	1.25	1.00	0.75	1.20	14	28.00	5.27	20	4.000
10.00	7	124.00	0.62	0.00	0.62	0.46	1.28	1.25	1.00	0.85	1.20	12	22.00	4.77	17	4.000
15.00	8	124.00	0.93	0.00	0.93	0.48	1.06	1.25	1.00	0.85	1.20	11	22.00	4.77	16	4.000
20.00	12	124.00	1.24	0.00	1.24	0.44	0.93	1.25	1.00	0.95	1.20	16	22.00	4.77	21	4.000
25.00	12	124.00	1.55	0.00	1.55	0.45	0.84	1.25	1.00	0.95	1.20	15	22.00	4.77	20	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
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} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

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

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
5.00	124.00	0.31	0.00	0.31	1.00	0.495	1.49	20	0.83	0.596	1.10	0.542	2.000
10.00	124.00	0.62	0.00	0.62	1.00	0.494	1.38	17	0.87	0.568	1.06	0.534	2.000
15.00	124.00	0.93	0.00	0.93	1.00	0.493	1.35	16	0.88	0.559	1.01	0.551	2.000
20.00	124.00	1.24	0.00	1.24	0.99	0.491	1.53	21	0.82	0.601	0.98	0.615	2.000
25.00	124.00	1.55	0.16	1.39	0.99	0.544	1.49	20	0.83	0.654	0.96	0.679	2.000

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

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	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
FS:	Calculated factor of safety against soil liquefaction

