MILLER PACIFIC ENGINEERING GROUP

GEOTECHNICAL INVESTIGATION
ALAMEDA UNIFIED SCHOOL DISTRICT
OTIS ELEMENTARY SCHOOL MODULAR CLASSROOM
3010 FILLMORE STREET
ALAMEDA, CALIFORNIA

September 10, 2015

Project 1911.024

Prepared For:

Alameda Unified School District
2060 Challenger Drive
Alameda, California 94501

Attention: Mr. Robbie Lyng, Director of Operations and Facilities

CERTIFICATION

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I. INTRODUCTION

This letter summarizes our geotechnical investigation for the proposed two story modular classroom structure at the Otis Elementary School Campus located at 3010 Fillmore Street in Alameda, California. The approximate site location is presented on the Site Location Map, Figure 1. The purpose of our geotechnical investigation is to evaluate the soil and groundwater conditions beneath the project area and provide geotechnical recommendations for the design and construction of the project.

Our services are provided in accordance with the terms of our Agreement for Professional Engineering and Testing Services dated June 24, 2015. The scope of our Phase 1 services includes the following:

- Review of available geologic mapping and geotechnical background information;
- Exploration of subsurface conditions with two cone penetration tests (CPTs);
- Evaluation of potential geologic hazards and respective mitigation measures;
- Development of foundation options and corresponding geotechnical design criteria;
- Development of recommendations and design criteria for site preparation and grading;
- Development of recommendations for underground utility trench excavation and backfill;
- Other geotechnical items relevant to the proposed development.

The scope of our geotechnical services does not include any investigation or evaluation of the potential for contaminated soils or hazardous materials to be present at the project site.

II. PROJECT DESCRIPTION

The proposed project includes the demolition or relocation of existing portable structures near the southeast corner of the campus and construction of a new two story modular classroom structure. The proposed project area is shown on the Site Plan, Figure 2. We understand that only minor grading (cuts and fills of less than one half foot) are planned to develop the proposed building area.

III. SITE CONDITIONS

A. Regional Geology

The site is located within the Coast Range Geomorphic Province of California. The regional bedrock geology consists of complexly folded, faulted, sheared, and altered sedimentary, igneous, and metamorphic rock of the Franciscan Complex. Bedrock is characterized by a diverse assemblage of greenstone, sandstone, shale, chert, and melange, with lesser amounts of conglomerate, calc-silicate rock, schist and other metamorphic rocks.

The regional topography is characterized by northwest-southeast trending mountain ridges and intervening valleys that were formed by movement between the North American and the Pacific Plates. Continued deformation and erosion during the late Tertiary and Quaternary Age (the last several million years) formed the prominent coastal ridges and the inland depression that is now the San Francisco Bay. The more recent seismic activity within the Coast Range Geomorphic Province is concentrated along the San Andreas Fault zone, a complex group of generally north to northwest trending faults.

Regional geologic mapping by USGS (Helley and Graymer, 1997; Graymer, 2000) indicates that Otis Elementary School is underlain by artificial fill of Holocene age while areas to the north are underlain by aeolian (dune) sands of Holocene and Pleistocene age. Helley and Graymer’s map refers to this unit as Merritt Sand (map unit Qms) displaying yardang dune morphology, while Graymer’s more recent map identifies them as Quaternary Dune Sand (map unit Qds, as shown on Figure 3) displaying barchan dune morphology, offering an estimated age of approximately 6,000- to 71,000-years old, and indicating previous recognition of paleosols within the deposits. Both maps indicate the southerly extent of the native dune sands is located just north of the northern boundary of the Otis Elementary School campus. Aside from dune morphology, the map unit descriptions offered in the accompanying reports are nearly identical, with the respective units each described as “fine-grained, very well sorted, well-drained eolian deposits”, and each report indicates the units are likely time-correlative and similarly interfingered with other alluvial deposits, including Bay Mud. A regional geologic map is shown on Figure 3.

B. Surface Conditions

The site is currently developed with existing classroom structures and associated asphalt flatwork. The area surrounding the proposed building area is relatively flat. The surrounding area consists of existing asphalt play areas, minor landscaping, and other existing classroom structures.

C. Field Exploration and Laboratory Testing

We explored the subsurface soil and groundwater conditions with two Cone Penetration Tests (CPTs) at the approximate locations shown on the Site Plan, Figure 2. The CPTs were conducted with truck-mounted equipment on July 21, 2015. CPT 1 was extended to a depth of 86 feet below the ground surface, and CPT 2 was extended to a depth of 50 feet below the ground surface. A schematic of the CPT apparatus is provided on Figure A-1 and a CPT Soil Interpretation Chart is provided on Figure A-2. CPT logs are shown on Figures A-3 and A-4.

D. Subsurface Conditions

The subsurface conditions are consistent with the mapped geology. Review of subsurface data collected from two CPTs conducted at the site indicate that the proposed building area is underlain by about five to six feet of loose to medium-dense sandy to clayey fill over a five to six foot thick layer of soft clay and organic material, interpreted as Bay Mud or similar marsh deposits. Beneath the soft clay, each CPT encountered predominantly medium-dense to dense silty sand and sandy silt extending to a depth of 40 to 50 feet, underlain by stiff clayey silt to silty clay with sandy lenses between 40/50 feet and the maximum depth explored (86 feet). Simplified geologic cross sections of the proposed building area are shown on Figure 4.

Groundwater was measured at approximately six feet below the ground surface during our CPT investigation. It is anticipated that the groundwater level beneath the site is influenced by tidal activity in the nearby San Francisco Bay. Given the low site elevations and proximity to San Francisco Bay, the highest historic groundwater elevation is assumed to coincide with the ground surface.

E. Seismicity

Active Faults in the Region - The site is located within a seismically active area and will therefore experience the effects of future earthquakes. Earthquakes are the product of the build-up and sudden release of strain along a “fault” or zone of weakness in the earth's crust. Stored energy may be released as soon as it is generated or it may be accumulated and stored for long periods of time. Individual releases may be so small that they are detected only by sensitive instruments, or they may be violent enough to cause destruction over vast areas.

Faults are seldom exposed at the earth’s surface as simple, single narrow breaks. More typically they are comprised of more or less complex zones of variable width containing a main trace with few to several secondary or branch faults. Within the Bay Area, seismically active faults are associated along the San Andreas Fault zone. During earthquakes, fault displacement may either be horizontal, vertical, or a combination, and the energy released by the movement is radiated outward in the form of waves. Not all earthquakes cause surface ground rupture. In general, ground rupture rarely occurs from earthquakes with magnitudes less than 6.0. The amplitude and frequency of earthquake ground motion waves partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy movement becomes a long, potentially damaging, high-amplitude wave motion when moving through soft ground materials, such as Bay Mud, or loose valley alluvium.

Several active faults are present in the area both east and west of the site, including the San Andreas, Hayward, San Gregorio, Mt. Diablo, and Calaveras faults. An “active” fault is one that shows displacement within the last 11,000 years and, therefore, is considered more likely to generate a future earthquake than a fault that shows no sign of recent rupture. The California Division of Mines and Geology (1998) has mapped various active and inactive faults in the region. These faults, defined as either California Building Code Source Type “A” or “B,” are shown in relation to the project site on the attached Active Fault Map, Figure 5. The closest known active fault that could produce significant seismic shaking is the Hayward fault, located approximately 6 kilometers northeast of the site.

Historic Fault Activity – Numerous earthquakes have occurred in the region within historic times. The results of our computer database search indicate that at least 25 earthquakes (Richter Magnitude 5.0 or larger) have occurred within 100 kilometers (62 miles) of the site area between 1735 and 2014. The five most significant historic earthquakes to affect the project site are summarized in Table A.

TABLE A
SIGNIFICANT EARTHQUAKE ACTIVITY
Alameda Unified School District
Otis Elementary Modular Classroom
Alameda, California

Epicenter (Latitude, Longitude)	Historic Richter Magnitude	Year	Distance
37.750, -122.550	8.2	1906	23 km
38.215, -122.312	6.0	2014	49 km
37.712, -121.728	5.8	1980	50 km
38.296, -122.755	5.7	1969	71 km
37.305, -121.708	6.1	1984	73 km

Reference: United States Geological Survey (2009), Earthquake Hazards Program," Earthquake Circular Area Search", http://neic.usgs.gov/neis/epic/epic_circ.html.

Probability of Future Earthquakes – The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San Francisco Bay region. The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities"[1,2,3] to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3 (aka UCERF, UCERF2, and UCERF3, respectively). In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

The 2003 study (UCERF) specifically analyzed fault sources and earthquake probabilities for the seven major regional fault systems in the Bay Area region of northern California. The 2008

[1] United States Geological Survey (2003), "Summary of Earthquake Probabilities in the San Francisco Bay Region, 2002 to 2032," The 2003 Working Group on California Earthquake Probabilities, 2003.
[2] United States Geological Survey (2008), "The Uniform California Earthquake Rupture Forecast, Version 2," The 2007 Working Group on California Earthquake Probabilities, Open File Report 2007-1437, 2008.

study (UCERF2) applied many of the analyses used in the 2003 study to the entire state of California and updated some of the analytical methods and models. The most recent 2013 study (UCERF3) further expanded the database of faults considered and allowed for consideration of multi-fault ruptures, among other improvements. As a result, the apparent over-prediction of moderate (M6.5-7.0) earthquakes generated by the UCERF2 model has been removed, and the UCERF3 model suggests an approximate 43% increase in the rate of all M>5.0 earthquakes statewide.

Conclusions from the most recent UCERF3 indicate the highest probability of an M>6.7 earthquake on any of the active faults in the San Francisco Bay region by 2045 is assigned to the San Andreas Fault, located approximately 23 kilometers west of the site, at 33%. The nearest known active fault, the Hayward Fault, is assigned a 32% probability of an M>6.7 earthquake by 2045. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

IV. GEOLOGIC HAZARDS EVALUATION

A. General

The principal geologic hazards which could potentially affect the project site are strong seismic shaking from future earthquakes in the San Francisco Bay Region, liquefaction, settlement/subsidence, and tsunami inundation. Other hazards are not considered significant at the site. Geologic hazards, their impacts, and recommended mitigation measures are discussed below.

B. Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Geological Survey (CGS) produced 1:24,000 scale maps showing all known active faults and defining zones within which special fault studies are required. Based on currently available published geologic information, the project site is not located within an Alquist-Priolo Earthquake Fault Zone (CDMG, 2000). The potential for fault surface rupture on the Otis Elementary School campus is therefore considered to be low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

[3] Field, E.H. et al (2015), "Long-Term Time-Dependent Probabilities for the Third Uniform California Earthquake Rupture Forecast (UCERF3)", Bulletin of the Seismological Society of America, Volume 105, No. 2A, 33pp., April

C. Seismic Shaking

The site will likely experience strong seismic ground shaking from future earthquakes in the San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 5, could cause moderate to strong ground shaking at the site.

Deterministic Seismic Hazard Analysis – Deterministic Seismic Hazard Analysis (DSHA) predicts the intensity of earthquake ground motions by analyzing the characteristics of nearby faults, distance to the faults and rupture zones, earthquake magnitudes, earthquake durations, and site-specific geologic conditions. Empirical relations (Abrahamson and Silva, Boore and Atkinson, Campbell and Borzognia, Chiou and Youngs, Idriss (2008) for the stiff subsurface conditions were utilized to provide approximate estimates of median peak site accelerations. A summary of the principal active faults affecting the site, their closest distance, moment magnitude of characteristic earthquake and probable peak ground accelerations (PGA), which an earthquake on the fault could generate at the site are shown in Table B.

TABLE B
DETERMINISTIC PEAK GROUND ACCELERATION
Alameda Unified School District
Otis Elementary Modular Classroom
Alameda, California

Fault	Approx. Distance to Fault (km)	Max. Moment Magnitude	Peak Ground Acceleration (g)[1]	84th Percentile Ground Acceleration (g)[2]
Hayward	6.1	7.3	0.37	0.59
San Andreas	23.4	8.0	0.22	0.39
Mount Diablo Thrust	23.6	6.6	0.19	0.24
Calaveras	19.9	6.9	0.18	0.30
San Gregorio	31.4	7.4	0.15	0.27

1) Values determined using Vs^{30} = 270 m/s for “stiff soil” (Site Class “D”) in accordance with the 2013 California Building Code.
2) Values determined using Pacific Earthquake Engineering Research Center (Al Atik, 2009) Earthquake NGA 2008 Excel Spreadsheet.

Reference: Caltrans (2015), ARS Online (web-based acceleration response spectra calculator tool), http://dap3.dot.ca.gov/ARS_Online/.

2015, doi: 10.1785/0120140093

Probabilistic Seismic Hazard Analysis – Probabilistic Seismic Hazard Analysis (PSHA) analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the PGA for the 10% chance of exceedance in 50 years (475 year statistical return period), utilizing the 2008 Interactive Deaggregation (USGS, 2008). The results of the probabilistic analyses are presented below in Table C.

TABLE C
PROBABILISTIC SEISMIC HAZARD ANALYSES
Alameda Unified School District
Otis Elementary Modular Classroom
Alameda, California

	Statistical Return Period	Mean Moment Magnitude	PGA
2% in 50 years	2,475 years	6.8	0.85 g
10% in 50 years	475 years	6.8	0.54 g

Reference: USGS 2008 Interactive Deaggregation (2008), V_{S30} = 270 m/s

The potential for strong seismic shaking at the project site is moderate to high. Due to its close proximity, the Hayward Fault (approximately 6 kilometers to the northeast) presents the highest potential for strong ground shaking. The most significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*

Mitigation: *Minimum mitigation measures should include designing the structures and foundations in accordance with the most recent version of the California Building Code. Recommended seismic coefficients are provided in Table D.*

D. Liquefaction Potential and Related Impacts

The project site lies within a California Seismic Hazard Zone of Required Investigation for Liquefaction, as mapped by CGS (2003), and as shown on Figure 6.
Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. Recent advances in liquefaction studies indicate that liquefaction can occur in granular materials with a high fines content (35 to 50% clayey and silty materials that pass the #200 sieve) provided the fines exhibit a plasticity less than 7. Granular layers with a potential for liquefaction were observed during our subsurface exploration.

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation. The earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the maximum credible earthquake peak ground acceleration (PGA) and depth. The soil resistance to liquefaction is based on the relative density, and the amount and plasticity of the fines (silts and clays). The relative density of cohesionless soil is correlated with Cone Penetration Test data measured in the field.

We analyzed the potential for liquefaction utilizing the CPT Liquefaction Assessment software program CLiq (2007, ver 1.7.6.49), and the procedures outlined by Idriss and Boulanger (2014). The design seismic conditions consisted of a magnitude 7.3 earthquake producing a PGA of 0.62 g, which corresponds to the PGA_M per ASCE 7-10 Section 11.8.3. The results of our liquefaction analyses are presented on Figures 9 and 10, and indicate the granular soil layers observed between roughly the ground surface and six feet, between roughly 10 to 16 feet, and between roughly 38 to 45 feet below the ground surface classify as liquefiable during the design seismic event. Therefore, we judge the risk of liquefaction at the site is moderate to high.

Based on procedures outlined by Idriss and Boulanger, 2014, the approximate 8-foot layer of potentially liquefiable soil observed 38-feet below the ground surface may experience 0.5- to 1.0-inches of post-liquefaction settlement. However, because there is a significant non-liquefiable soil “cap” overlying this liquefiable soil layer, we utilized the procedures outlined by Youd and Garris (1995) to determine if post-liquefaction settlement will be manifested in the form of ground surface settlement. As shown on Figure 11, based on the relative thicknesses of the non-liquefiable “cap” and the liquefiable layer, post-liquefaction settlements are not expected to result in ground surface settlement.

Evaluation: *Less than significant with mitigation.*

Mitigation: *A deep foundation system is recommended to mitigate the effects of liquefaction on the proposed building. The foundation design criteria provided in this report should be followed*.

E. Seismically Induced Ground Settlement

Seismic ground shaking can induce settlement of unsaturated, loose, granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits. Due to the high groundwater level at the project site, only the upper six feet of soil or less is unsaturated and subject to potential seismically induced ground settlement.

