MILLER PACIFIC ENGINEERING GROUP

GEOTECHNICAL INVESTIGATION
NOVATO UNIFIED SCHOOL DISTRICT
SAN MARIN HIGH SCHOOL PERFORMING ARTS CENTER
NOVATO, CALIFORNIA

December 14, 2018

Project 769.183

Prepared for:
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CERTIFICATION

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1.0 INTRODUCTION

This report summarizes Miller Pacific Engineering Group's (MPEG) Geotechnical Investigation for the planned Performing Arts Center at San Marin High School, located at 15 San Marin Drive in northern Novato, California. A Site Location Map is shown on Figure 1. The purpose of our Geotechnical Investigation is to explore subsurface conditions, evaluate geotechnical hazards that may affect the planned development, and provide geotechnical design criteria for the project. In accordance with our proposal dated October 16, 2018, we are providing our geotechnical engineering services in three phases: 1) Geotechnical Investigation for the proposed improvements, 2) supplemental consultation and geotechnical design review, and 3) construction observation and testing. This report completes our Phase 1 services and includes the following:

- Review of readily available published geologic and geotechnical reference data;
- Exploration of subsurface conditions with two exploratory soil borings;
- Laboratory testing of select samples to determine the pertinent engineering properties of the soil layers;
- Evaluation of geologic hazards and development of conceptual mitigation measures;
- Development of geotechnical recommendations and design criteria (i.e., site grading, seismic, foundation, etc.) for the project; and,
- Preparation of this report summarizing our findings.

2.0 PROJECT DESCRIPTION

Preliminary drawings prepared by Quattrocchi Kwok Architects indicate the existing Student Center (Building "H"), located south of the existing tennis courts, will be demolished to accommodate a new, approximately 9,200 square-foot performing arts/theater structure. We anticipate the Performing Arts Center will contain large open spaces requiring long structural spans that may result in significant foundation loads. Minor to moderate site grading is anticipated to construct the building pad, and new retaining walls may be needed to support lower-level or below-grade areas, such as orchestra pits or loading docks.

3.0 SITE CONDITIONS

3.1 Regional Geology

Marin County lies within the Coast Ranges geomorphic province of California, a region characterized by active seismicity, steep, young topography, and abundant landsliding and erosion owing partly to its relatively high annual rainfall. The regional basement rock consists of sedimentary, igneous, and metamorphic rock of the Jurassic-Cretaceous age (65-190 million years ago) Franciscan Complex and marine sedimentary strata of the Great Valley Sequence, which is of similar age. Within central and northern California, the Franciscan and Great Valley rocks are locally overlain by a variety of late Cretaceous and Tertiary-age sedimentary and volcanic rocks which have been deformed by episodes of folding and faulting. The youngest geologic units in the region are Quaternary-age (last 1.8 million years) sedimentary deposits. These unconsolidated deposits partially fill many of the valleys of the region.

Regional geologic mapping (CGS, 2002), indicates the site is underlain predominantly by older (Pleistocene) alluvial deposits, which typically consists of poorly- to moderately-sorted, moderately-consolidated clay, silt, sand, and gravel. Younger alluvial deposits of similar composition underlie the lower-lying, southeastern corner of the campus, while upland areas north

and northwest of the site are underlain by Franciscan mélange bedrock. A regional geologic map is shown for reference on Figure 3.

3.2 Surface Conditions

The project site is located in the central part of the existing San Marin High School campus. The campus generally consists of gentle, south-facing slopes near the southwestern foot of Mount Burdell. The proposed building site is currently developed with an existing structure to be demolished, and grades are thus relatively flat, with elevations of about +107-feet above mean sea level. Surrounding areas are generally developed with existing school buildings and interior quad/flatwork areas.

3.3 Field Exploration and Laboratory Testing

We explored subsurface conditions in the general vicinity of the planned improvements with two exploratory soil borings drilled at the approximate locations shown on Figure 2. Borings were drilled on November 10, 2018, to maximum explored depths of 26.0- and 51.5-feet below the ground surface, using a truck-mounted Mobile B-53 drill rig equipped with 6-inch hollow-stem, continuous flight augers. Materials encountered in the borings were examined and logged by our Geologist, who collected samples at select intervals for laboratory testing. Brief descriptions of the terms and methodology used in classifying earth materials are provided on the attached soil and rock classification charts, Figures A-1 and A-2, respectively. Our exploratory boring logs are shown on Figures A-3 through A-7.