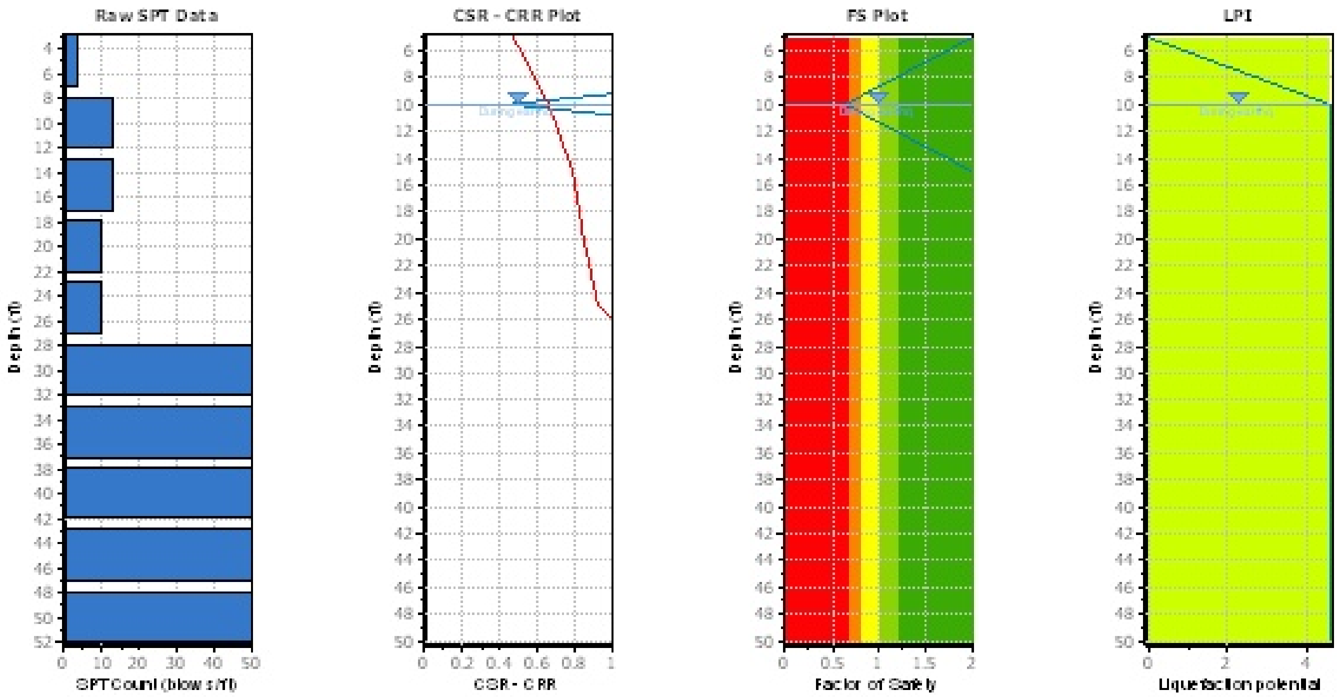
SPT BASED LIQUEFACTION ANALYSIS REPORT

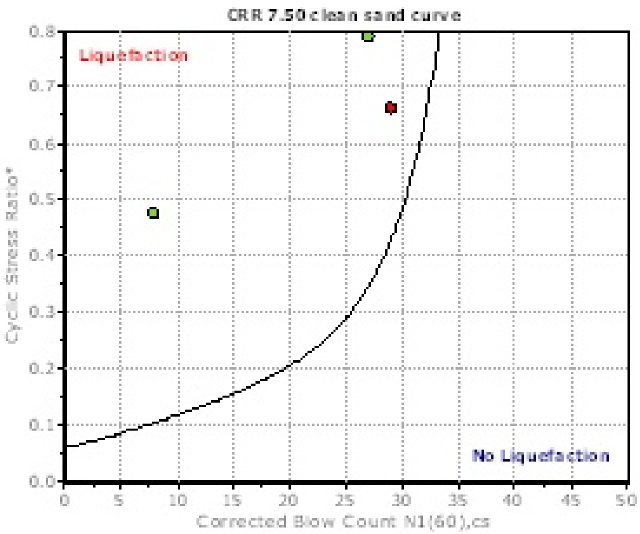
Project title : Eureka High School-Albee Stadium **SPT Name: B-04-20**

Location : Lat: 40.789 N., Lng:-124.1555 W.

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	11.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	10.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	8.7
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.76 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		