Evaluation: *Less than significant with mitigation.*
Mitigation: *A deep foundation system is recommended for the proposed building, which will mitigate the effects of shallow seismically induced ground settlement.*

F. Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep slopes or channel banks. The closest slope is located a significant distance from the campus. Therefore, the risk of lurching and ground cracking is not considered a significant risk to the proposed improvements.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

G. Erosion

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated water runoff. These conditions do not exist at the site. However, there is always some potential for localized erosion due to concentrated surface water flows.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Mitigation measures include designing a site drainage system to collect surface water and discharging it into an established storm drainage system.*

H. Seiche and Tsunami

Seiche and tsunamis are short duration, earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche would be dependent upon ground motions and fault offset from nearby active faults. The project site is located near the San Francisco Bay, and is subject to inundation by seiche or tsunami. The attached Figure 7 shows a tsunami inundation map for the project vicinity.

According to data from the National Oceanic and Atmospheric Administration (NOAA), approximately 77 credible seiche or tsunami have been recorded or observed within the San Francisco Bay area since 1700. Of these, only 14 resulted in recorded wave run-up at Alameda, with the 1964 Alaska earthquake (Magnitude 9.2) producing a maximum wave height of 0.80-meters (NOAA 2015).

There have been eight credible local seiche events observed in San Francisco Bay between 1854 and 1906, six of which are attributed to earthquake activity and two to landslides. The Mare Island earthquake caused the largest seiche with 0.6 meter amplitude waves near Benicia, and is attributed to slip on the Rodgers Creek fault. No confirmed seiche have been recorded in San Francisco Bay since 1906. In light of the recorded history of seiche in San Francisco Bay, we judge that the risk of seiche or tsunami in excess of the height observed in 1964 (approximately 2.6-feet) is low.

Planned finish grades within the development area are generally above +10 feet MSL. Therefore, while some inundation of lower lying playfields and other areas immediately proximal to the shoreline may be anticipated, the risk of water reaching the proposed modular building due to seiche or tsunami is judged to be low. In the event of a larger seiche or tsunami than historical data predicts, effects would consist mainly of shallow inundation by floodwater, flotsam, and debris. In light of this relatively low risk, it is our opinion that providing mitigation for seiche/tsunami inundation is neither warranted nor cost-effective.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

I. Expansive Soil

Expansive soils are capable of exerting significant expansion pressures on building foundations, interior floor slabs and exterior flatwork. Distress from expansive soil movement can include cracking of brittle wall coverings (stucco, plaster, drywall, etc.), racked door and/or window frames, and uneven floors and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress. Low plasticity, sandy surficial soil was encountered in our CPT probes, and therefore expansive soil is not considered a risk for the proposed improvements.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

J. Settlement/Subsidence

Significant settlement can occur when new loads are placed at sites due to consolidation of soft compressible clays (i.e. Bay Mud) or compression of loose soils. A roughly five to six foot thick layer of soft compressible clay (Bay Mud) was observed during our subsurface exploration that has a significant potential for consolidation settlement under the loading applied by any new fill and the proposed two story modular classroom. Therefore, the risk of settlement impacting the proposed structure at the project site is moderate to high.

Evaluation: *Less than significant with mitigation.*
Mitigation: *A deep foundation system is recommended to support the proposed building on stiff/dense soil beneath the soft clay layer. Foundation recommendations provided in this report should be followed.*

K. Slope Stability/Landslides

Slope instability generally occurs in relatively steep slopes and/or on slopes underlain by weak materials. The project site is located in an area of relatively flat terrain with only minor grade changes, and there are no known landslide deposits in the vicinity that could affect the site. Therefore, slope stability/landslides are not considered to be a risk to the proposed improvements at the project site.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

L. Hazardous Materials

Hazardous materials were not observed during our subsurface explorations. Environmental testing for hazardous materials was beyond the scope of our services. However, based on our study, it is our opinion the potential of significant hazardous materials located at the project site is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

M. Flooding

The project site is not mapped within a FEMA 100-year or 500-year flood zone (ABAG, 2015); therefore, large scale flooding is not a significant hazard at the project site. However, the project Civil Engineer or Architect is responsible for site drainage and should evaluate localized flooding potential and provide appropriate mitigation measures. A FEMA Flood Map for the project site and vicinity is shown on Figure 8.

Evaluation: *Less than significant with mitigation.*
Mitigation: *The project Civil Engineer or Architect should evaluate the risk of localized flooding and provide appropriate storm drainage design.*

N. Soil Corrosion

Corrosive soil can damage buried metallic structures, cause concrete spalling, and deteriorate rebar reinforcement. Laboratory testing was performed on a representative sample of the near-surface site soils to evaluate pH, electrical resistivity, chloride and sulfate contents. These laboratory test results are presented on Figure A-5.

The results of our corrosivity testing indicate the upper soil layer has a pH of about 5.80, a chloride concentration of 23 parts per million (ppm), and a sulfate concentration of 105 ppm. Per Caltrans Corrosion Guidelines (2012) a soil is considered corrosive if the pH level is less than 5.5, the chloride concentration is greater than 500 ppm, and/or the sulfate concentration is 2,000 ppm or greater. Therefore, based on the results of the corrosion testing, corrosive soil is not considered a significant risk to the proposed improvements.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

O. <u>Radon-222 Gas</u>

Radon-222 is a product of the radioactive decay of uranium-238 and raduim-226, which occur naturally in a variety of rock types, mainly phosphatic shales, but also in other igneous, metamorphic, and sedimentary rocks. While low levels of radon gas are common, very high levels, which are typically caused by a combination of poor ventilation and high concentrations of uranium and radium in the underlying geologic materials, can be hazardous to human health.

The project site is located in Alameda County, California, which is mapped in radon gas Zone 2 by the United States Environmental Protection Agency (USEPA, 2014). Zone 2 is classified by the EPA as exhibiting a “moderate” potential for Radon-222 gas.

Evaluation: *Less than significant with mitigation.*
Mitigation: *The use of a vapor barrier beneath the proposed building is recommended to reduce the potential radon exposure risk.*

P. <u>Volcanic Eruption</u>

Several active volcanoes with the potential for future eruptions exist within northern California, including Mount Shasta, Lassen Peak, and Medicine Lake in extreme northern California, the Mono Lake-Long Valley Caldera complex in east-central California, and the Clear Lake Volcanic Field, located in Lake County approximately 89 miles north of the project site. The most recent volcanic eruption in northern California was at Lassen Peak in 1917, while the most recent eruption at the nearest volcanic center to the project site, the Clear Lake Volcanic Field, was about 10,000 years ago. All of northern California’s volcanic centers are currently listed under “normal” volcanic alert levels by the USGS California Volcano Observatory (USGS, 2015). While the aforementioned volcanic centers are considered “active” by the USGS, the likelihood of damage to the proposed improvements due to volcanic eruption is generally low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

Q. Naturally Occurring Asbestos (NOA)

Naturally occurring asbestos is commonly found in association with serpentinite and associated ultramafic rock types. These rocks are a major constituent of the Franciscan Complex, which underlies vast portions of the greater San Francisco Bay Area. The site is underlain by relatively thick native alluvial soils, and while it lies in a region dominated in part by Franciscan Complex bedrock, no evidence suggesting the presence of serpentinite or related rock types was observed during our exploration. Therefore, the likelihood of naturally-occurring asbestos at the site is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

V. CONCLUSIONS AND RECOMMENDATIONS

A. General

Based on our investigation and previous experience with similar sites and projects, we conclude that the planned improvements are feasible from a geologic and geotechnical standpoint. The primary geotechnical issues to address in design of the project are seismic ground shaking, liquefaction, and settlement.

Due to anticipated settlement as a result of new building loads on undocumented fill and Bay Mud, and the potential for seismic induced liquefaction related settlement, we recommend that the proposed two story modular classroom structure be supported on a deep foundation system, as discussed in further detail below.

Loading imposed by the placement of new fill in the development area will induce settlement in the soft clay layer at a depth of roughly five to ten feet beneath the site. Although the proposed building will be supported on a deep pile foundation system, concrete walkways around the building would be negatively impacted by settlement caused by the placement of new fill. Where landscape areas and parking areas around the new building will be raised by the placement of fill, a “no new load” design should be utilized. Light weight (lava rock) fill (approximately 65 pounds per cubic foot) can be used beneath fill areas to compensate for the new loads caused by raised elevations.

B. Seismic Design

The project site is located in a seismically active area. Therefore, structures should be designed in conformance with the seismic provisions of the California Building Code (CBC) to mitigate the effects of potential ground shaking to the proposed structures. However, since the goal of the building code is protection of life safety, some structural damage may still occur during strong ground shaking.

At a minimum, we recommend the project Structural Engineer utilize the 2013 CBC coefficients shown in Table D below to determine the base shear values. Additionally, the Project Structural Engineer should determine the appropriate reference code to utilize for design.

Due to the presence of some loose sandy soil layers beneath the project site that are prone to liquefaction, we judge the site should be classified as “Site Class F” per the 2013 California Building Code. However, per the CBC and section 20.3.1 of the 2010 ASCE-7 standard, no site-specific response analysis is required since it is anticipated that the proposed building will have a fundamental period of less than 0.5 seconds. For design purposes, we recommend using the coefficients and site values shown in Table D below.

TABLE D
2013 CBC SEISMIC DESIGN FACTORS
Alameda Unified School District
Otis Elementary Modular Classroom
Alameda, California

Factor Name	Coefficient	Value
Site Class[1]	$S_{A,B,C,D,E,\text{ or }F}$	S_D
Spectral Acc. (short)	S_s	1.61 g
Spectral Acc. (1-sec)	S_1	0.63 g
Site Coefficient[2]	F_a	1.00
Site Coefficient	F_v	1.50
Spectra Parameter	S_{MS}	1.61 g
Spectra Parameter	S_{M1}	0.95 g
Design Spectra Parameter	S_{DS}	1.07 g
Design Spectra Parameter	S_{D1}	0.63 g

1. Soil Profile/Site Class Type S_D Description: Stiff Soil Profile, Shear Wave Velocity between 600 and 1,200 feet per second, Standard Penetration Test N values between 15 and 50 blows per foot, and Undrained Shear Strength between 1,000 and 2,000 psf.

C. Site Preparation and Grading

The general grading recommendations presented below are appropriate for construction in the late spring through fall months. From winter through the early spring months, on-site near surface soils will likely be saturated by rainfall and will not be compactable without drying by aeration or the addition of lime (or a similar product) to dry the soils. Site preparation and grading should conform to the recommendations and criteria outlined below.

1. Surface Preparation – Clear all grass, roots, over-sized debris, and organic material from areas that will be within the new project work area. These "organic" soils will not be suitable for use as structural fill and should be removed from the site or stockpiled for use in landscape areas. Clear all structures, old concrete footings, walls, over-size debris, and organic matter from the site.

Any existing concrete foundations should be removed where they conflict with new grades and foundations. Old utility pipes should be removed and the resulting excavations properly backfilled with compacted soils. Any areas of loose soil observed during the site preparation need to be over-excavated down to firm soil and replaced with compacted fill.

Following clearing, stripping and required excavations, the exposed soils within the project area (extending to 5 feet beyond perimeter footings and 3 feet beyond exterior slabs or pavements) should be scarified to a depth of 8-inches; moisture conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction[4] (R.C.). In areas where new asphalt or concrete pavement will be installed and subject to vehicular loads, the upper 6-inches should be moisture conditioned to within 2 percent of the optimum moisture content and compacted to at least 95 percent R.C. Subgrades should be maintained at a moist condition up until final pavement, flatwork, or structures are constructed to prevent shrinkage cracks due to drying.

2. Compacted Fill – On-site fill, backfill, and scarified subgrades should be conditioned to within 3 percent of the optimum moisture content. Properly moisture conditioned and cured on-site materials should subsequently be placed in loose horizontal lifts of 8 inches thick or less, and uniformly compacted to at least 90 percent R.C. Fills greater than 5-feet in thickness should be further compacted to at least 92 percent R.C. Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, “Moisture-Density Relations of soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop.”

3. Materials – Most onsite soils appear suitable for use as fill (in areas where light weight fill is not required, such as utility trench backfill). If imported fill (normal weight) is required, the material shall consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40 and a Plasticity Index of less than 15, and (3) have a maximum particle size of 3 inches. Any imported fill material needs to be tested to determine its suitability for use as fill material.

4. Lightweight Fill – To reduce settlements, lightweight fill should be utilized in areas where site grades are raised. Lava rock, with a maximum unit weight of 65 pounds per cubic foot or less, is probably the most practical lightweight fill material to utilize at the project site. Lava rock gradation should consist of ¾-inch to 1½-inch lava gravels. The lightweight fill should be placed in lifts no greater than 12 inches thick and statically compacted with a lightweight smooth drum compactor to the planned pad elevation.

[4] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density, as determined by laboratory test procedure (ASTM 1557).

5. Temporary Cut Slope Construction – For planning purposes, temporary cut slopes should be inclined or retained in accordance with Cal OSHA criteria based on an OSHA Type C soil profile.

Performance of temporary cut slopes will be heavily influenced by the length of time the cut is unsupported, seepage and surface runoff over or through the cut face, bedding planes of soil materials, and other factors. The project grading contractor shall be responsible for the performance of temporary cut slopes and the above recommendations should be confirmed during construction.

D. Foundation Design

Due to settlement that would be anticipated from Bay Mud consolidation under new building loading, and the potential for liquefaction induced settlement during a future seismic event, a deep foundation system supported in firm materials below the compressible bay mud and liquefiable sands is recommended for the proposed two story modular classroom structure. Suitable deep foundation options at the site could include driven piles, auger-cast piers, or torque-down piles. Traditional drilled piers are not recommended due to the high groundwater conditions, caving sandy soils, and “squeezing” Bay Mud soils. With adequately embedded foundations into the firm alluvial soils, building settlements should be small (less than one inch). Some differential settlement between the building and exterior grade should be expected due to on-going settlement and settlement associated with grading required to achieve the exterior finished grades. More detailed discussion of the deep foundation options is presented below:

In areas where future settlement is expected, deep foundations that extend through existing fill and Bay Mud will experience “down drag” effects as the bay mud settles, inducing negative skin friction on the pile. Therefore, deep foundations will need to be designed for both downdrag and structural loads.

Driven Piles

Driven piles are precast concrete or steel piles driven with a large pile hammer until a suitable driving resistance and bearing capacity is achieved. Disadvantages with driven piles are that the deeper alluvial soils are interbedded sand and clays with different strengths. The depth to achieve full pile capacity could vary across the building site. Pre-cast piles can be costly to extend or cut-off, if needed. Driven piles will also cause significant noise and vibrations.

Auger Cast Piles

Auger Cast Piles (ACP) are installed by rotating a continuously flight hollow shaft auger in to displace the soil to a specified depth. High strength cement grout is pumped under pressure through the hollow shaft as the auger is slowly withdrawn. Reinforcing steel is installed while the cement grout is still fluid, or in the case of full length single reinforcing bars, through the hollow shaft of the auger prior to the withdrawal and grouting process. The resulting reinforced grout column hardens and forms an auger cast pile.

Torque Down Piles (TDP)

Torque down piles (TDP) are full displacement, concrete-filled steel pipe piles that consist of a large diameter steel shaft with a tapered closed ended conical tip with a helix to aid in installation. TDP are drilled into the ground to depth and the steel shaft displaces the surrounding soil as it advances. Once the design depth is achieved, steel reinforcement and concrete fills the steel pipe pile. TDP achieves vertical capacity through both skin friction between the soil and the steel pipe and the end bearing of the closed end tip. Since the TDP is a displacement pile, no soil spoils are generated during construction. Additionally, TDP are drilled into the ground, not driven, therefore the construction process is relatively quiet and excess vibrations are not generated.

Vertical capacities depend on the driven depth and structural capacity of the piles. 70 kip design loads are typical, and much higher design loads can be achieved with deeper and larger diameter piles. In areas were settlement is expected to occur, “down drag” forces need to be taken into consideration. We should coordinate with the Project Structural Engineer to develop building specific deep foundation design criteria. Load testing should be performed on at least one pile to confirm the anticipated subsurface conditions and verify the design capacity has been achieved. For project planning purposes, the attached Figures 12 and 13 present ultimate static capacity versus depth and ultimate seismic capacity versus depth for various torque down pile diameters

E. Capillary Break and Vapor Barrier

We recommend that concrete slabs have a minimum thickness of six inches and be reinforced with steel reinforcing bars (not welded wire mesh). To improve interior moisture conditions, a 6-inch layer of clean, free draining, 3/4-inch angular gravel should be placed beneath the interior concrete slab to form a capillary moisture break. The rock must be placed on a properly moisture-conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 15 mils or thicker and meeting the requirements of ASTM E-1745 Class A, should be placed over the rock layer and be installed per ASTM 1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slab resulting in mold growth or other adverse conditions.