Laboratory testing of select soil samples included determination of moisture content, dry density, unconfined compressive strength, percentage of particles passing the #200 (75-µm) sieve, gradation, and plasticity index in general accordance with applicable ASTM standards. Moisture, density, strength, minus-200, and plasticity index results are presented on the boring logs, Figures A-3 through A-7. Gradation analysis results are shown on Figure A-8 and a plasticity chart is shown on Figure A-9. The field exploration and laboratory testing program are discussed in further detail in Appendix A.

In addition, we previously explored subsurface conditions in the project area for design of the existing Cafeteria/Multi-Purpose Building (MPEG, 2011) and for the planned STEM building (MPEG, 2018), each located just south of the proposed Performing Arts Center. The locations of relevant borings from these projects are shown for reference on Figure 2. Copies of the boring logs from previous work by MPEG are included in Appendix B.

3.4 Subsurface Conditions

Our subsurface exploration generally confirms the regionally-mapped geologic conditions at the site. The project site is underlain by interbedded alluvial deposits variously composed of medium stiff to very stiff, medium-plasticity sandy clay and medium dense to dense clayey sand with variable gravel content. Franciscan bedrock was encountered at a depth of about 38-feet in Boring 1, and about 17-feet in Boring 2. Simplified geologic cross sections are provided on Figure 4.

Groundwater was encountered in Boring 1 and was measured at a depth of about 21.5-feet below the ground surface immediately following removal of the hollow-stem augers. Groundwater was not observed in Boring 2. Because the borings were not left open for an extended period, a stabilized depth to groundwater may not have been observed. Typically, groundwater levels fluctuate seasonally with higher levels expected during the wet winter months.

We researched the both the California Department of Water Resources Water Library (http://www.water.ca.gov/waterdatalibrary) and the California Waterboard Geotracker (https://geotracker.waterboards.ca.gov) websites to determine if existing groundwater elevation data was available in the immediate vicinity of the existing project site. The results of the search indicate that existing well data was not available within a 2,500-feet of the project site. Therefore, for analysis purposes, we assumed a historic high groundwater surface of 10.0-feet below the ground surface.

3.5 Seismicity

The project site is located within a seismically active region that includes the Central and Northern Coast Mountain Ranges. Several active faults are present east and west of the site including the San Andreas, San Gregorio, and Hayward Faults. An “active” fault is defined as 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 Department of Conservation, Division of Mines and Geology has mapped various active and inactive faults in the region (CDMG, 1972 and 2000). Faults are shown in relation to the project site on the attached Active Fault Map, Figure 5. The Rodgers Fault is the nearest known active fault and is located approximately 14.4 kilometers northeast of the site (Caltrans ARS, 2018).

3.5.1 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. Earthquakes (magnitude 2.0 and greater) that have occurred in the San Francisco Bay Area since 1985 have been plotted on a map shown on Figure 6.

3.5.2 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” (USGS, 2003, 2008; Field et al, 2015) 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. 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 study specifically analyzed fault sources and earthquake probabilities for the seven major regional fault systems in the Bay Area region, and the entire state of California and updated some of the analytical methods and models. The most recent 2015 study (UCERF3) further expanded the database of faults considered and allowed for consideration of multi-fault ruptures, among other improvements.

Conclusions from the most recent UCERF3 and USGS (Aagard et. al., 2016) studies indicate the highest probability of a M>6.7 earthquake on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Rodgers Creek Fault, located approximately 14.4-kilometers northeast at 33%. 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.

4.0 GEOLOGIC HAZARDS EVALUATION

The principal geologic hazard which could potentially affect the project site is strong seismic shaking from future earthquakes in the San Francisco Bay Region. Other hazards, such as fault surface rupture, liquefaction, tsunami inundation, slope instability, and others, are not considered significant at the site. More detailed discussion of each geologic hazard considered, their anticipated impacts, and recommended mitigation measures are discussed below.

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Geological Survey (CDMG)/California Geologic Survey (CGS) (1972, 2000) 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 neither located within an Alquist-Priolo Earthquake Fault Zone (CGS, 2000). The potential for fault surface rupture on the campus is therefore considered to be low.

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

4.2 Seismic Shaking

The site will likely experience 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.

4.2.1 Probability of Future Earthquakes

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 (Campbell and Borzognia & Chiou and Youngs, (2008)) for the very dense/soft rock 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 median and +1 standard deviation (+1σ), peak ground accelerations (PGA) which an earthquake on the fault could generate at the site are shown in Table A.