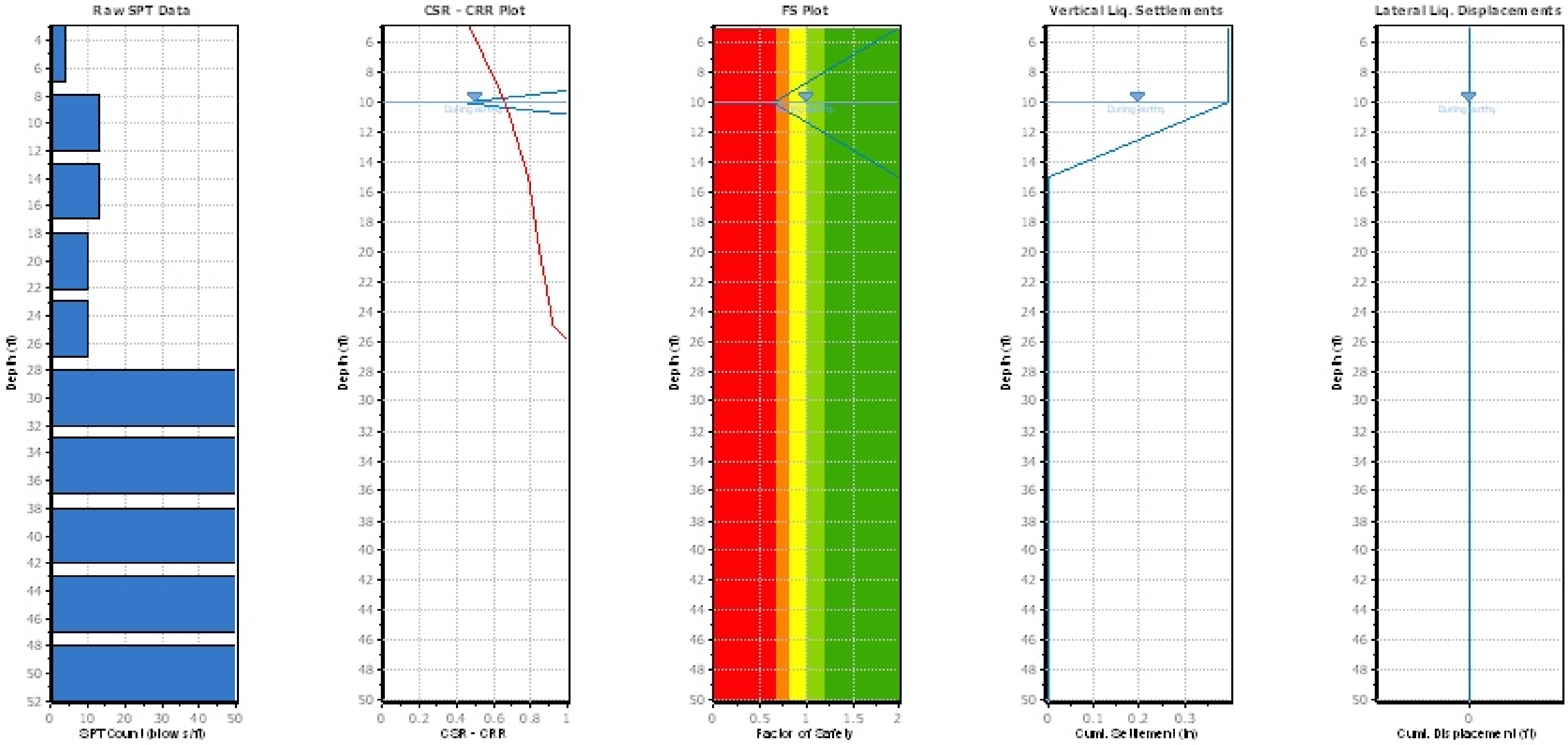
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	4	0.00	88.00	5.00	Yes
10.00	13	31.00	107.00	3.00	Yes
15.00	13	31.00	107.00	5.00	No
20.00	10	31.00	107.00	5.00	No
25.00	10	31.00	107.00	5.00	No
30.00	50	31.00	107.00	5.00	Yes
35.00	50	31.00	107.00	5.00	Yes
40.00	50	31.00	107.00	5.00	Yes
45.00	50	31.00	107.00	5.00	Yes
50.00	50	31.00	107.00	5.00	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	4	88.00	0.22	0.00	0.22	0.57	1.70	1.25	1.00	0.75	1.20	8	0.00	0.00	8	4.000
10.00	13	107.00	0.49	0.00	0.49	0.37	1.33	1.25	1.00	0.85	1.20	23	31.00	5.40	29	0.429
15.00	13	107.00	0.76	0.11	0.65	0.39	1.21	1.25	1.00	0.85	1.20	21	31.00	5.40	27	4.000
20.00	10	107.00	1.02	0.27	0.76	0.42	1.15	1.25	1.00	0.95	1.20	17	31.00	5.40	23	4.000
25.00	10	107.00	1.29	0.42	0.87	0.43	1.09	1.25	1.00	0.95	1.20	16	31.00	5.40	22	4.000
30.00	50	107.00	1.56	0.58	0.98	0.09	1.01	1.25	1.00	1.00	1.20	76	31.00	5.40	82	4.000
35.00	50	107.00	1.82	0.73	1.09	0.12	1.00	1.25	1.00	1.00	1.20	75	31.00	5.40	81	4.000
40.00	50	107.00	2.09	0.89	1.20	0.10	0.99	1.25	1.00	1.00	1.20	75	31.00	5.40	81	4.000
45.00	50	107.00	2.36	1.05	1.31	0.10	0.98	1.25	1.00	1.00	1.20	74	31.00	5.40	80	4.000
50.00	50	107.00	2.63	1.20	1.43	0.11	0.97	1.25	1.00	1.00	1.20	73	31.00	5.40	79	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
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} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

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

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
5.00	88.00	0.22	0.00	0.22	1.00	0.495	1.15	8	0.95	0.523	1.10	0.475	2.000	