It should be noted that where the gravel capillary break layer is placed beneath floor slabs (especially a below-grade mat slab), there is a chance that water will collect in the gravel layer and become trapped. If this condition occurs, the potential for moisture problems at the surface of the slab will be increased. One method of minimizing the potential for this to occur would be to construct a subdrain trench through and just below the gravel layer so that water collected in this area can escape. The subdrain should extend at least 12 inches below the base of the slab and 6 inches below the bottom of the gravel layer, and would consist of a four inch diameter, perforated pipe (Schedule 40 PVC) surrounded by gravel. The subdrain would connect to the gravel layer beneath the slab, and the pipe should lead (at a minimum one percent slope) to a storm drain or another suitable outlet point. The outlet pipe should transition to nonperforated pipe at a point two feet inside the perimeter footing of the structure. A compacted clayey soil plug or other type of moisture barrier should be placed around the pipe at the point where the outlet pipe leaves the building footprint to prevent seepage from back-flowing into the underslab gravel layer.

The industry standard approach to floor slab moisture control, as discussed above, does not assure that floor slab moisture transmission rates will meet the building use requirements or that indoor humidity levels will be low enough to inhibit mold growth. Building design, construction, and intended use have a significant role in moisture problems and should be carefully evaluated by the owner, designer, and builder in order to meet the project requirements.

F. Site and Foundation Drainage

The site is relatively flat and there is a possibility that new grading could result in adverse drainage patterns and water ponding around buildings and other improvements. Careful consideration should therefore be given to design of finished grades at the site. We recommend that landscaped areas adjoining new structures be sloped downward at least 0.25-feet for 5-feet (5 percent) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10-feet in the first 5-feet (2 percent), except where not permitted due to ADA requirements. Roof gutter downspouts may discharge onto the pavements, but should not discharge onto any landscaped areas. Provide area drains for landscape planters adjacent to buildings and parking areas and collect downspout discharges into a nonperforated pipe collection system. Site drainage improvements should be connected into the existing campus storm drainage system.

G. Exterior Concrete Slabs-on-Grade

Where exterior concrete slabs or walkways are constructed, we recommend they be at least 6-inches thick and reinforced with steel bars (not wire mesh). Concrete slabs should be underlain by at least 6-inches of Caltrans Class 2 Aggregate Baserock compacted to at least 92 percent relative compaction. Additionally, contraction joints should be incorporated in the concrete slab in both directions, no greater than 8 feet on center, and the reinforcing bars shall extend through the control joints. We recommend that concrete walkways should be doweled to the perimeter foundation of the building to minimize the risk that differential settlement at this connection will result in a vertical offset (tripping hazard). The Structural Engineer should provide the design details for concrete flatwork.

H. Utility Trench Backfills

Excavations for utilities will typically be in loose to medium dense sandy soils. Trench excavations having a depth of five feet or more and will be entered by workers must be sloped, braced, or shored in accordance with current Cal/OSHA regulations. On site soils appear to be Type C. All excavations where collapse of excavation sidewall, slope or bottom could result in injury or death of workers should be evaluated by the contractor's safety officer and designated competent person prior to entering in accordance with current Cal/OSHA regulations.

Bedding materials for utility pipes should be well graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than 5 percent finer than the No. 200 sieve. Provide the minimum bedding beneath the pipe in accordance with the manufacturer's recommendation, typically 3 to 6-inches. Trench backfill may consist of on-site soils, moisture conditioned and placed in thin lifts and compacted to at least 90 percent. Backfill for trenches within pavement areas should consist of non-expansive granular fill. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits. Where utility lines cross under or through perimeter footings, they should be sealed to reduce moisture intrusion into the areas under the slabs and/or footings.

VI. SUPPLEMENTAL GEOTECHNICAL SERVICES

We must review the design plans and specifications for the project when they are nearing completion to confirm that the intent of our geotechnical recommendations has been incorporated and in the project plans, and to provide supplemental recommendations, if needed. During construction, we must observe and test site grading, foundation excavations/installation of deep foundations for the structures, and associated improvements to confirm that the soil conditions encountered during construction are consistent with the design criteria and that the work is being performed in accordance with our recommendations and the approved plans and specifications for the project.

VII. LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical engineering practices in the San Francisco Bay Area at the time the report was prepared. This report has been prepared for the exclusive use of the Alameda Unified School District and/or its assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soil in this geographic area.

Our approved scope of work did not include an environmental assessment of the site. Consequently, this report does not contain information regarding the presence or absence of toxic or hazardous wastes.

The evaluations and recommendations do not reflect variations in subsurface conditions that may exist between CPT locations or in unexplored portions of the site. Should such variations become apparent during construction, the general recommendations contained within this report will not be considered valid unless MPEG is given the opportunity to review such variations and revise or modify our recommendations accordingly. No changes may be made to the general recommendations contained herein without the written consent of MPEG.

We recommend that this report, in its entirety, be made available to project team members, contractors, and subcontractors for informational purposes and discussion. We intend that the information presented within this report be interpreted only within the context of the report as a whole. No portion of this report should be separated from the rest of the information presented herein. No single portion of this report shall be considered valid unless it is presented with and as an integral part of the entire report.

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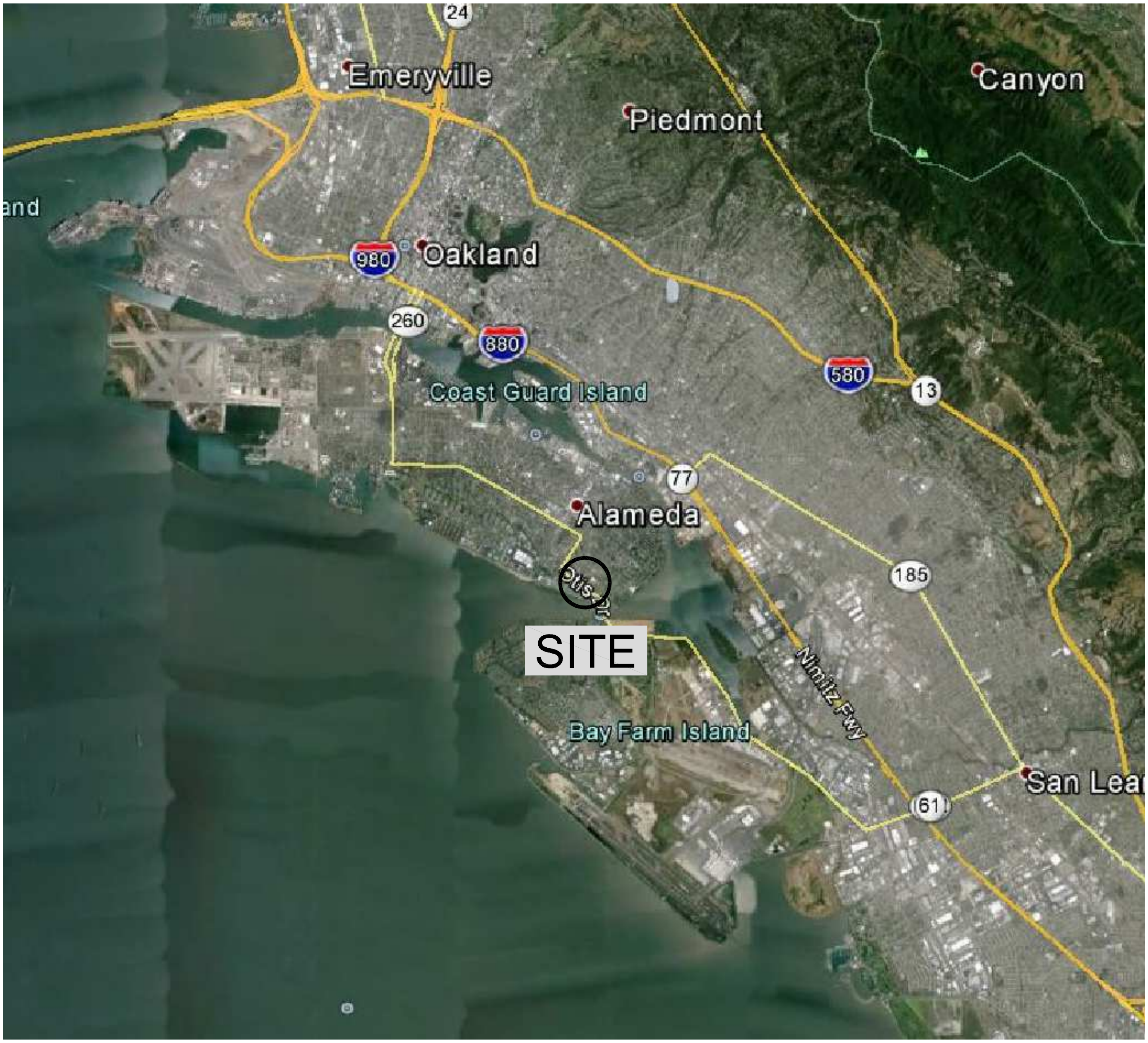


SITE: LATITUDE, 37.7534°
LONGITUDE, -122.2377°

SITE LOCATION

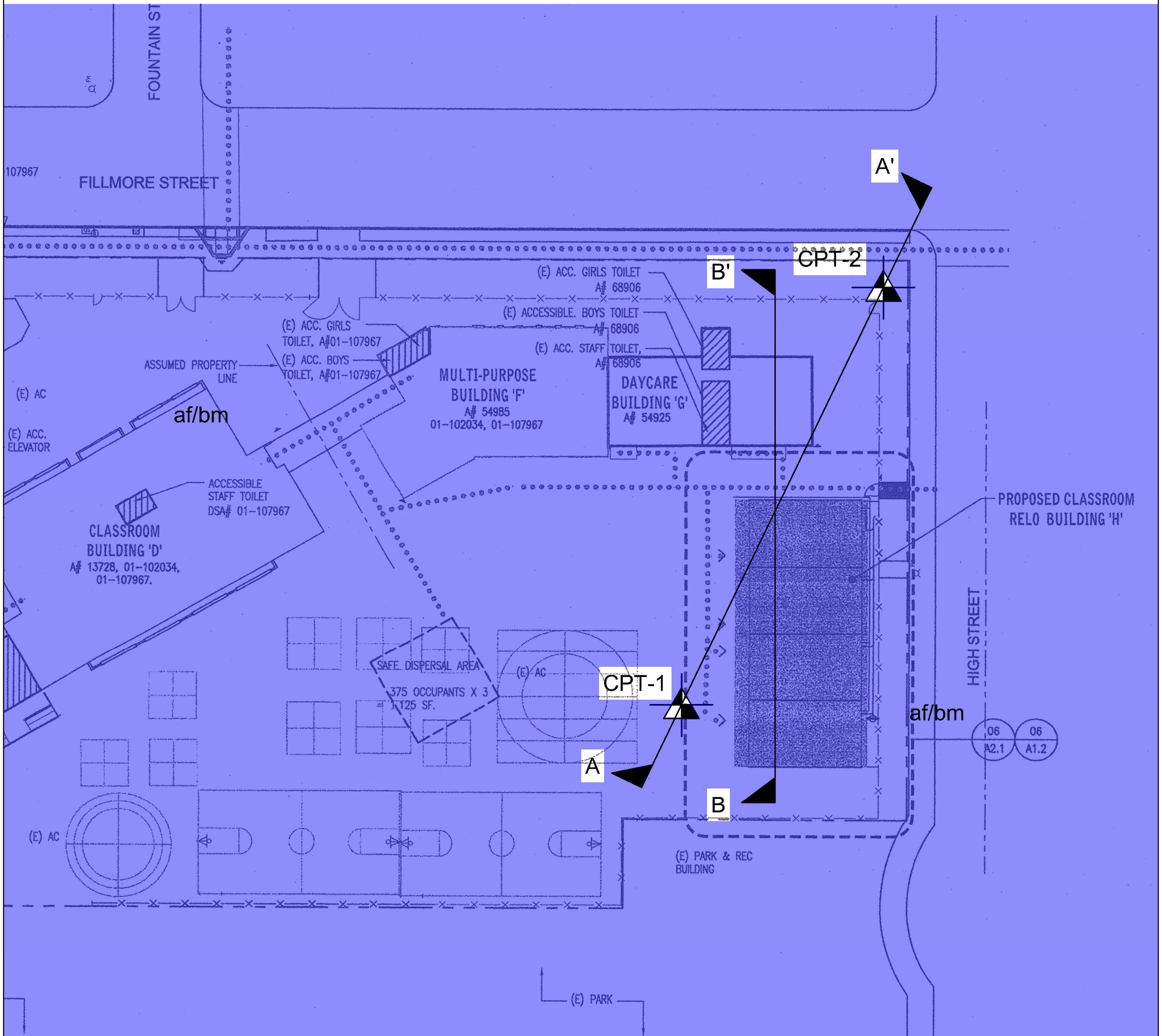
N.T.S.

NORTH

REFERENCE: Google Earth, 2015

Miller Pacific ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SITE LOCATION MAP		
		Alameda Unified School Dist. Otis Elementary School Alameda, California	Drawn MMT Checked	1 FIGURE
		Project No. 1911.024 Date: 9/9/15		

FILE: 1911.024 SLM.dwg



SITE PLAN

MPEG CPT Location, July 2015

af/bm Artificial Fill over Bay Mud

Cross-Section

NORTH

SCALE

0 30 60 120 FEET

REFERENCE: Site Plan Provided By LPA

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FILE: 1911.024 SP.dwg

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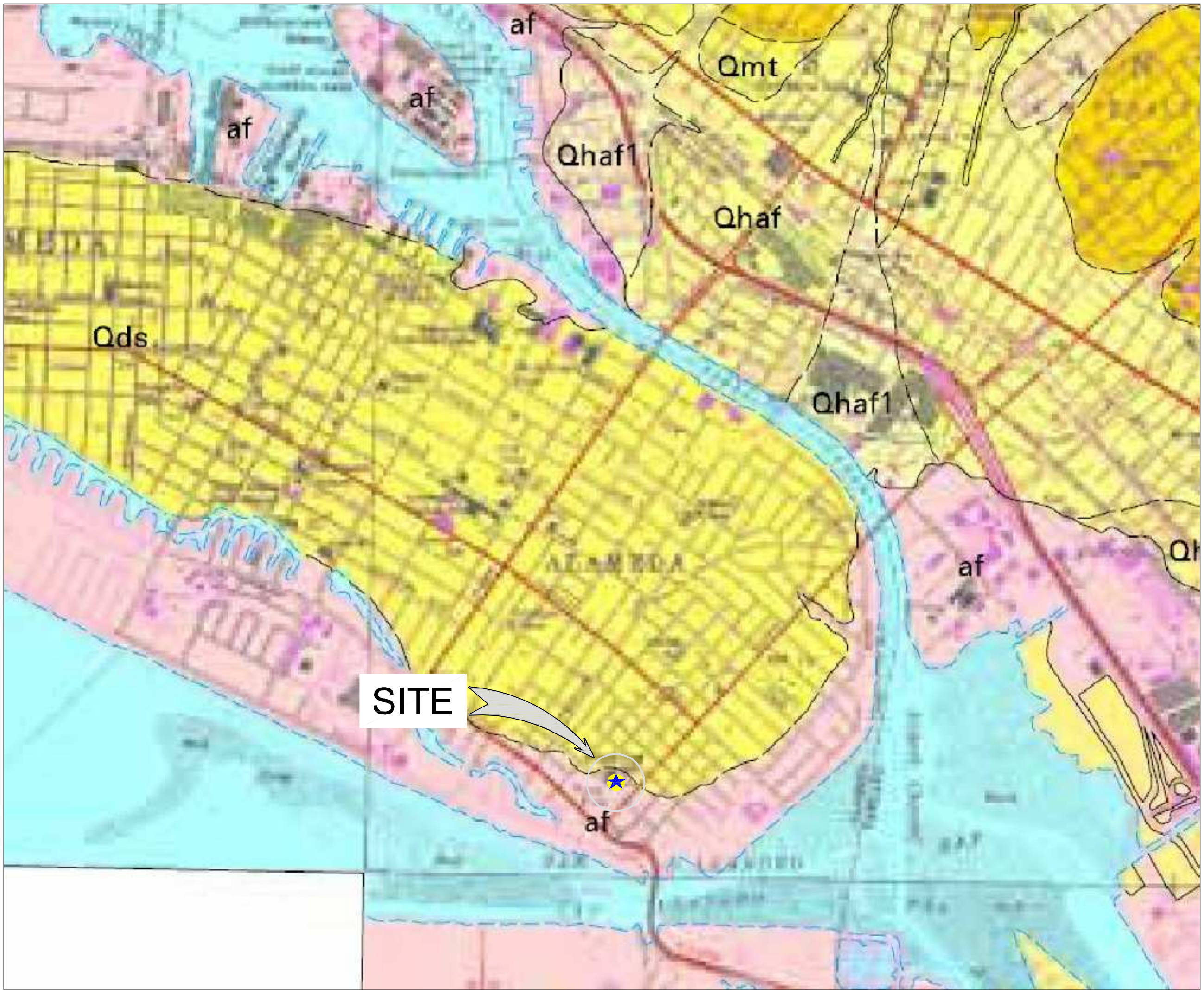
SITE PLAN

Alameda Unified School Dist.
Otis Elementary School
Alameda, California

Project No. 1911.024 Date: 9/9/15

Drawn NGK
Checked

2
FIGURE



REGIONAL GEOLOGIC MAP

NO SCALE

NORTH

KEY TO MAP UNITS:

af	ARTIFICIAL FILL (HOLOCENE) Man made deposit of various materials and ages. Some are compacted and quite firm, but fills made before 1865 are nearly everywhere not compacted and consist simply of dumped materials.
Qds	DUNE SAND (HOLOCENE AND PLEISTOCENE) Fine-grained, very well sorted, well-drained, eolian deposits.They occur mainly in large sheets, as well as many small hills, most displaying Barchan morphology.Dunes display as much as 30 m of erosional relief and are presently being buried by basin deposits (Qhb) and bay mud (Qhbm). They probably began accumulating after the last interglacial high stand of sea level began to recede about 71 ka, continued to form when sea level dropped to its Wisconsin minimum about 18 ka, and probably ceased to accumulate after sea level reached its present elevation (about 6 ka). Atwater (1982) recognized buried paleosols in the dunes, indicating periods of nondeposition

REFERENCE: Graymer, R.W. (2000), "Geologic Map of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California", United States Geological Survey Miscellaneous Field Studies Map MF-2342, Version 1.0, Map Scale 1:50,000.