TABLE A
DETERMINISTIC PEAK GROUND ACCELERATION
Novato Unified School District
San Marin High School Performing Arts Center
Novato, California

Fault	Fault Distance[1]	Moment Magnitude[1]	Fault Mechanism	Median PGA[2,3,4,]	+1σ PGA[2,3,4]
San Andreas	17.8 km	8.0	Strike Slip	0.26 g	0.42 g
Rodgers Creek	14.4 km	7.3	Strike Slip	0.25 g	0.40 g
Hayward	21.4 km	7.3	Strike Slip	0.19 g	0.33 g
San Gregorio	25.1 km	7.4	Strike Slip	0.18 g	0.31 g
Maacama	45.9 km	7.4	Strike Slip	0.11 g	0.21 g

Reference:
1. Caltrans ARS (2018)
2. Campbell and Borzognia (2008)
3. Chiou and Youngs (2008)
4. Values determined using Vs_{30} = 270 m/s for Site Class "D". See Section 5.2 of this report for additional discussion regarding site classification.

4.2.2 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 two separate probabilistic conditions; the 2% chance of exceedance in 50 years (2,475-year statistical return period) and the 10% chance of exceedance in 50 years (475-year statistical return period), utilizing the USGS Uniform Hazard Tool (USGS, 2018). The results of the probabilistic analyses are presented below in Table B.

TABLE B
PROBABILISTIC SEISMIC HAZARD ANALYSES
Novato Unified School District
San Marin High School Performing Arts Center
Novato, California

	Statistical Return Period	Magnitude	PGA
2% in 50 years	2,475 years	7.1	0.72 g
10% in 50 years	475 years	7.1	0.46 g

Reference: USGS Unified Hazard Tool (USGS, 2018)

The potential for strong seismic shaking at the project site is high. Due to their close proximity, the San Andreas and Rodgers Creek Faults (approximately 17.8 kilometers and 14.4 kilometers southwest and northeast of the site, respectively) 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 (2016) of the California Building Code. Recommended seismic coefficients are provided in Section 5.2 of this report.*

4.3 Liquefaction Potential and Related Impacts

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 soils with a fines content (soil particles that pass the #200 sieve) greater than 30%, provided the fines exhibit a plasticity less than 7. Although the project site is not mapped within an area susceptible to liquefaction (ABAG, 2018); saturated granular layers were observed during our subsurface exploration.

4.3.1 Liquefaction Evaluation

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation, known as the Cyclic Resistance Ratio (CRR). The earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the maximum considered earthquake peak ground acceleration (PGA) and depth. Soil resistance to liquefaction is based on its relative density, and the amount and plasticity of the fines (silts and clays). The relative density of cohesionless soil is correlated with the Cone Penetration Test (CPT) tip resistance and Standard Penetration Test (SPT) blow count data measured in the field and corrected for hammer efficiency, overburden and percent fines.

We analyzed the potential for liquefaction utilizing the data from our borings and the procedures outlined by Idriss and Boulanger (2008 & 2010), considering a magnitude 8.0 earthquake producing a PGA of 0.50 g which corresponds to the PGA_M value (ASCE 7-13 Section 18.1). The results of our analyses, shown on Figures 7 and 8, indicate that granular layers observed in Boring 1 at depths of about 15 and 30-35 feet may liquefy during the design seismic event.

However, laboratory testing indicates soils at these depths contain greater than 30% fines, and that the fines fraction has a plasticity index of 38. Therefore, we judge the granular soils encountered at depths of about 15 and 30-35 feet are unlikely to liquefy during a seismic event.

Based on the results of our laboratory testing and liquefaction analysis as discussed above, we judge there is a low risk of damage to improvements due to liquefaction.

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

4.4 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. We did not observe loose, clean, granular soil layers above the historic high groundwater table. Therefore, we judge seismically induced ground settlement is not considered a significant geologic hazard at the project site.

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

4.5 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. These conditions do not exist at the site, therefore the risk of lurching and ground cracking at the project site is low.

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

4.6 Erosion

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated water runoff. Granular soils are located at the ground surface; however, the site is relatively flat. Therefore, widespread erosion is not considered a significant geologic hazard. 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. The project Civil Engineer of Architect is responsible for designing the site drainage system and, an erosion control plan could be developed prior to construction per the*

current guidelines of the California Stormwater Quality Association's Best Management Practice Handbook (2015).

4.7 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 or tsunami would be dependent upon ground motions and fault offset from nearby active faults. The project site is at an increased elevation and not located near a large body of water. Therefore, seiche and tsunami events are not considered significant geologic hazards at the site.

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

4.8 Flooding

The project site is not mapped within a FEMA 100-year or 500-year flood zone (ABAG, 2018); therefore, large scale flooding does not present a significant risk to the project. However, the project Civil Engineer or Architect is responsible for site drainage and should evaluate localized flooding potential and provide appropriate mitigation.