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

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
10.00	107.00	0.49	0.00	0.49	1.00	0.494	1.94	29	0.68	0.729	1.10	0.662	0.647	●
15.00	107.00	0.76	0.16	0.60	1.00	0.621	1.82	27	0.72	0.866	1.10	0.787	2.000	●
20.00	107.00	1.02	0.31	0.71	0.99	0.706	1.62	23	0.79	0.899	1.06	0.848	2.000	●
25.00	107.00	1.29	0.47	0.82	0.99	0.767	1.58	22	0.80	0.957	1.04	0.924	2.000	●
30.00	107.00	1.56	0.62	0.93	0.98	0.812	2.20	82	0.59	1.381	1.04	1.332	2.000	●
35.00	107.00	1.82	0.78	1.04	0.98	0.845	2.20	81	0.59	1.437	1.00	1.432	2.000	●
40.00	107.00	2.09	0.94	1.16	0.97	0.870	2.20	81	0.59	1.480	0.97	1.519	2.000	●
45.00	107.00	2.36	1.09	1.27	0.97	0.888	2.20	80	0.59	1.511	0.95	1.596	2.000	●
50.00	107.00	2.63	1.25	1.38	0.96	0.901	2.20	79	0.59	1.533	0.92	1.663	2.000	●

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	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
FS:	Calculated factor of safety against soil liquefaction