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FILE: 1911.024 Geology Map.dwg

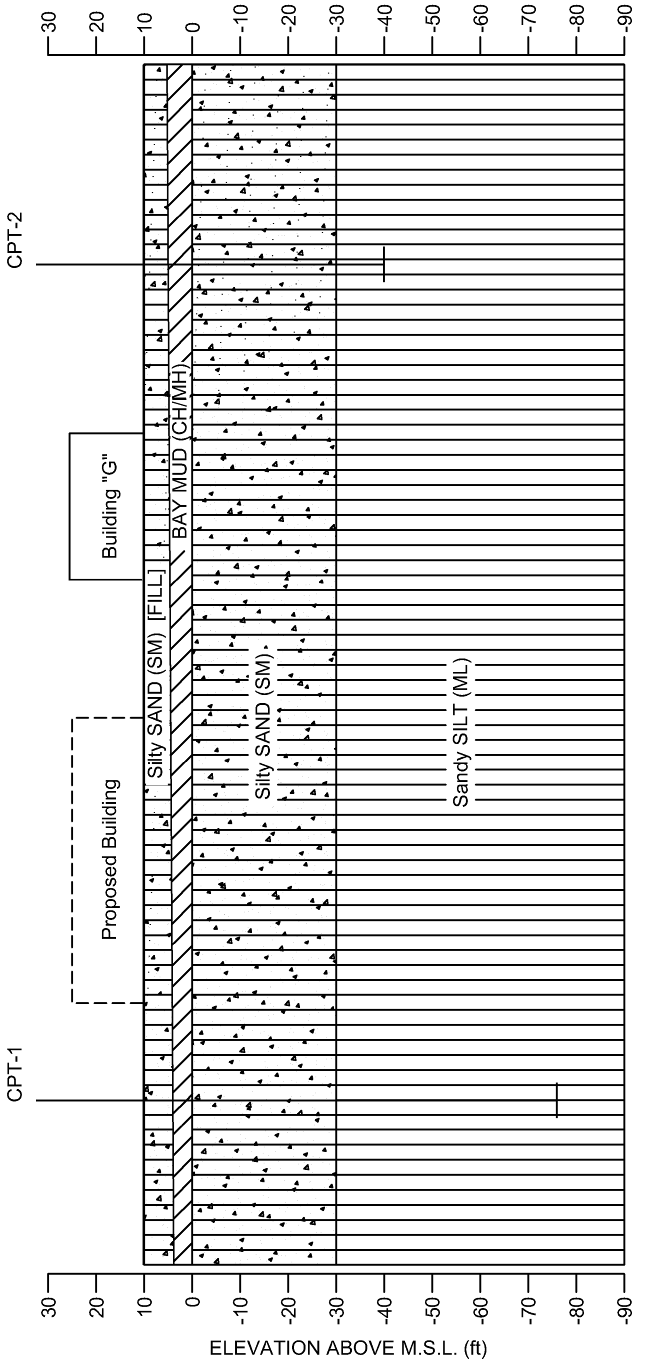
REGIONAL GEOLOGIC MAP

Alameda Unified School Dist.
Otis Elementary School
Alameda, California

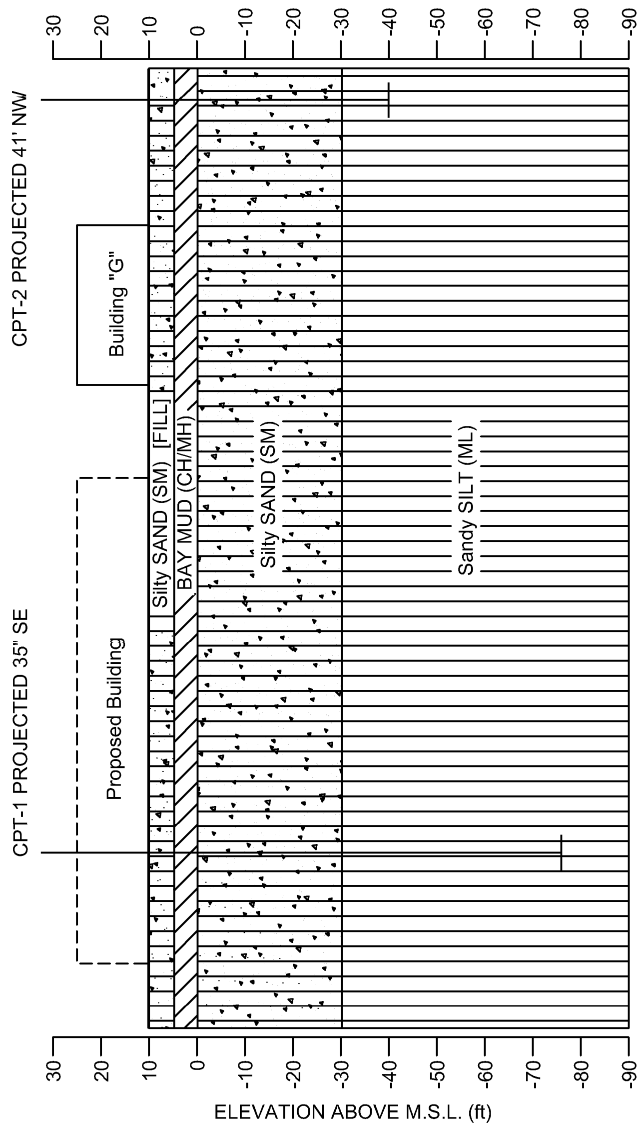
Project No. 1911.024 Date: 9/9/15

Drawn NGK
Checked

3
FIGURE



SECTION A-A'



SECTION B-B'

SCALE

0 30 60 120 FEET

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SIMPLIFIED GEOLOGIC CROSS-SECTIONS

Alameda Unfied School Dist.
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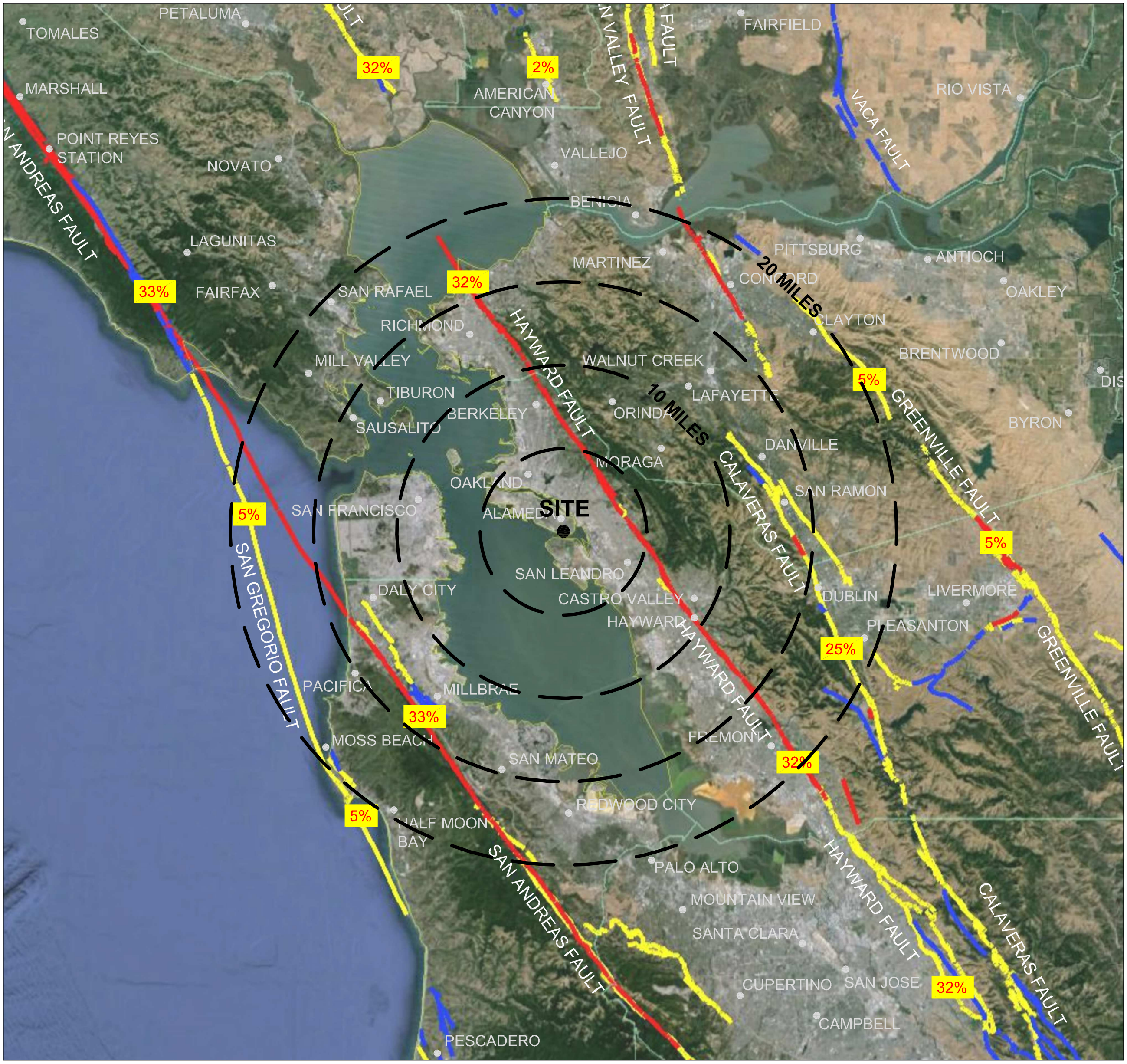
Project No. 1911.024 Date: 9/9/15

Drawn NGK
Checked

4
FIGURE



SITE COORDINATES: LAT. 37.7534, LON. -122.2377

LEGEND

(COLOR INDICATES AGE OF MOST RECENT KNOWN MOVEMENT)

HISTORIC (<150 YEARS)

HOLOCENE (<11,000 YEARS)

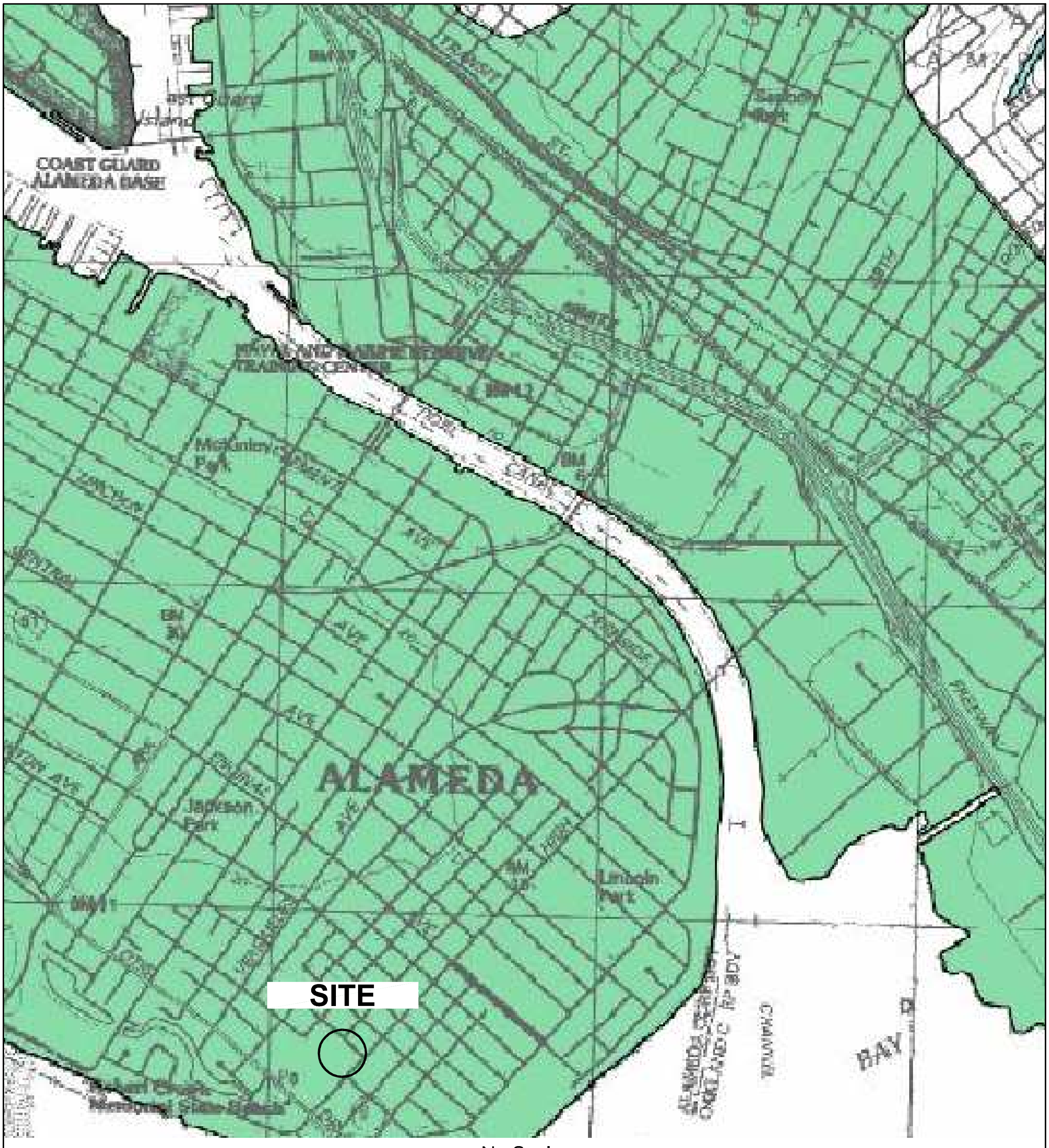
LATE QUATERNARY (<1.0M YEARS)

32% PROBABILITY OF AT LEAST ONE M>6.7 EARTHQUAKE BETWEEN 2015 AND 2045 FOR FAULTS SHOWN

DATA SOURCE:

1) Working Group on California Earthquake Probabilities (WGCEP)(2014), "Long-Term Time-Dependent Probabilitites for the Third Uniform California Earthquake Rupture Forecast (UCERF3), Bulletin of the Seismological Society of America (BSSA), Volume 105, No. 2A, 33pp, April 2015.