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

4.9 Dam Failure Inundation

The project site is located approximately 1.5-miles southeast from Stafford Lake Dam. Stafford Lake Dam is under the Division of Dam Safety (DSOD) jurisdiction which routinely monitors and evaluates the dam conditions. Additionally, an inundation map of Novato Creek from a hypothetical failure of Stafford Dam (City of Novato) shows the limit of inundation at the site as being along Novato Boulevard to the south. Therefore, based on the aforementioned references the threat of dam failure inundation is low.

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

4.10 Expansive Soil

Expansive soils will shrink and swell with fluctuations in moisture content and 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 due to their low bearing pressures. Highly plastic and/or expansive soils were not observed within the upper 5-feet during our subsurface exploration. Therefore, the risk of expansive soil affecting the proposed improvements is low.

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

4.11 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 granular soils. Soft, compressible soils were not encountered during our exploration for this or previous projects at the site. Therefore, the risk of long term settlement to the proposed structures at the project site is low.

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

4.12 Slope Instability/Landsliding

Slope instability generally occurs on relatively steep slopes and/or on slopes underlain by weak materials. The campus lies on level terrain, therefore, slope instability/landsliding is not considered a significant geologic hazard at the project site.

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

4.13 Soil Corrosion

Corrosive soil can damage buried metallic structures, cause concrete spalling, and deteriorate rebar reinforcement. We previously performed on representative samples of near-surface site soils from the adjacent planned STEM building site to evaluate pH, electrical resistivity, chloride and sulfate contents. These laboratory test results are presented on Figure A-10.

The results of our corrosivity testing indicate the upper soil layers have a pH of 6.7, a chloride concentration of 66 parts per million (ppm), and a sulfate concentration of 183 ppm. Per Caltrans Corrosion Guidelines (2015) 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 and our previous experience at the campus, corrosive soil is not anticipated to present a significant risk to the project.

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

4.14 Radon-222 Gas

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 Marin County, California, which is mapped in radon gas Zone 3 by the United States Environmental Protection Agency (USEPA, 2018). Zone 3 is classified by the EPA as exhibiting a “low” potential for Radon-222 gas with average predicted indoor screening levels less than 2 pCi/L. Therefore, the potential for hazardous levels of radon at the project site is low.

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

4.15 Volcanic Eruption

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 60-miles northeast 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, 2018). 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.*

4.16 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. Therefore, the likelihood that significant deposits of naturally-occurring asbestos will be encountered at the site is low.

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

4.17 Hazardous Materials

Hazardous materials were not observed during our subsurface exploration. While environmental testing for hazardous materials was beyond the scope of our services, we did observe enclosures that contain HVAC units and other industrial equipment that has the potential for creating hazardous materials. Therefore, we judge the potential for hazardous materials being present on the project site, currently or in the future, is low to moderate.

Evaluation: *Less than significant with mitigation.*
Mitigation: *The campus should comply with all local, state, and federal guidelines to minimize the exposure to hazardous materials. If a possible hazardous material spill occurs on campus, a qualified environmental specialist should be consulted.*

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Based on our experience with similar projects in the Novato area, we conclude that, from a geotechnical standpoint, the site is feasible for the planned improvements. The primary geotechnical issues to address in design of the project are designing structures to withstand strong seismic shaking and providing uniform foundation support in anticipation of minor liquefaction-related settlements.

5.2 Seismic Design

Based on the results of our exploration and laboratory testing, we recommend classifying the site as "Site Class D". The goal of the building code is protection of life safety, some structural damage may still occur during strong ground shaking. If ground improvement is not included in the construction scope, we should be consulted to provide a non-linear site-specific response analysis.

TABLE C
2016 CBC FACTORS
Novato Unified School District
San Marin High School Performing Arts Center
Novato, California

Factor Name	Coefficient	2016 CBC Site Specific Value
Site Class[1]	$S_{A,B,C,D,E,\text{ or }F}$	S_D
Site Coefficient	F_a	1.0
Site Coefficient	F_v	1.5
Spectral Acc. (short)	S_S	1.50 g
Spectral Acc. (1-sec)	S_1	0.60 g
Spectral Response (short)	SM_S	1.50 g
Spectral Response (1-sec)	SM_1	0.90 g
Design Spectral Response (short)	SD_S	1.00 g
Design Spectral Response (1-sec)	SD_1	0.60 g
MCE_G[2] PGA adjusted for Site Class	PGA_M	0.50 g
Seismic Design Category	A,B,C,D, or E	D

Notes:
1. Site Class D Description: "Stiff Soil" profile with shear wave velocities between 600 and 1,200 ft/sec, Standard Penetration Test (SPT) blow counts between 15 and 50 blows per foot, and undrained shear strength between 1,000 and 2,000 psf.
2. Maximum Considered Earthquake Geometric Mean.