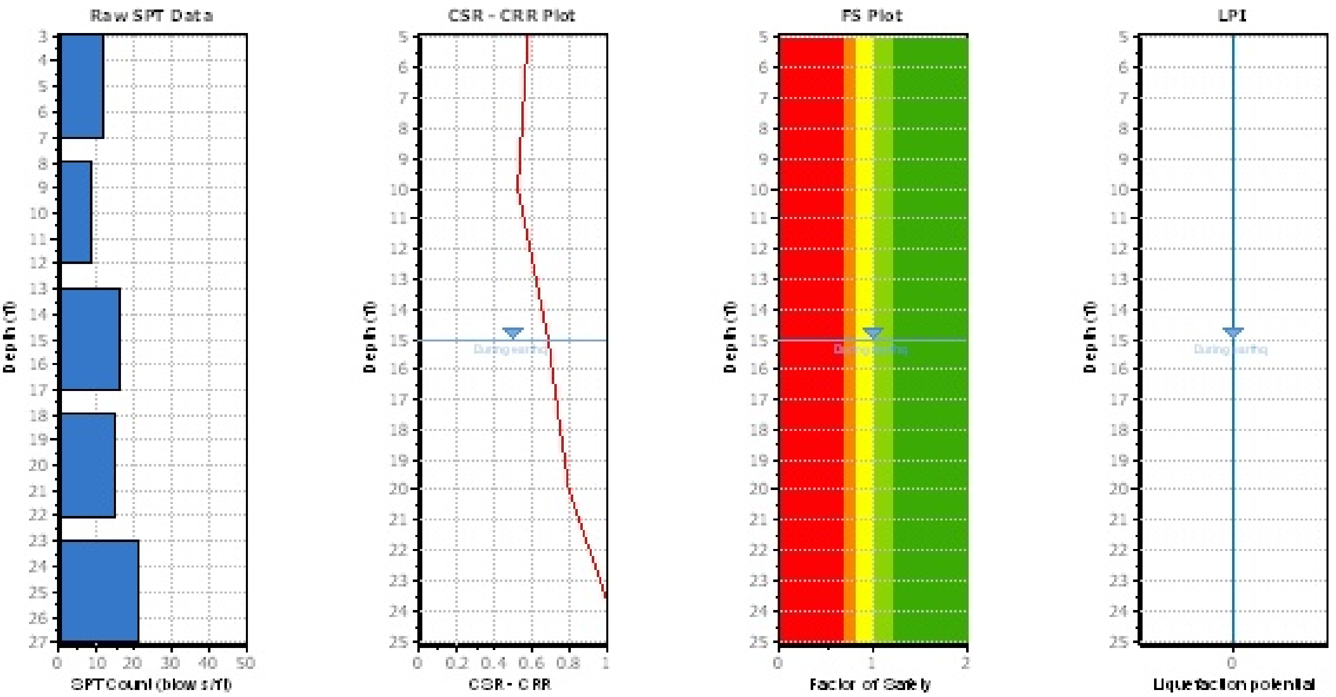
SPT BASED LIQUEFACTION ANALYSIS REPORT

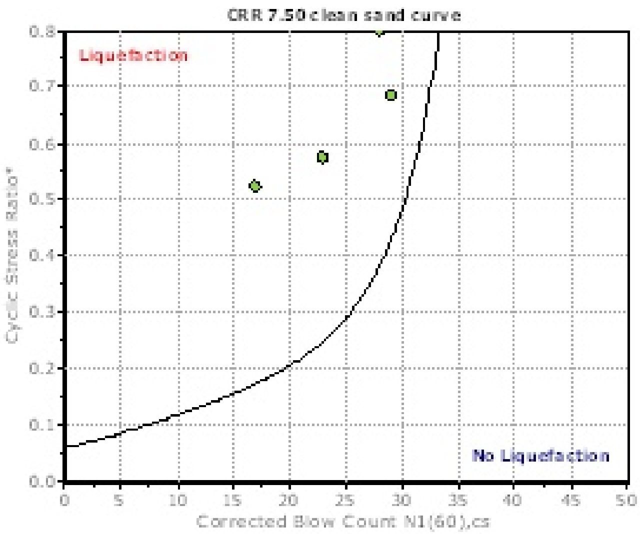
Project title : Eureka High School-Albee Stadium **SPT Name: B-05-20**

Location : Lat: 40.789 N., Lng:-124.1555 W.

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	20.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	15.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	8.7
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.76 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		





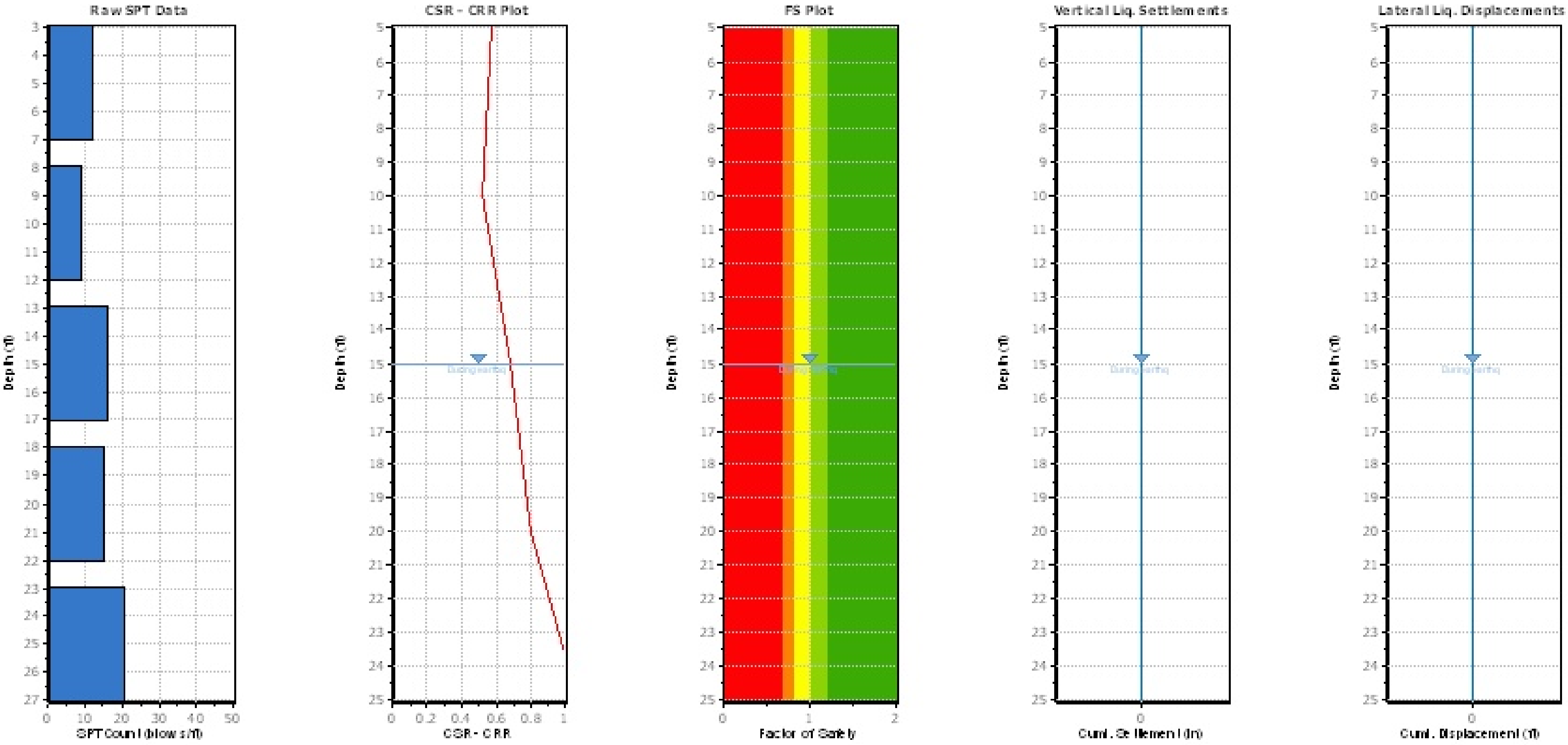
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	12	0.00	102.00	0.00	Yes
10.00	9	0.00	102.00	0.00	No
15.00	16	80.00	102.00	0.00	No
20.00	15	80.00	102.00	0.00	No
25.00	21	80.00	102.00	0.00	No