Miller Pacific ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	ACTIVE FAULT MAP
A CALIFORNIA CORPORATION, © 2013, ALL RIGHTS RESERVED FILE: 1911.024 FM.dwg		Alameda Unified School Dist. Otis Elementary School Alameda, California Project No. 1911.024 Date: 9/9/15
Drawn MMT Checked		5 FIGURE



Legend:

No Scale

Areas where historical occurence of liquefaction, or local gelogical, geotechnical and ground-water conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Local Road

NORTH

Map Reference: Davis, James F., "State of California Seismic Hazard Zones, Oakland East and Part of Las Trampas Ridge Quadrangles, Official Map," California Geological Survey, February 14, 2003, Scale 1:24,000.

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FILE: 1911.024 Liq.dwg

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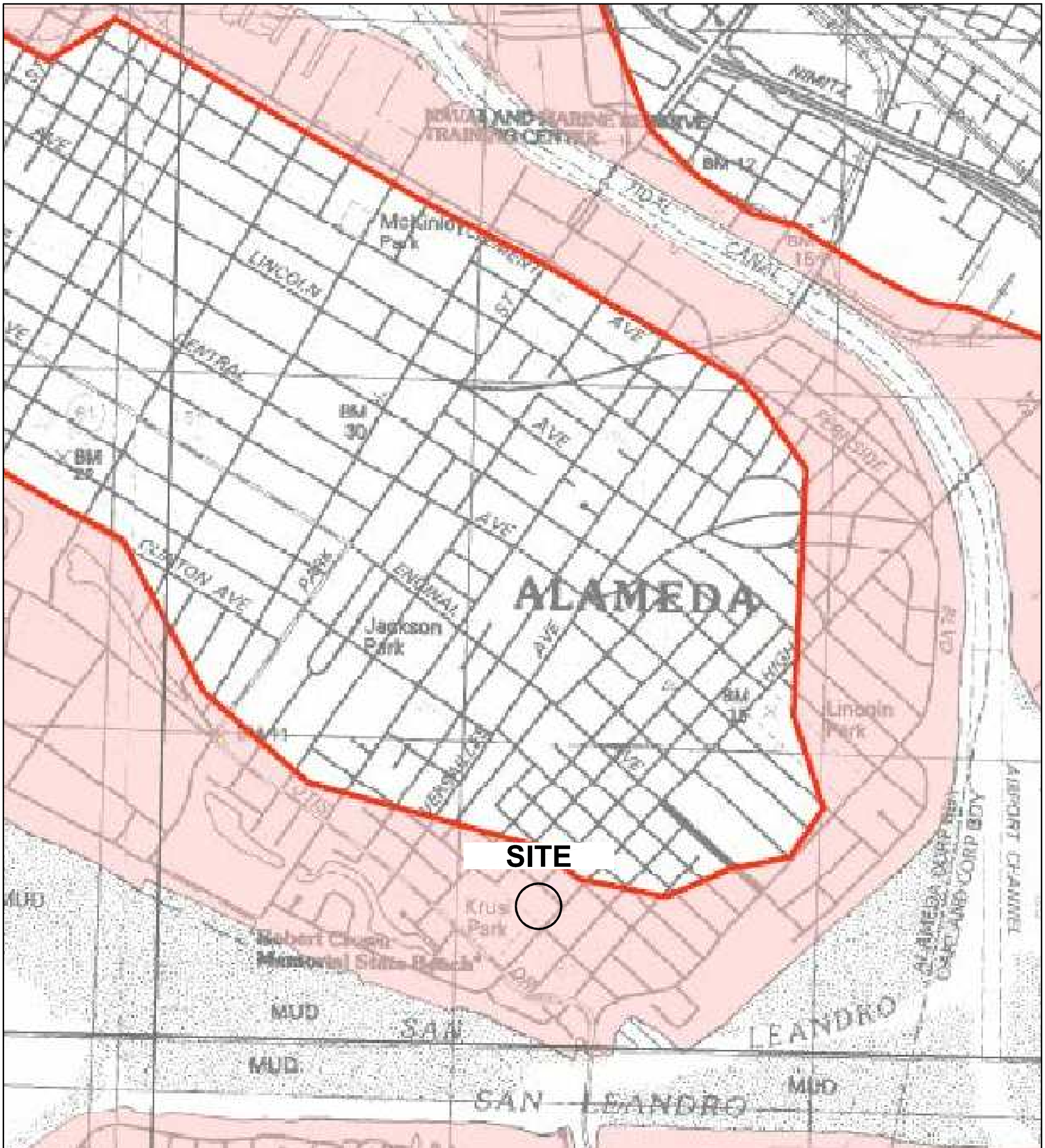
LIQUEFACTION SUSCEPTIBILITY MAP

Alameda Unified School District
Otis Elementary School
Alameda, California

Project No. 1911.024 Date: 9/9/15

Drawn NGK
Checked

6
FIGURE



Tsunami Inundation:

No Scale

Tsunami Inundation Area

Tsunami Inundation Line

NORTH

Map Reference: "Tsunami Inundation Map for Emergency, Planning, State of California ~ County of Alameda, Oakland East Quadrangle," California Emergency Management Agency, California Geological Survey, University of Southern California, July 31, 2009, Scale 1:24,000.

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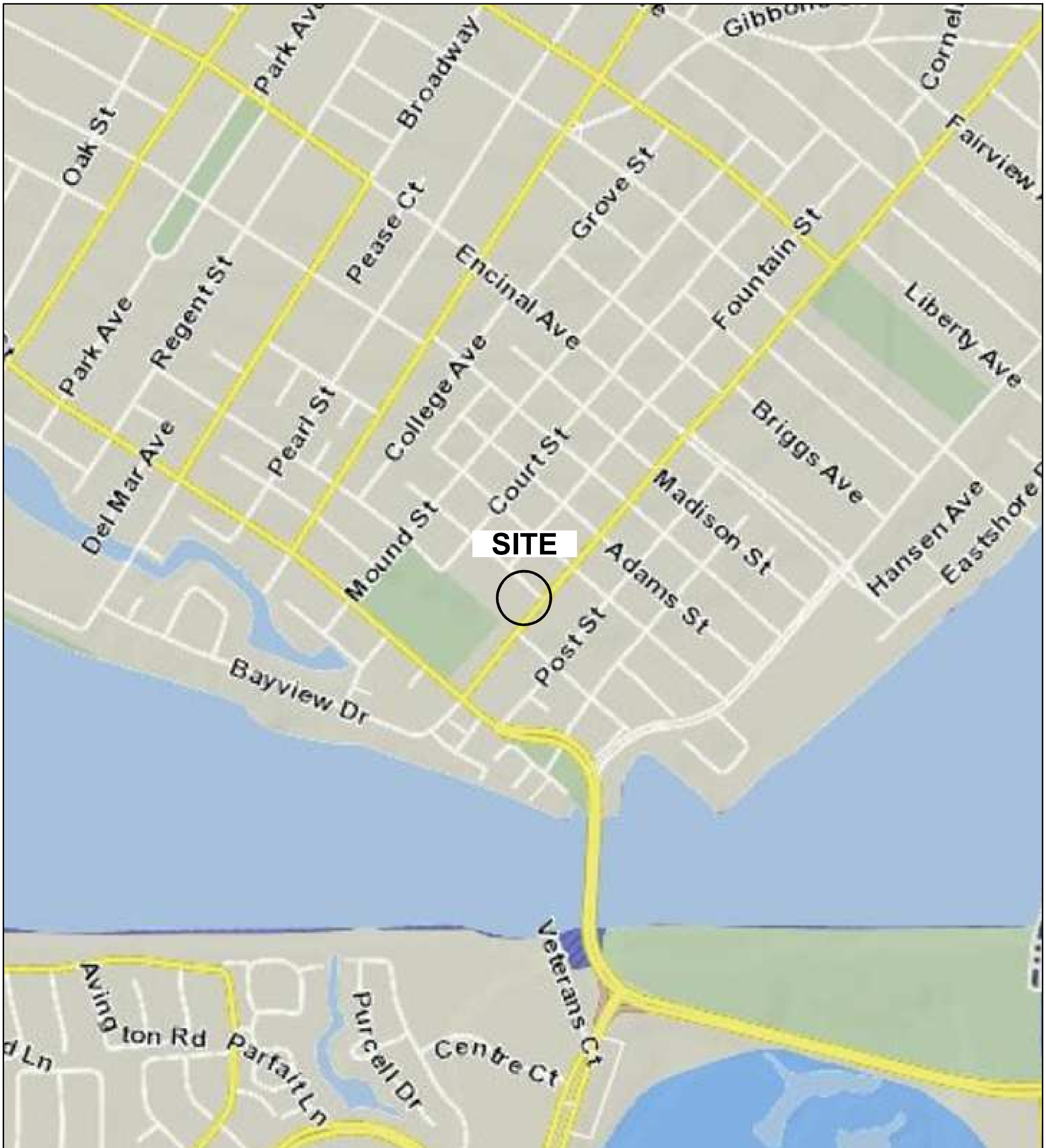
FILE: 1911.024 Tsunami.dwg



Flood Hazard Area:

Zone V- 100yr. | Zone X - 500yr.

Zone A - 100yr. | Urbanized Area

Map Reference: ABAG Geographic Information System.

No Scale

Zone V: This code identifies an area inundated by 1% annual chance flooding with velocity hazard (wave action).
Zone A: This code identifies an area inundated by 1% annual chance flooding.

Zone X 500yr: This code identifies an area inundated by .02% annual chance flooding and area inundated by 1% annual chance of flooding with average depth of less than 1 foot of with drainage areas less than 1 square mile or an area protected by levees form 1% annual chance flooding.

NORTH

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FILE: 1911.024 Flood.dwg

FEMA DIGITAL FLOOD INSURANCE RATE MAP (DFIRM)

Alameda Unified School Dist.
Otis Elementary School
Alameda, California

Project No. 1911.024 Date: 9/9/15

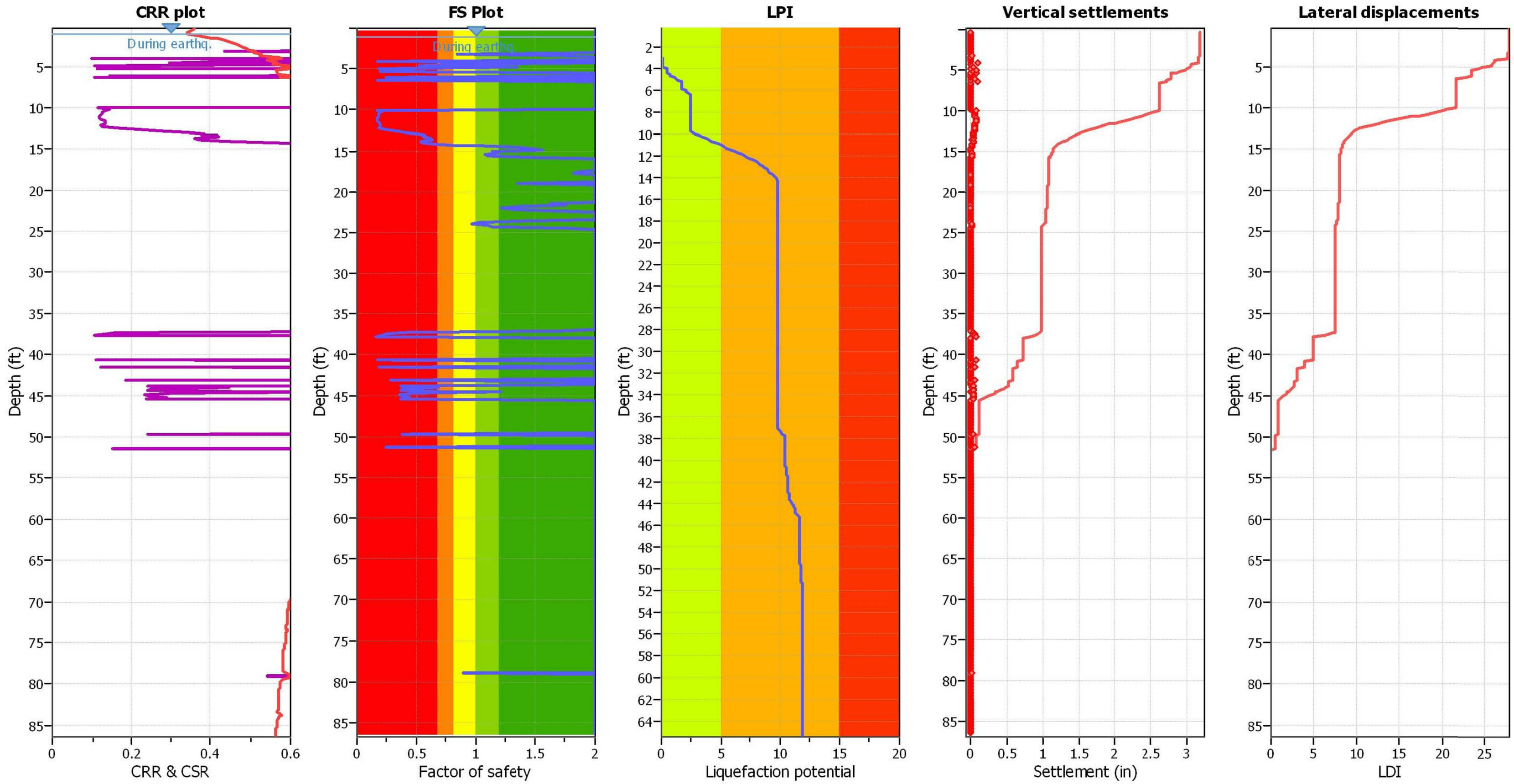
Drawn NGK
Checked

8
FIGURE

This software is licensed to: Miller Pacific Engineering Group

CPT name: CPT-01

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

CLiq v.1.7.6.49 - CPT Liquefaction Assessment Software - Report created on: 7/29/2015, 11:40:36 AM
Project file: H:\Jobs\1900-1999\1911.024 - Otis Elementary\1911.024 CPT Liquefaction.clq

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FILE: 1911.024 CPT Liq.dwg

LIQUEFACTION POTENTIAL - CPT-1

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Alameda, California

Project No. 1911.024 Date: 9/9/15

Drawn MMT
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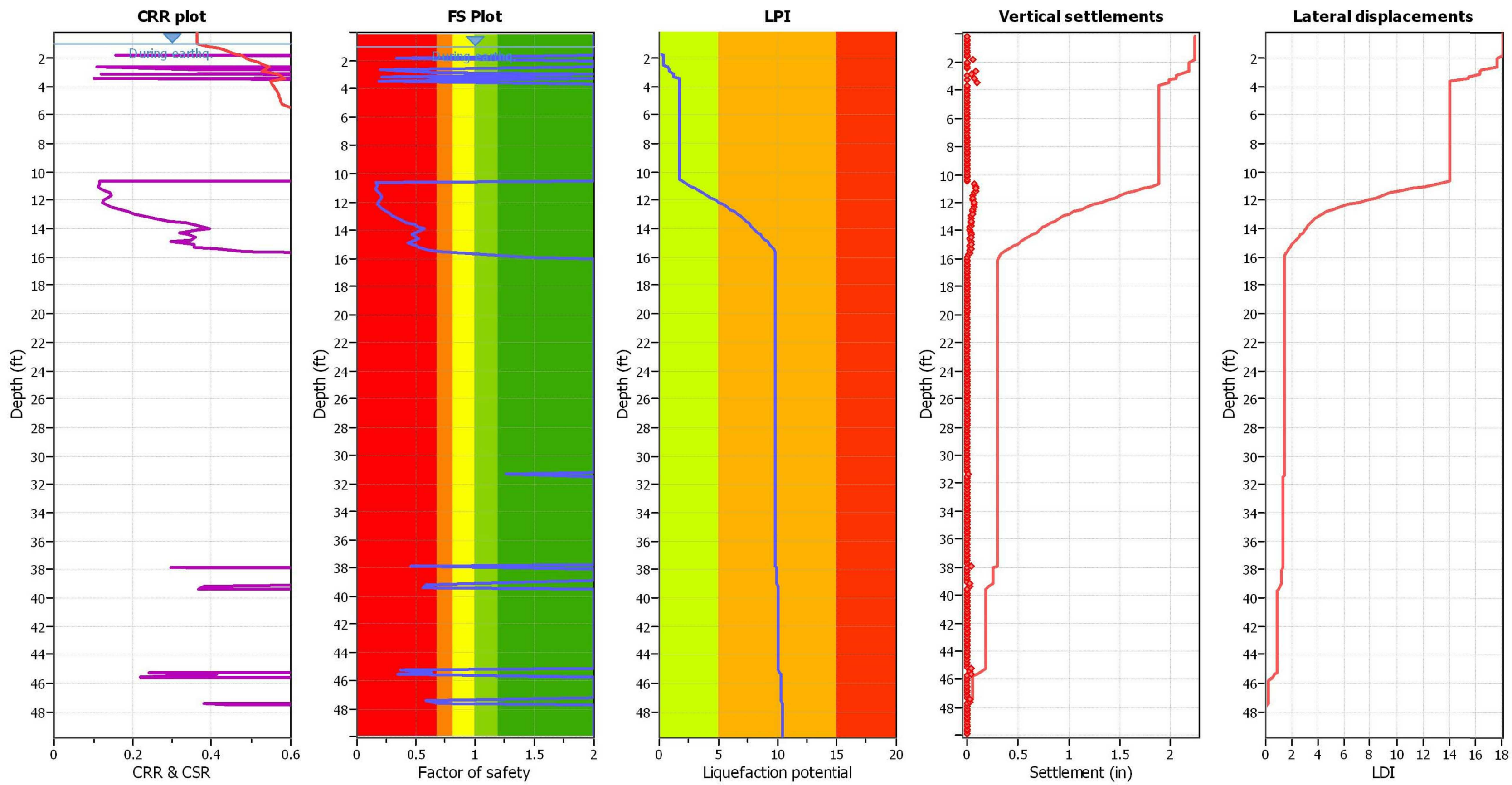
9
FIGURE