5.3 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 soils may be saturated due to rainfall and may be difficult to compact without drying by aeration or the addition of lime and/or cement (or a similar product) to dry the soils. Site preparation and grading should

conform to the recommendations and criteria outlined below. General recommendations for wintertime construction are provided later in this report.

5.3.1 Surface Preparation

Clear all trees, brush, roots, over-sized debris, loose stockpiled soils, and organic material from areas to be graded. Trees that will be removed (in structural areas) must also include removal of stumps and roots larger than two inches in diameter. Excavated areas (i.e., excavations for stump removal) should be restored with properly moisture conditioned and compacted fill as described in the following sections. Any loose soil or rock at subgrade will need to be excavated to expose firm natural soils or bedrock. Debris, rocks larger than six inches and vegetation are not suitable for structural fill and should be removed from the site. Alternatively, vegetation strippings may be used in landscape areas.

Where fills or other structural improvements are planned on level ground, the subgrade surface should be scarified to a depth of about eight inches, moisture conditioned to above the optimum moisture content, and compacted to a minimum of 90% relative compaction (ASTM D-1557). Relative compaction should be increased to a minimum of 95% where new asphalt pavements are planned. 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." If soft, wet or otherwise unsuitable materials are encountered at the subgrade elevation during construction, we will provide supplemental recommendations/field directives to address the specific condition.

5.3.2 Excavations

Excavations for new utilities or other below-grade facilities at the site will generally encounter medium-dense to dense clayey and gravelly sand. Based on our exploration, we anticipate most site excavations can be reasonably accomplished with “traditional” excavation equipment, such as medium-size dozers, excavators, and backhoes.

5.3.3 Compacted Fill

On-site fill, backfill, and scarified subgrades should be conditioned to within 2% 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 a minimum of 90% relative compaction.

5.3.4 Materials

Based on our laboratory testing, onsite soils are suitable as use for fill provided they are prepared as described above, some seasonal movement should be expected. If imported fill 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 20, and (3) have a maximum particle size of 6 inches. Any imported fill material needs to be tested to determine its suitability for use as fill material.

5.3.5 Permanent and Temporary Cut and Fill Slopes

Based on our exploration and laboratory testing, we judge the medium-dense to dense alluvial soils at the site should generally be considered “Type B” soils per the Cal/OSHA classification scheme. As such, all temporary cut slopes (such as may be needed while

retaining walls are constructed and backfilled) should not exceed inclinations of 1:1 (H:V). If steeper temporary cut slopes are needed, they will need to be shored or braced in accordance with Cal/OSHA regulations. Permanent cut slopes in native or engineered fill materials should not exceed 2:1.

We do not anticipate new fill slopes will be required for the project. If needed, new un-reinforced fill slopes should not exceed inclinations of 2:1. Steeper slopes are possible but will need to be specifically designed. Any fill slopes constructed on existing ground which slopes steeper than 5:1 will need to be keyed, benched, and fully drained. If permanent fill slopes are planned, we should be contacted to provide supplemental recommendations.

5.4 Foundation Design

Provided soils are moisture conditioned and compacted as described above, a rigid interconnected shallow foundation system may be utilized to support the proposed structure. The rigid foundation system may consist of interconnected shallow foundations or a “waffle” concrete slab-on-grade. Foundation design criteria are shown in Table D.

TABLE D
SHALLOW FOUNDATION DESIGN CRITERIA
Novato Unified School District
San Marin High School Performing Arts Center
Novato, California

Concrete Mat Slab Foundation

Minimum thickness:	8 inches
Modulus of subgrade reaction, k_s:	100 pci
Maximum unsupported interior span[1]:	30 feet
Maximum unsupported edge/corner span[1]:	10 feet
Base friction:	0.30

Shallow Foundations

Minimum embedment below existing grade:	18-inches
Minimum width[2]:	
One-story:	12 inches
Two-story:	15-inches
Three-story:	18-inches
Allowable bearing pressure:	
Dead plus live loads:	2,500 psf
Total including wind and seismic	3,000 psf
Base friction coefficient:	0.30
Lateral passive resistance[3,4]:	350 pcf
Maximum unsupported interior span[1]:	30 feet
Maximum unsupported edge/corner span[1]:	10 feet

Notes:
1.) Assumes rigid slab behavior with idealized fixed conditions.
2.) Design shallow foundations to similar bearing pressures, i.e., size footing widths to maintain uniform bearing loads. Maintain above optimum moisture contents until concrete slabs are completed.
3.) May increase design values by 1/3 for total design loads including wind and seismic.
4.) Neglect upper 6-inches unless confined by concrete. Equivalent Fluid Pressure, not to exceed 3,500 psf.