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	12	102.00	0.26	0.00	0.26	0.42	1.70	1.25	1.00	0.75	1.20	23	0.00	0.00	23	4.000
10.00	9	102.00	0.51	0.00	0.51	0.47	1.41	1.25	1.00	0.85	1.20	17	0.00	0.00	17	4.000
15.00	16	102.00	0.77	0.00	0.77	0.37	1.13	1.25	1.00	0.85	1.20	23	80.00	5.54	29	4.000
20.00	15	102.00	1.02	0.00	1.02	0.38	1.01	1.25	1.00	0.95	1.20	22	80.00	5.54	28	4.000
25.00	21	102.00	1.27	0.16	1.12	0.33	0.98	1.25	1.00	0.95	1.20	30	80.00	5.54	36	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
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} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

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

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
5.00	102.00	0.26	0.00	0.26	1.00	0.495	1.62	23	0.79	0.630	1.10	0.573	2.000
10.00	102.00	0.51	0.00	0.51	1.00	0.494	1.38	17	0.87	0.568	1.09	0.523	2.000
15.00	102.00	0.77	0.00	0.77	1.00	0.493	1.94	29	0.68	0.727	1.06	0.684	2.000
20.00	102.00	1.02	0.16	0.86	0.99	0.580	1.88	28	0.70	0.831	1.04	0.801	2.000
25.00	102.00	1.27	0.31	0.96	0.99	0.647	2.20	36	0.59	1.101	1.03	1.073	2.000

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

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	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
FS:	Calculated factor of safety against soil liquefaction

References

- Ronald D. Andrus, Hossein Hayati, Nisha P. Mohanan, 2009. Correcting Liquefaction Resistance for Aged Sands Using Measured to Estimated Velocity Ratio, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 6, June 1

- Boulanger, R.W. and Idriss, I. M., 2014. CPT AND SPT BASED LIQUEFACTION TRIGGERING PROCEDURES. DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING COLLEGE OF ENGINEERING UNIVERSITY OF CALIFORNIA AT DAVIS

- Robertson, P.K. and Cabal, K.L., 2007, Guide to Cone Penetration Testing for Geotechnical Engineering. Available at no cost at http://www.geologismiki.gr/

- Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J., Liao, S., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R., and Stokoe, K.H., Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 127, October, pp 817-833

- Zhang, G., Robertson. P.K., Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180

- Zhang, G., Robertson. P.K., Brachman, R., 2004, Estimating Liquefaction Induced Lateral Displacements using the SPT and CPT, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 130, No. 8, 861-871

- Pradel, D., 1998, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 124, No. 4, 364-368

- R. Kayen, R. E. S. Moss, E. M. Thompson, R. B. Seed, K. O. Cetin, A. Der Kiureghian, Y. Tanaka, K. Tokimatsu, 2013. Shear-Wave Velocity–Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 139, No. 3, March 1

Eureka, CA | Arcata, CA | Redding, CA | Willits, CA | Fort Bragg, CA | Coos Bay, OR | Klamath Falls, OR