This software is licensed to: Miller Pacific Engineering Group

CPT name: CPT-02

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

CLiq v.1.7.6.49 - CPT Liquefaction Assessment Software - Report created on: 7/29/2015, 11:40:37 AM
Project file: H:\Jobs\1900-1999\1911.024 - Otis Elementary\1911.024 CPT Liquefaction.clq

14

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FILE: 1911.024 CPT Liq.dwg

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LIQUEFACTION POTENTIAL - CPT-2

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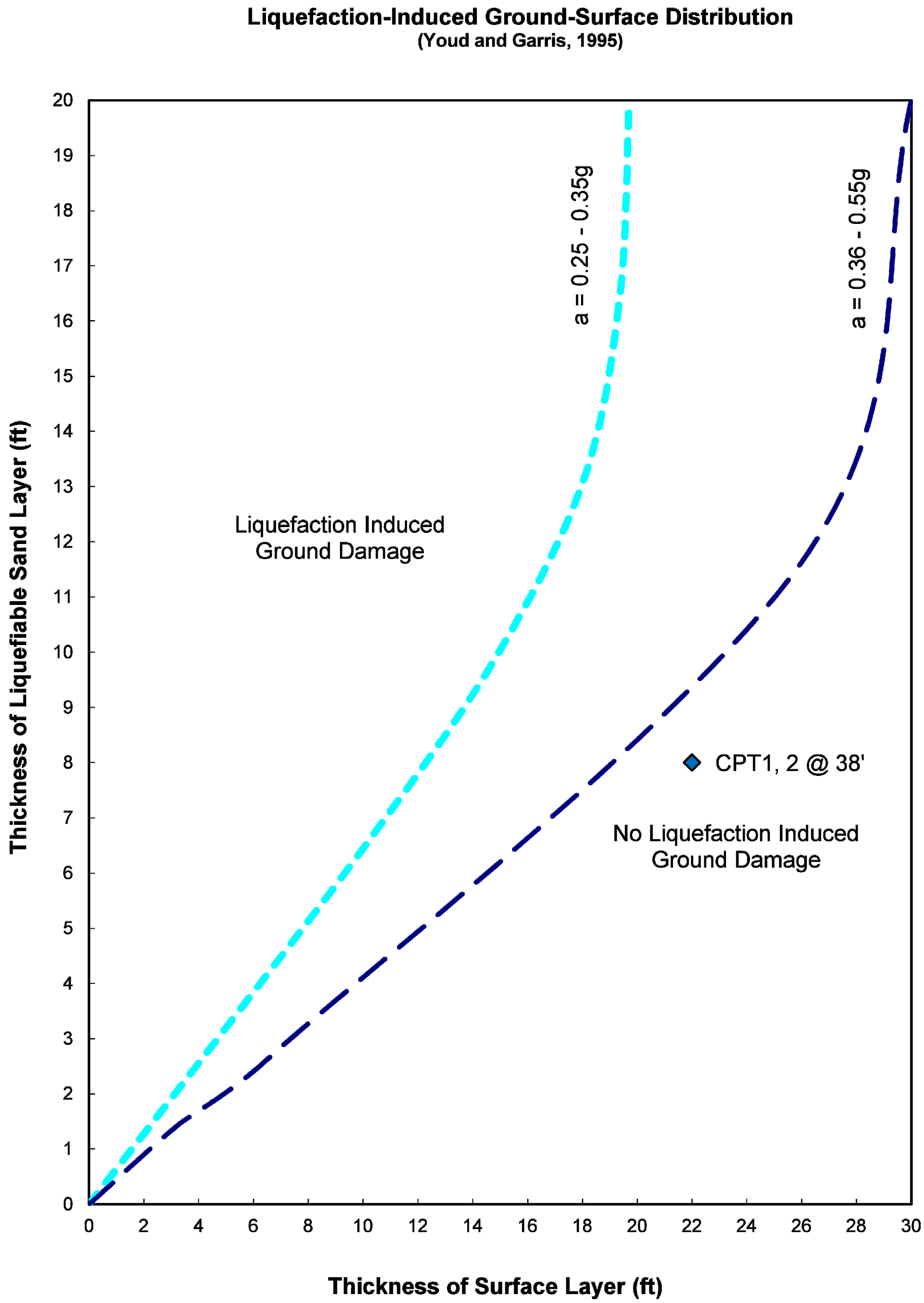
10
FIGURE



Miller Pacific ENGINEERING GROUP	504 Redwood Blvd.	LIQUEFACTION ANALYSIS - SURFACE EFFECTS		
	Suite 220			
	Novato, CA 94947	Alameda Unified School Dist.	Drawn MMT	11
	T 415 / 382-3444	Otis Elementary School	Checked	
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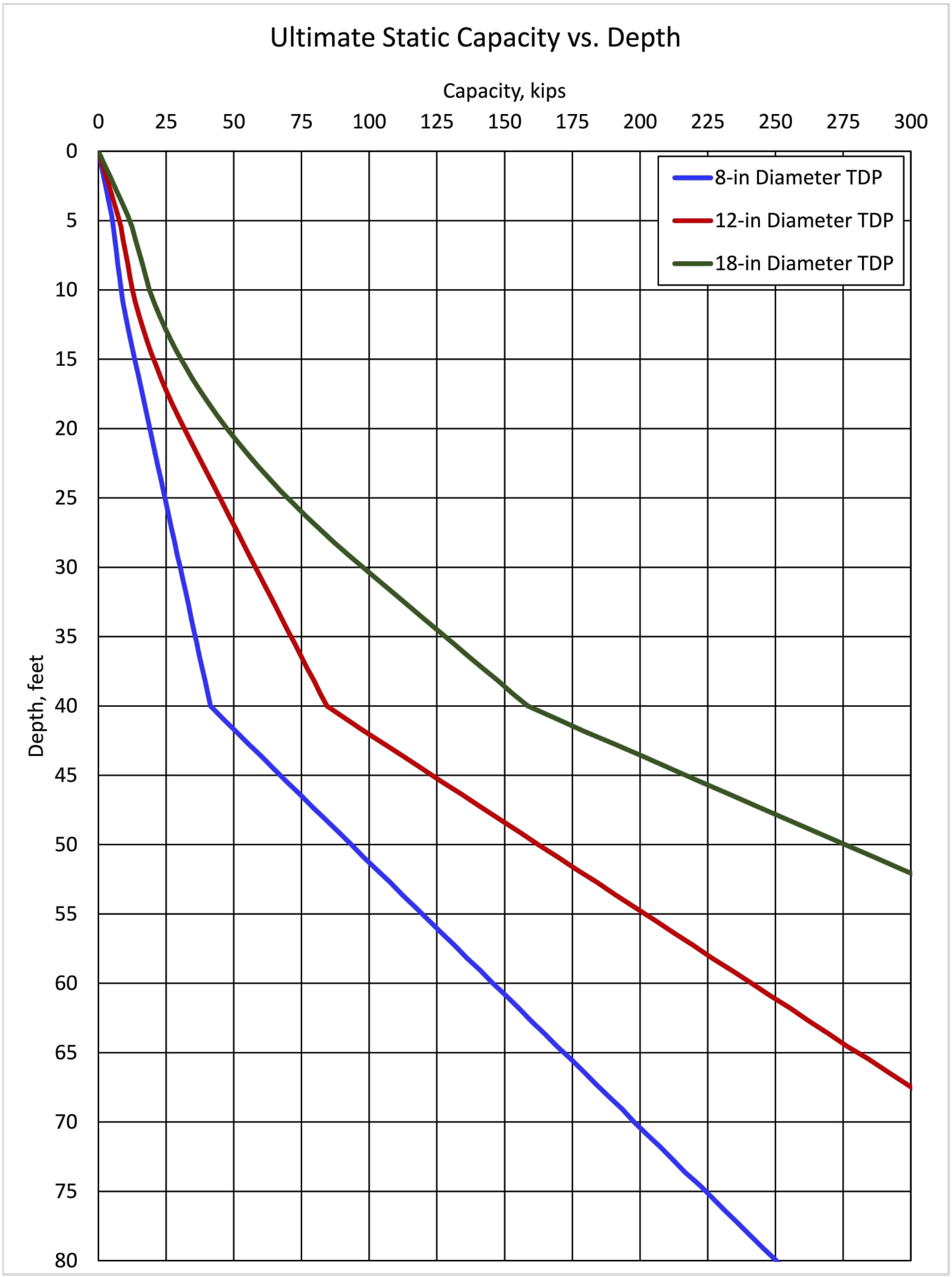
FILE: 1911.024 CPT Liq.dwg	www.millerpac.com	Project No. 1911.024 Date: 9/9/15		FIGURE



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FILE: 1911.024 TDP.dwg

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TORQUE DOWN PILE - CAPACITY vs. DEPTH (STATIC)

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Project No. 1911.024 Date: 9/4/15

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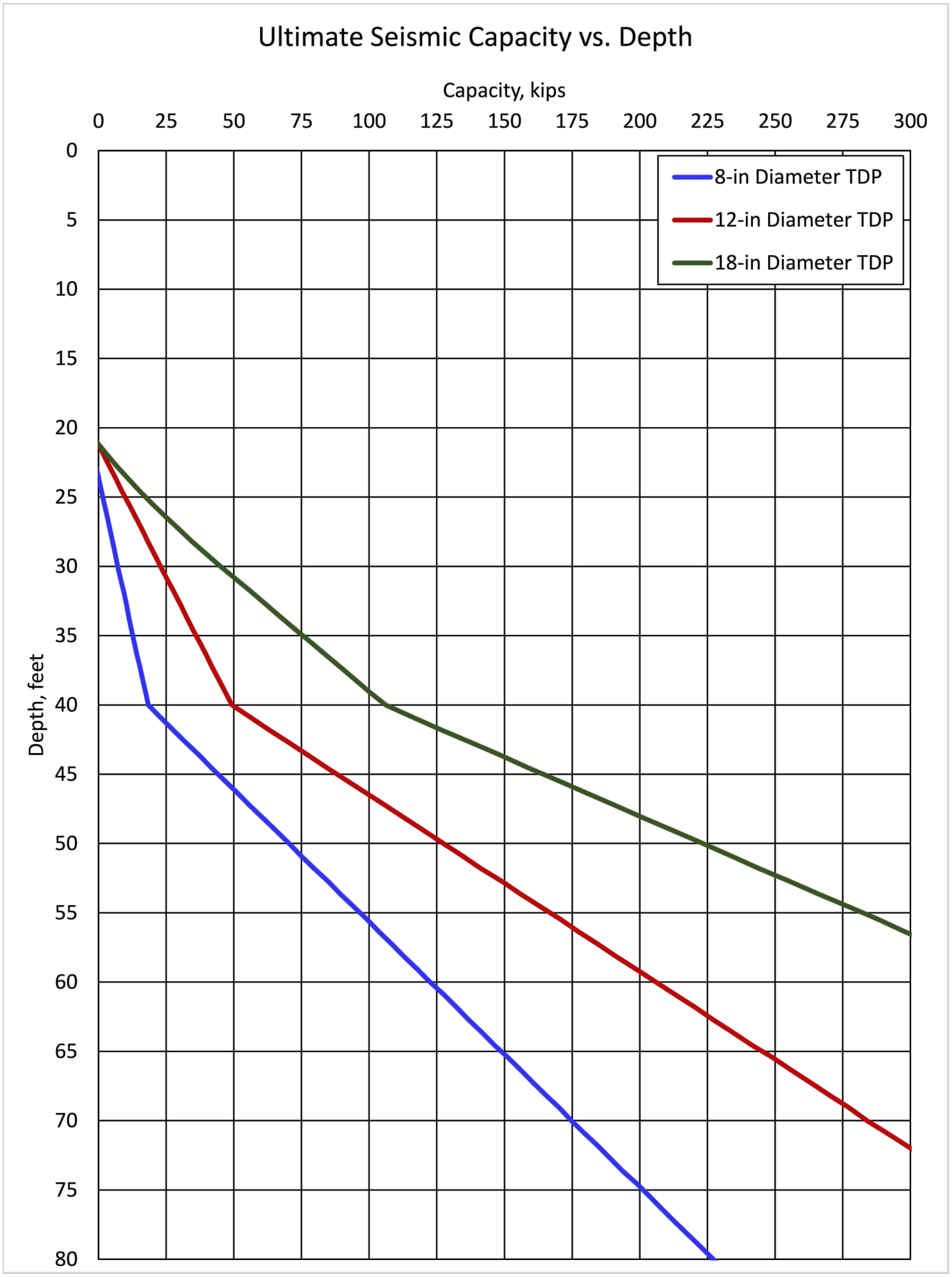
12
FIGURE



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FILE: 1911.024 TDP.dwg

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TORQUE DOWN PILE - CAPACITY vs. DEPTH (SEISMIC)

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13
FIGURE

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APPENDIX A
SUBSURFACE EXPLORATION

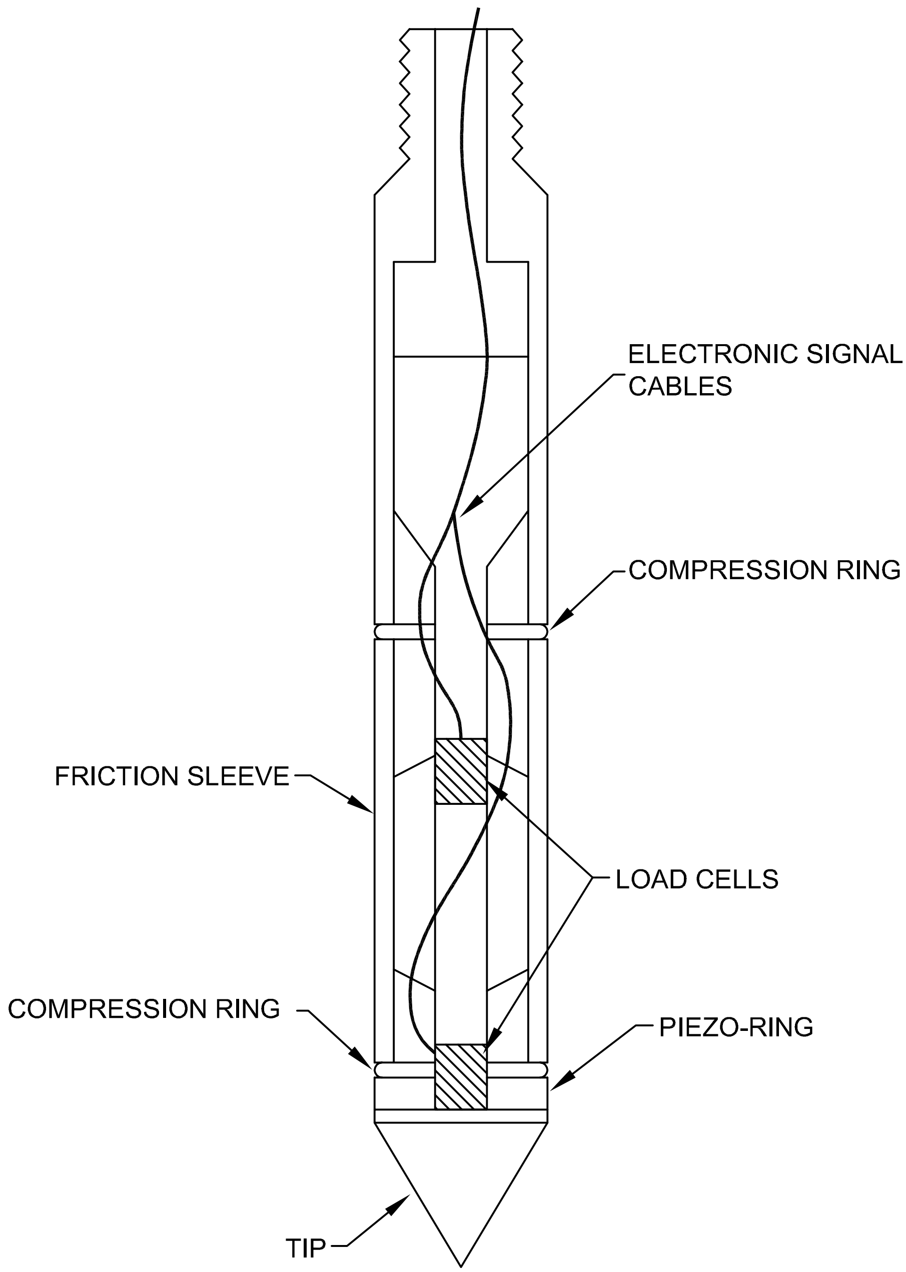
Cone Penetration Testing

The Cone Penetration Test (CPT) is a special exploration technique that provides a continuous profile of data throughout the depth of exploration. It is particularly useful in defining stratigraphy, relative soil strength and in assessing liquefaction potential. We performed two CPT's with truck-mounted equipment on July 21, 2015 for the express purpose of preparing this report.