If building loads are significant, the structure may be supported on a deep foundation system. Suitable deep foundation options could include drilled concrete piers, helical piles, driven piles, auger-cast piles, and others. Of these, we anticipate driven systems are undesirable given the proximity of other existing structures. We expect that helical piles or traditional drilled piers would be most effective and economical if deep foundations are necessary. If a deep foundation system is preferred, we can provide specific design criteria for the chosen system upon request.

5.5 Retaining Walls

New retaining walls could be required to support below-grade facilities such as loading docks or orchestra pits. It is anticipated that these walls would be constructed of reinforced concrete. While not indicated on current plans, it is also possible landscape or other site walls may also be

constructed. Many wall types are available; in our experience, soldier-pile and timber lagging, reinforced concrete, and soil nail/shotcrete walls are often cost effective for retaining cuts, while mechanically-stabilized earth MSE) walls (such as Keystone, Versa-Lok, and others) are often more cost-effective for retaining new fills.

Retaining walls that can deflect at the top, such as landscape walls, can be designed using the unrestrained criteria shown in Table D. Walls that are structurally connected and not allowed to deflect, such as basement walls, are considered restrained. Restrained conditions are commonly designed using a uniform earth pressure distribution rather than an equivalent fluid pressure. Lateral support can be obtained from either passive soil resistance (i.e., keyways) or frictional sliding resistance of footings or from tiebacks. In addition to the soil loads, retaining walls higher than 5-feet should be designed to resist temporary seismic loads.

TABLE E
RETAINING WALL DESIGN CRITERIA
Novato Unified School District
San Marin High School Performing Arts Center
Novato, California

Foundation

Refer to the foundation design criteria in Table D.

Lateral Earth Pressure	Unrestrained[1,2]	Restrained[1,3]
Level Ground	40 pcf	25 X H psf
2:1 Slope	60 pcf	40 X H psf

Seismic Surcharge[3] 15 X H psf

Soil Nails/Tiebacks and Reinforced Earth MSE Concrete Block Wall

	Phi[4]	C (psf)[5]	Gamma (pcf)[6]
Retained Soils	30	500	125
Foundation Soils	35	750	135

Min. Diameter Grouted Holes: 6 inches

Skin Friction: 2,000 psf

Notes:

(1) Interpolate earth pressures for intermediate slopes.
(2) Equivalent fluid pressure.
(3) Rectangular uniform pressure distribution (H = height of wall). Factor of safety may be reduced to 1.0 or greater for seismic conditions.
(4) Angle of Internal Friction, effective stress, unitless
(5) Apparent (effective) Cohesion, for seismic conditions 500 psf of additional cohesion may be included for both materials
(6) Unit Weight of Soil
(7) Tiebacks should be designed for load-testing up to 200% of the design load. Load testing to be performed in general accordance with the procedures recommended by the Post-Tensioning Institute (1996).

All walls higher than 3-feet require drainage to prevent the build-up of hydrostatic pressure. Either Caltrans Class 1B permeable material within filter fabric, drainage panels, or Caltrans Class 2 permeable material can be used. The project Architect should design a water-proofing system for walls adjacent to living space. The drainage should be collected in 4-inch, perforated, Schedule 40 PVC drain line placed at the base of the wall or discharged through weep-holes in the case of soil nail or cast-in-place concrete walls. Seepage collected in the drains should be conveyed in a closed pipe system to a suitable discharge outlet well away from the structures. To maintain the wall drainage system, clean-outs must be provided for perforated pipes at the upstream end. Sweep fittings should be used at all major changes in direction. A typical retaining wall drain detail is shown on Figure 9. Retaining wall backfill should be compacted in accordance with the recommendations presented in site grading.

5.6 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. 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%) 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%). 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 tight pipe collection system. Site drainage improvements should be connected into the existing campus storm drainage system.

5.7 Concrete Slabs-On-Grade

Where interior concrete slabs are needed, we recommend they be at least 5-inches thick and reinforced with steel bars (not wire mesh). Contraction joints should be incorporated in the concrete slab in both directions, no greater than 10 feet on center. Additionally, the reinforcing bars shall extend through the control joints. For improved performance, concrete slabs on grade may be increased to 6-inches thick. To improve interior moisture conditions, a 4-inch minimum layer of clean, free draining, 3/4-inch angular gravel should be placed on a properly moisture conditioned and compacted subgrade to form a capillary moisture break. A plastic membrane vapor barrier, 15-mils or thicker, should be placed over the drain rock. The vapor barrier shall meet the Class A requirements outlined in ASTM E 1745 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 slabs resulting in poor performance of floor coverings, mold growth or other adverse conditions.

Exterior concrete slabs should be at least 4-inches thick and reinforced as described above for interior slabs. For improved performance, exterior concrete slabs shall be underlain with at least 4-inches or more of Caltrans Class 2 Aggregate Base compacted to at least 92 percent relative compaction. Some movement should be expected for exterior concrete slabs as the underlying soils react to seasonal moisture changes.

5.8 Asphalt Pavement

We understand asphalt pavement areas for vehicular parking and pedestrian use may be constructed. Typically, asphalt pavement sections are designed utilizing two variables, the R-Value (a measure of the subgrade resistance) and the Traffic Index (a measure of the amount of daily

traffic). Based on the subsurface conditions we judge an R-Value of 15 is appropriate for the site. We have calculated pavement sections for the project site in accordance with Caltrans procedures for flexible pavement design utilizing the values described above and various Traffic Index (T.I.) values. The resulting supplemental pavement sections are presented in Table F below.

TABLE F
ASPHALT PAVEMENT SECTION
Novato Unified School District
San Marin High School Performing Arts Center
Novato, California

T.I.	Asphalt Concrete	Aggregate Base Rock
4.0	2.5 inches	6.5 inches
5.0	3.0 inches	8.0 inches
6.0	3.5 inches	10.5 inches

The aggregate baserock should conform to Caltrans Class 2 Aggregate Baserock (Class 2 AB) outlined in Section 26 of the Caltrans Standard Specifications. The Class 2 AB shall be placed in layers on a properly prepared and firm and unyielding subgrade as described in the previously discussed grading recommendations. The Class 2 AB should be compacted to at least 95% relative compaction. Additionally, the Class 2 AB section should be firm and unyielding under heavy construction equipment.

5.9 Utility Trench Excavations and Backfills

Excavations for utilities will most likely extend into loose to medium dense granular soils. Trench excavations having a depth of five feet or more that 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 B. 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% of particles passing the No. 4 sieve and no more than 5% 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 to within 2% of the optimum moisture content, placed in thin lifts and compacted to a minimum of 90% relative compaction. 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.

5.10 Wintertime Construction

Wintertime/wet weather site work is feasible during the construction phase of this project provided weather conditions do not adversely impact the planned grading, and proper erosion control measures are implemented to prevent excessive silt and mud from entering the storm drain system.

High soil moisture contents and muddy site conditions may impact placing fills, compacting subgrades, and excavating foundation trenches. Several alternatives may be considered to improve the site conditions to allow site work to proceed in rainy conditions:

- Prior to the onset of winter rains, maintain a drier site by covering the work area and any stockpiled materials with plastic visqueen sheeting or other impermeable membrane. Where asphalt pavements, other hardscape or drainage improvements currently exist in work areas, consider leaving these improvements in place until the last possible moment to maintain a drier subgrade condition.

- Lime/cement treat the subgrade soils when site work commences to “weatherproof” the site. The disadvantage to this alternative is that future landscaping will likely require excavation and replacement of the treated soils for acceptable plant growth.

- Finally, imported, drier fill materials could be used to stabilize the site. Soft or wet on-site materials could be excavated to firm materials and drier (preferably granular) soils with good drainage characteristics would be imported to restore site grades. This alternative might also require future excavation and replacement of landscaping soils.

If construction occurs relatively early in the winter, we judge the first option (covering the site prior to winter rains) could be an effective method of maintaining a workable site. When the construction schedule and weather conditions are known, we can meet with the project team to further discuss alternatives to continuation of wintertime construction.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

We must review the plans and specifications for the project when they are nearing completion to confirm that the intent of our geotechnical recommendations has been incorporated and provide supplemental recommendations, if needed. During construction, we must observe and test site grading, foundation excavations for the structures and associated improvements to confirm that the soils encountered during construction are consistent with the design criteria.

7.0 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 Novato 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 soils 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 boring 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.

8.0 LIST OF REFERENCES

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SITE COORDINATES
LAT. 38.1198°
LON. -122.6113°

SITE LOCATION

N.T.S.

NORTH

REFERENCE: Google Earth, 2018

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SITE LOCATION MAP

San Marin High School
Performing Arts Center
Novato, California

Project No. 769.183 Date: 12/12/2018

Drawn MMT
Checked

1
FIGURE

SITE PLAN AND GEOLOGIC MAP

1" = 100'

NORTH

LEGEND

Approximate boring location completed by MPEG, November 2018

Approximate boring location completed by MPEG, January 2018

Approximate boring location completed by MPEG, April 2011

Qoa - Alluvium (See Figure 3)

REFERENCE: Google Earth, 2018

SCALE

0 50 100 200 FEET

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SITE PLAN AND GEOLOGIC MAP

San Marin High School
Performing Arts Center
Novato, California

Project No. 769.183 Date: 12/12/2018

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Checked

2
FIGURE

REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

NORTH

LEGEND

Qha — UNDIVIDED ALLUVIUM (HOLOCENE)
Alluvium deposited on fans, terraces, or in basins. Typically unconsolidated, poolry- to moderately-sorted clay, silt, sand, and gravel.