The CPT is a cylindrical probe, 35 mm in diameter, which is pushed into the ground at a constant rate of 2 cm/sec. The device is illustrated on Figure A-1. It is instrumented to obtain continuous measurements of cone bearing (tip resistance) and sleeve friction. The data is sensed by strain gages and load cells inside the instrument. Electronic signals from the instrument are continuously recorded by an on-board computer at the surface, which permits an initial evaluation of subsurface conditions during the exploration.

The recorded data is transferred to an in-office computer for reduction and analysis. The analysis of cone bearing and sleeve friction (i.e. friction ratio) indicates the soil type, the cone bearing alone indicates soil density or strength, and the pore pressure indicates the presence of clay. Variations in the data profile indicate changes in stratigraphy. This test method has been standardized and is described in detail by the ASTM Standard Test Method D3441 "Deep, Quasi-Static Cone and Friction Cone Penetration Tests of Soil." The interpretation of CPT data is illustrated on Figure A-2, and the CPT data logs are presented on Figures A-3 and A-4.

It should be noted that the CPT logs description of soils encountered reflect conditions only at the location of the CPTs at the time they were advanced. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate and changes in surface and subsurface drainage.



CONE PENETROMETER

(NO SCALE)

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FILE: 1911.024 Cone.dwg

CONE PENETROMETER

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Project No. 1911.024 Date: 9/9/15

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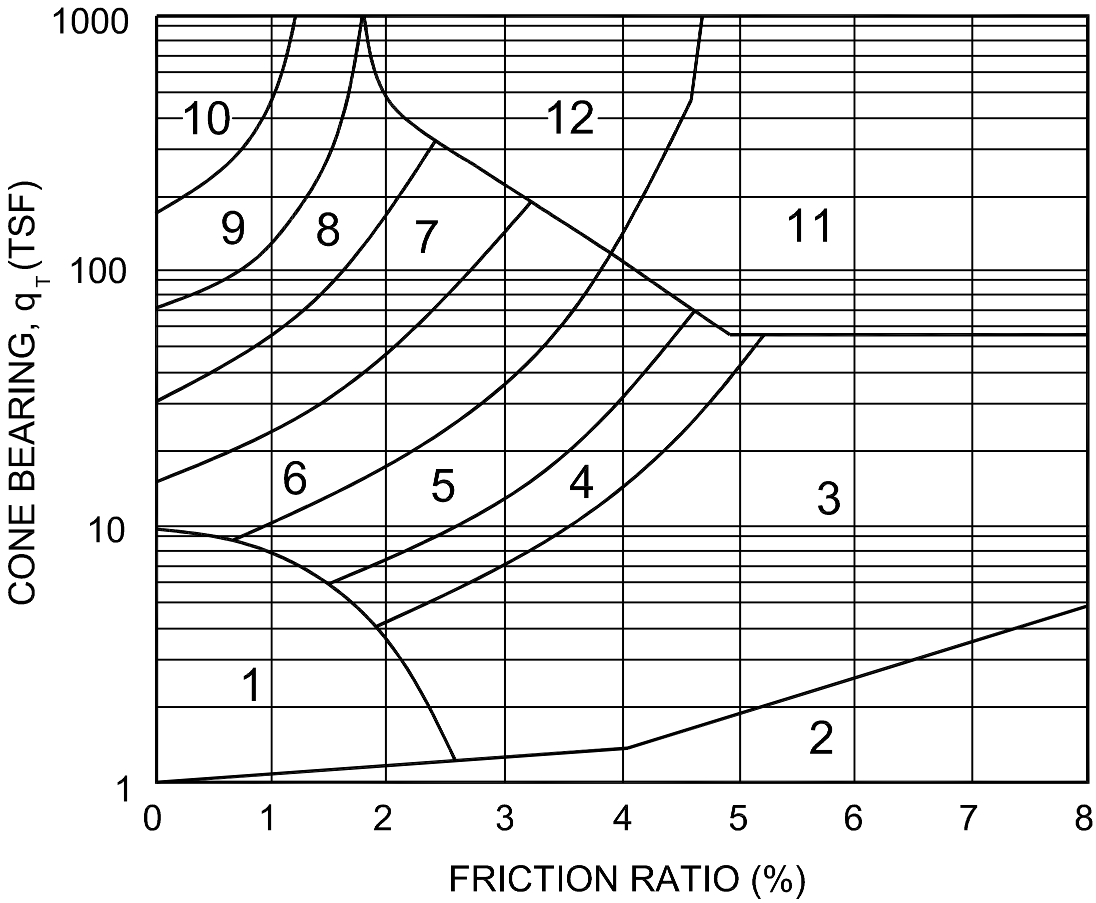
A-1
FIGURE



Zone:	Qc/N	Soil Behavior Type:
1)	2	Sensitive Fine Grained
2)	1	Organic Material
3)	1	Clay
4)	1.5	Silty Clay to Clay
5)	2	Clayey Silt to Silty Clay
6)	2.5	Sandy Silt to Clayey Silt
7)	3	Silty Sand to Sandy Silt
8)	4	Sand to Silty Sand
9)	5	Sand
10)	6	Gravelly Sand to Sand
11)	1	Very Stiff Fine Grained (*)
12)	2	Sand to Clayey Sand (*)

(*) Overconsolidated or Cemented

Reference: Robertson, P.K. (1986), "In-Situ Testing and Its Application to Geotechnical Engineering," Canadian Geotechnical Journal, Vol. 23; No. 23; No. 4, pp. 573-594

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FILE: 1911.024 CPT Chart.dwg

CPT SOIL INTERPRETATION CHART

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Project No. 1911.024 Date: 9/9/15

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A-2
FIGURE

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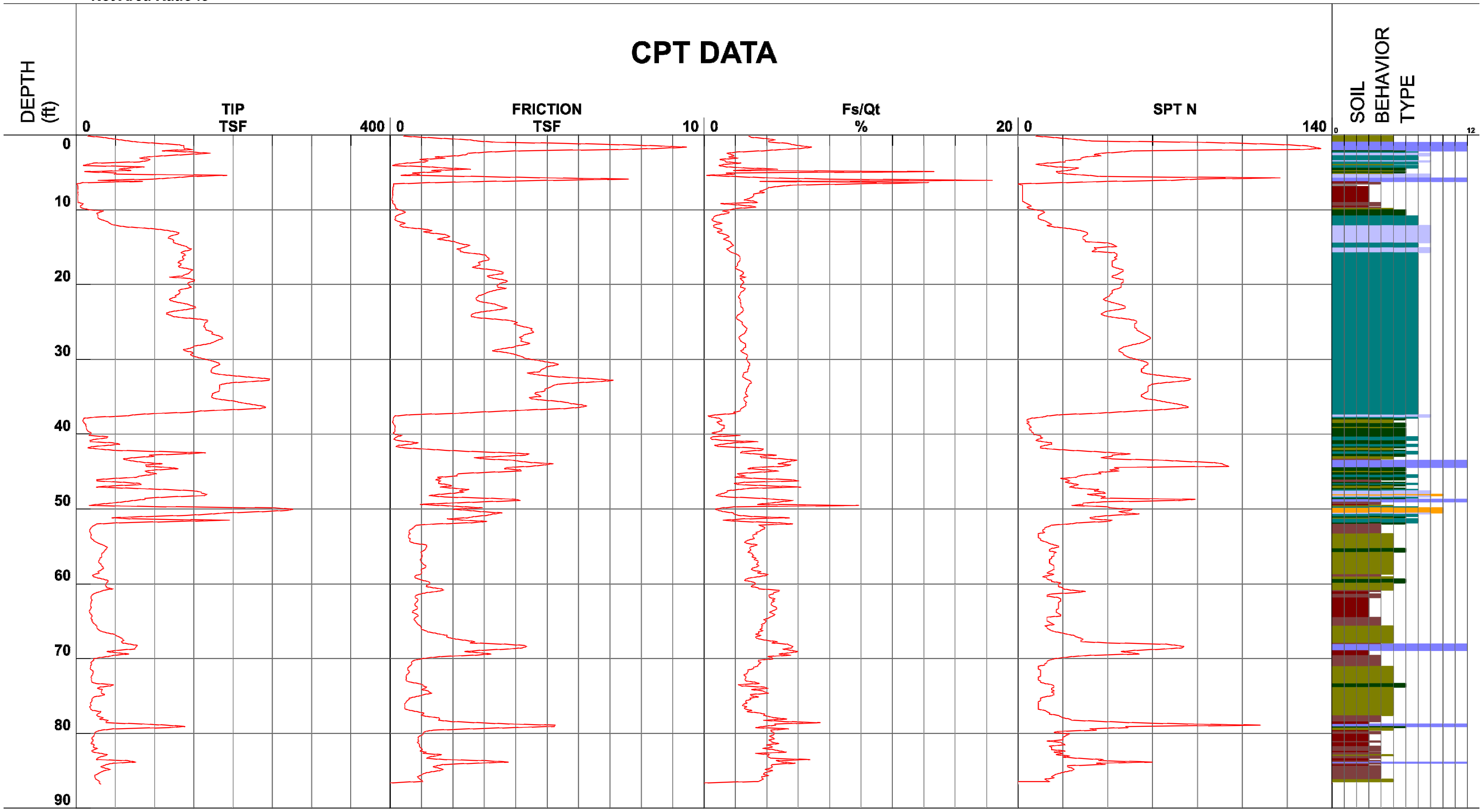
Project	Alameda Schools	Operator	CB	Filename	SDF(551).cpt
Job Number	1911.024	Cone Number	DDG1333	GPS	
Hole Number	CPT-01	Date and Time	7/21/2015 12:17:25 PM	Maximum Depth	86.78 ft
EST GW Depth During Test			6.00 ft		

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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FILE: 1911.024 CPT.dwg

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CONE PENETRATION TEST PLOT

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Project No. 1911.024 Date: 9/9/15

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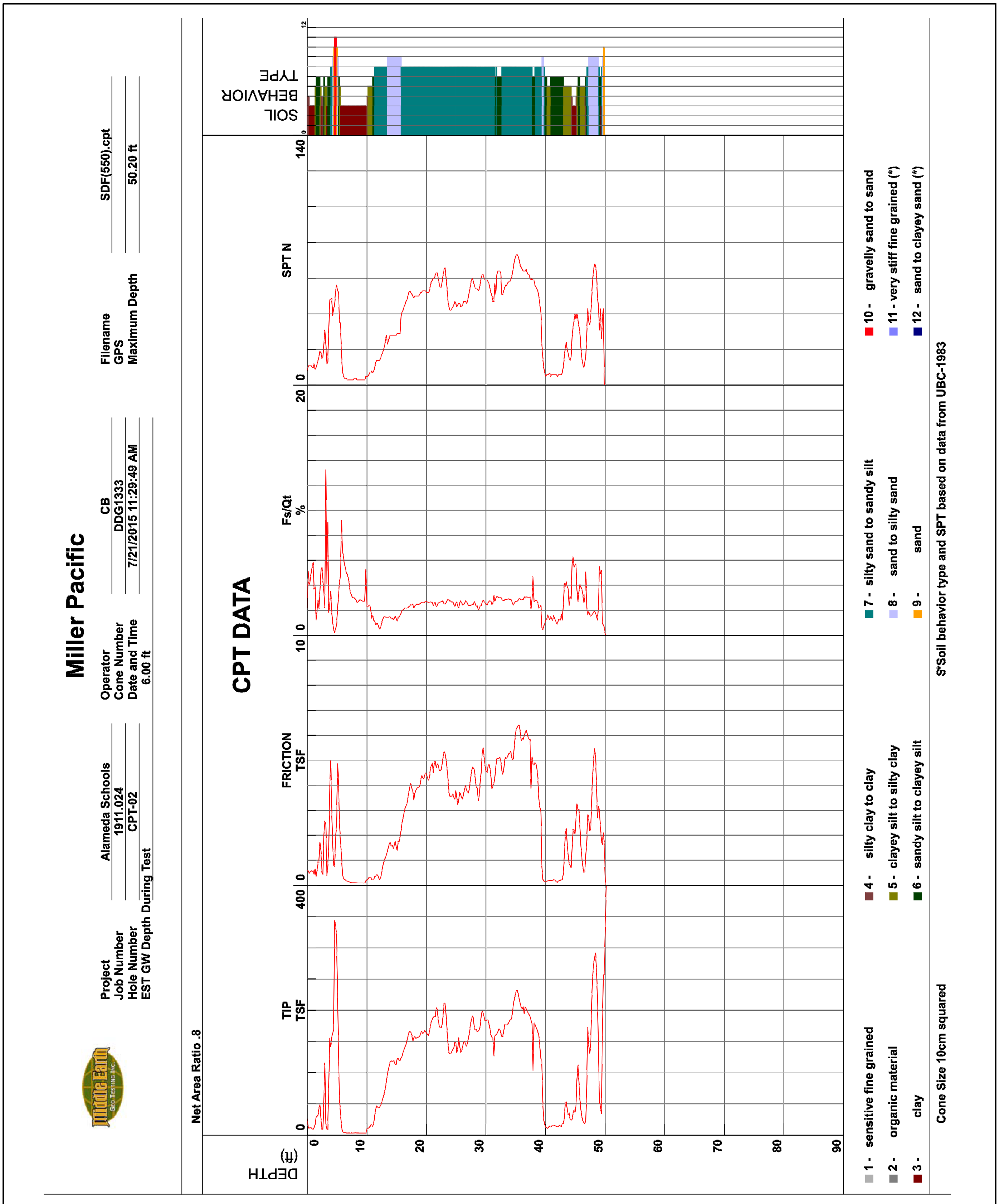
A-3
FIGURE



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CONE PENETRATION TEST PLOT

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Project No. 1911.024 Date: 9/9/15

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A-4
FIGURE

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e-mail: *entech@pacbell.net*

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-Technical Support

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COMPANY:	Miller Pacific Engineering Group, 1333 McDowell Blvd. N, Ste. C, Petaluma, CA 94954		ANALYST(S)	SUPERVISOR
ATTN:	Dan Caldwell	DATE of	D. Salinas	D. Jacobson
JOB & SITE:	Otis Elementary School, 3010 Fillmore Street,	DATE RECEIVED / COMPLETION	S. Santos	LAB DIRECTOR
PROJ.NUM.:	1911.024 Alameda, California	7/31/2015 / 8/11/2015		G.S. Conrad PhD

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL MIN. RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
06484-1	OES1-FS/A	Bulk @ 0-1'	**5.80**	**2,017**	**[496]**	**105**	**23**
Method	Detection	Limits --->	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
06484-1	OES1-FS/A	Bulk @ 0-1'				**+339.0**	
Method	Detection	Limits --->	---	0.1	0.1	1	0.1

*** COMMENTS ***

Resistivity is just over 2,000 ohm-cm which is mediocre (assign Ø-5 pts, depending on specs), and soil reaction (i.e., pH) is moderately acidic (assign Ø pts); sulfate and chloride are low enough (SO4 @ <200 ppm, assign Ø pt, Cl @ >100 ppm, assign Ø pts, all specs); redox is mild (assign Ø-3.5 pts, depending on specs). The standard CalTrans times to perforation for 18 ga galvanized in this soil are >12 yrs, and ≈22 yrs for 12 ga; and calculated <u>average</u> pitting rate (following Uhlig) is ≈0.25 mm/yr, thus pitting to a 2 mm depth is only ≈8 yrs. Chloride is not high enough to have an adverse impact on concrete reinforcement; and sulfate is not high enough to have any adverse impact on cement, concrete, grout or mortar. Lime or mild cement treatment, in principle, could be of benefit in that raising soil pH to the 7.5-8.5 range would increase 18 ga time to perforation to >33 yrs; and pitting rate would decline to ~0.084 mm/yr, thus increasing the 2 mm depth time to <24 yrs. But lime treatment only last a few years in the open environment, i.e., it is only effective in protected installations; and while cement treatment has greater longevity, it is often not practical, and has an effective lifetime as well. Otherwise, steel longevity could be improved by such things as upgrading, e.g. increased gauge or more resistant steels, etc.; or by taking other actions such as use of cathodic protection of wrapped assets; or use of increased or special engineering fill, use of a polymer coating or use of bonded plastic coated steels, or use of plastic, fiberglass or concrete assets, etc. Total point value for this soil would be in the range of Ø-8.5 pts, depending on specifications, based mostly on soil's resistivity. Standard concrete mixes should be acceptable in this soil based on these results.

\\\NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Methods of Chemical Analysis, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extraction by Title 22, and detection EPA 376.2 (= SMEWW 4500-S D); cyanides - extraction by Title 22, and detection by ASTM D 4374 (=EPA 335.2).

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FILE: 1911.024 Corrosion.dwg

LABORATORY CORROSION TEST RESULTS

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A-5
FIGURE

September 23, 2015
File: 1911.024altr.doc

Alameda Unified School District
2060 Challenger Drive
Alameda, California 94501

Attn: Mr. Robbie Lyng, Director of Operations and Facilities

Re: Addendum to Geotechnical Report Dated September 10, 2015
Recommendations for Grading and Foundation Design
Proposed Relocation of Existing Portable Buildings
Otis Elementary School Modular Classroom Project
3010 Fillmore Street
Alameda, California

Introduction

As authorized, Miller Pacific Engineering Group is providing geotechnical engineering services for the proposed Otis Elementary School Modular Classroom project. As a part of the proposed Modular Classroom project, existing portable buildings currently located in or near the footprint of the proposed two story modular classroom building will be relocated to make room for the new construction. This letter provides supplemental recommendations for grading and foundation design for the relocation of the portable buildings. The geotechnical criteria and recommendations for the new building design contained in the September 10, 2015 geotechnical report should be considered to remain valid. Our work has been performed in general accordance with our agreement dated June 24, 2015.

The attached Site Plan, Figure 1, illustrates the location of the proposed two story modular classroom building and the proposed positioning of the relocated portable buildings. It is noted that the proposed location of the portable buildings is in close proximity to the proposed modular classroom. One of the Cone Penetration Tests (CPT-1) performed during our study is located in the proposed portable building footprint area, as shown on Figure 1.

The subsurface soil profile beneath the proposed portable building relocation area is essentially the same as the soil profile beneath the proposed modular classroom, and is described in the September 10, 2015 geotechnical report. The attached Figure 2 presents two simplified geologic cross-sections illustrating the subsurface conditions beneath the proposed modular classroom and the proposed relocated portable buildings.

Conclusions and Recommendations

Based on our investigation and previous experience with similar sites and projects, we conclude that the proposed relocation of the existing portable buildings is feasible from a geologic and geotechnical standpoint.

Liquefaction analyses for the project site were performed and are presented in the September 10, 2015 geotechnical report. As discussed in the referenced report, soil layers between the ground surface and six feet, and between ten feet and sixteen feet, are subject to liquefaction during a future significant seismic event. Our analyses show that up to approximately two inches of liquefaction induced settlement could occur at the ground surface (see Figures 9 and 10 in September 10, 2015 report).

The one story portable buildings are designed to withstand some differential movement, and are constructed on frames designed to be moved on the highway. Portable buildings are typically supported on pressure treated wood foundations bearing on the ground surface. Settlement of the ground surface of up to three inches or more caused by seismic induced liquefaction settlement or soft clay consolidation settlement would not be expected to cause structural damage to the portable buildings, or result if life safety concerns. If differential settlement were to occur, the buildings could be relatively easily re-leveled.

In our opinion, it is acceptable and appropriate to support the relocated portable buildings on typical pressure treated wood foundations bearing on the ground surface, provided that the building pads are prepared as recommended below.

Recommendations for Portable Building Pad Grading

Loading imposed by raising the site grades in the proposed portable building relocation area and the loading imposed by the portable buildings themselves will induce settlement in the soft clay layer at a depth of roughly five to ten feet beneath the ground surface. In order to reduce the potential for differential settlement between the portable buildings and adjacent concrete walkways (which could result in ADA compliance issues) due to consolidation of the soft clay (Bay Mud), we recommend that a “no new load” design should be utilized. Light weight (lava rock) fill (approximately 65 pounds per cubic foot) can be used beneath the portable building pad areas to compensate for the new loads imposed by raised site grades and the portable buildings.

We recommend that the upper two feet of existing soil beneath the proposed portable building pads, and extending five feet beyond the building lines in all directions, should be overexcavated and replaced as compacted, engineered fill prior to placing new fill. After the two feet of existing soil is removed, the bottom of the excavation should be scarified to a depth of 8 inches, moisture conditioned to permit proper compaction, and then be compacted to a minimum relative compaction of 90 percent. The portable building pads can then be constructed using a combination of on-site soil compacted to a minimum 90 percent relative compaction and compacted light weight fill, as necessary, to achieve a “no new load” condition. Site preparation, grading, compaction of on-site soil, and placement/compaction of light weight fill should be done in accordance with the recommendations presented in the September 10, 2015 geotechnical report.

September 23, 2015

We are pleased to have been of service to you. Please do not hesitate to contact us if you have any questions or if we can be of further assistance.

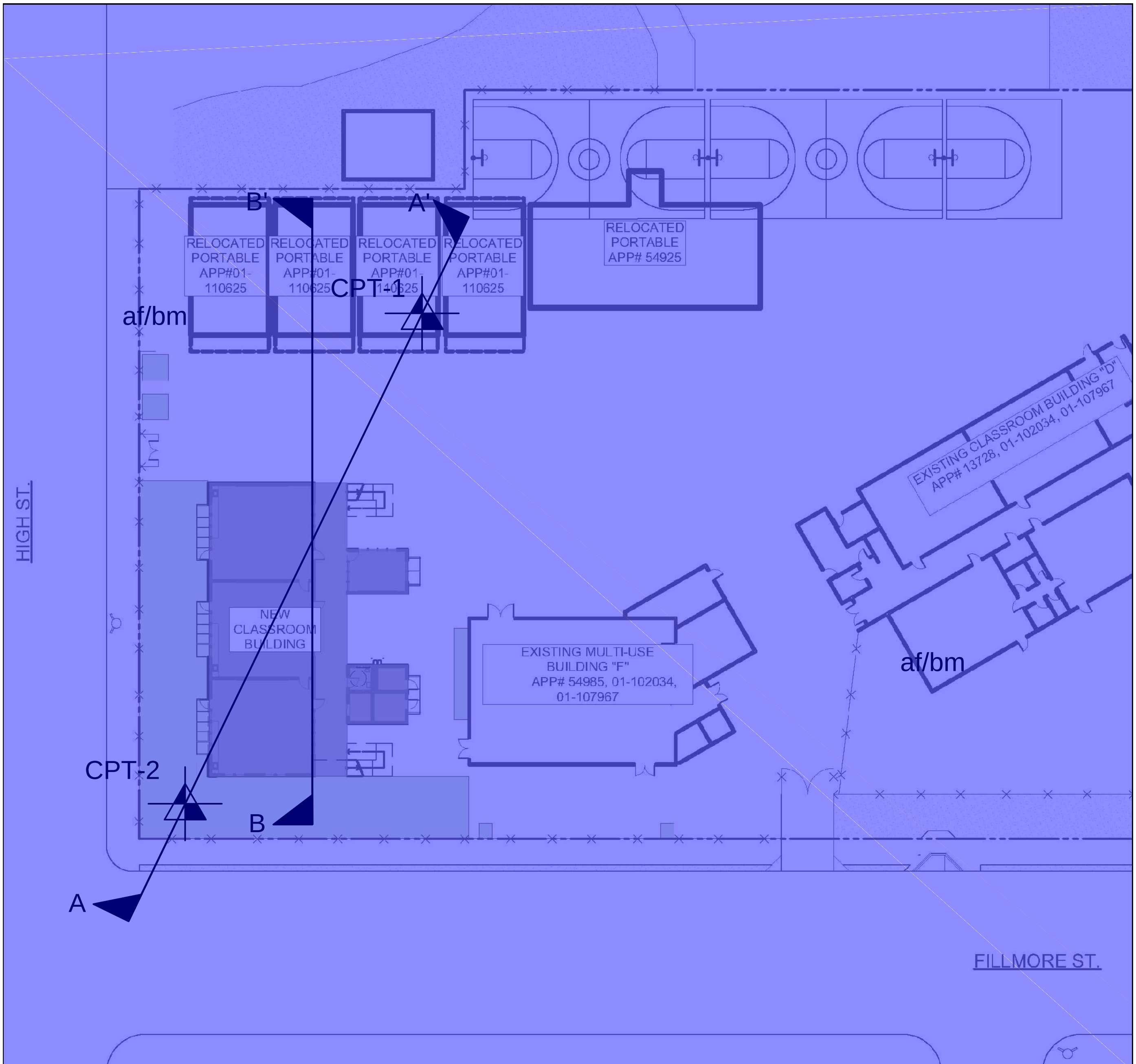
Very truly yours,
MILLER PACIFIC ENGINEERING GROUP

REVIEWED BY

Daniel S. Caldwell
Geotechnical Engineer No 2006
(Expires 9/30/17)

Michael Jewett
Engineering Geologist No. 2610
(Expires 1/31/17)

Attachments: Figures 1 and 2



SITE PLAN

MPEG CPT Location, July 2015

af/bm Artificial Fill over Bay Mud

Cross-Section

NORTH

SCALE

0 25 50 100 FEET

REFERENCE: Site Plan Provided By QKA

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FILE: 1911.024 SP.dwg

SITE PLAN

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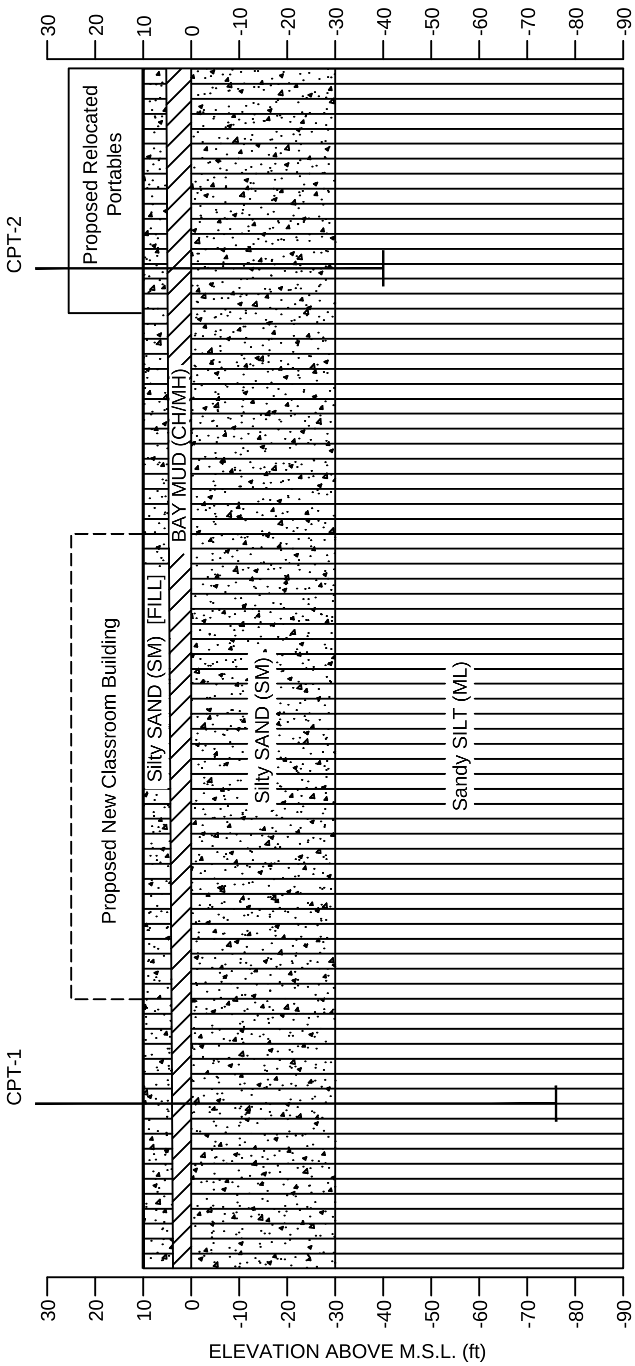
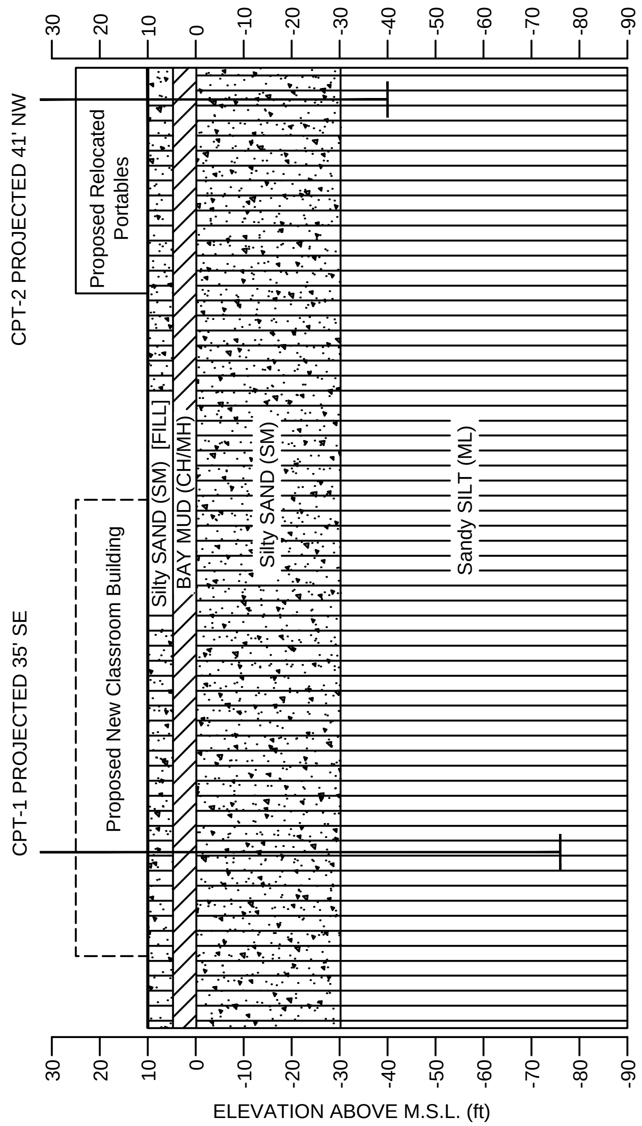
1
FIGURE



SECTION A-A'



SECTION B-B'

SCALE

0 30 60 120 FEET

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SIMPLIFIED GEOLOGIC CROSS-SECTIONS

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Project No. 1911.024 Date: 9/9/15

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2
FIGURE