Qoa — UNDIVIDED ALLUVIUM (EARLY TO LATE PLEISTOCENE)
Includes alluvial fan, terrace, basin, and stream channel deposits.

Qls — LANDSLIDES (HOLOCENE)
Includes debris-flow and block slump landslides. Arrows indicate direction of movement.

KJfm — FRANCISCAN MELANGE (JURASSIC-CRETACEOUS)
Tectonic mixture of masses or resistant rock types including sandstone, altered mafic volcanic rock (greenstone), chert, serpentinite, and exotic metamorphic rocks embedded in a sheared, shaley matrix.

GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE

Reference: Rice, S.R. et al (2002), "Geologic Map of the Novato 7.5-Minute Quadrangle, Marin and Sonoma Counties, California: A Digital Database, Version 1.0", California Department of Conservation, California Geological Survey, Map Scale 1:24,000.

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REGIONAL GEOLOGIC MAP

San Marin High School
Performing Arts Center
Novato, California

Project No. 769.183 Date: 12/12/2018

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3
FIGURE

CROSS SECTION A-A'

Scale 1" = 40'

CROSS SECTION B-B'

Scale 1" = 40'

LEGEND

CL	Silt and Clay: Soil with at least 50% fine-grained material, includes sandy clay, sandy clay with gravel, and sandy gravelly clay [Alluvium]
SC	Sand with Fines: Soils containing at least 50% coarse grained material, includes clayey sand, gravelly sand, and clayey gravelly sand [Alluvium]
ML	Silt and Clay: Soil with at least 50% fine-grained material, includes silts and very fine sands, silty or clayey fine sands, or clayey silts [Alluvium]
KJfm	Franciscan Melange. Tectonic mixture of masses or resistant rock types including sandstone, altered mafic volcanic rock (greenstone), chert, serpentinite, and exotic metamorphic rocks embedded in a sheared, shaley matrix.

SCALE

0 20 40 80 FEET

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GEOLOGIC CROSS SECTION

San Marin High School
Performing Arts Center
Novato, California

Project No. 769.183 Date: 12/12/2018

Designed
Drawn MMT
Checked

4
FIGURE

SITE COORDINATES
LAT. 38.1198°
LON. -122.6113°

SCALE
0 12.5 25 50 MILES

NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Known Active Faults in the San Francisco Bay Region, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

MPEG 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 San Marin High School Performing Arts Center Novato, California Project No. 769.183 Date: 12/12/2018	Drawn MMT Checked	5 FIGURE

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SITE COORDINATES
LAT. 38.1198°
LON. -122.6113°

SCALE

0 12.5 25 50 MILES

NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Earthquakes Greater Than Magnitude 2.0 in the San Francisco Bay Region from 1985-2014, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

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HISTORIC EARTHQUAKE EPICENTERS

San Marin High School
Performing Arts Center
Novato, California

Project No. 769.183 Date: 12/12/2018

Drawn MMT
Checked

6
FIGURE

Project: **SMHS Performing Arts Center** Analyzed by: **MFJ**

Job No: **769.183** Date: **12/13/2018**

Soil & Seismic Properties

Magnitude, M_w: **8.0**

PGA, g: **0.50**

Layer	UCS Type	γ_d, pcf	ω%	Depth, ft Upper	Depth, ft Lower	γ_t, pcf	Thickness, ft
1	**SC**	**120**	**15%**	0	**38**	138	38
2	**ROCK**	**130**	**20%**	38	**50**	156	12

GW Depth: **10.0** feet

Cyclic Resistance Ratio (CRR)

MC to SPT: **0.6**

Borehole Dia: **6.0**

Hammer Eff: **0.8**

Sample	Depth, ft	N_{FIELD}, bpf	Fines Cont., %	Sampler Type	Liner Correction	UCS Type	σ, psf	σ', psf	Sampler Corr.	$N_{COR.}$, bpf
B1	**10.0**	**23**	**50**	**SPT**	**NO**	SC	1380	1380	1.0	23.0
B1	**15.0**	**17**	**37**	**SPT**	**NO**	SC	2070	1758	1.0	17.0
B1	**20.0**	**26**	**35**	**SPT**	**NO**	SC	2760	2136	1.0	26.0
B1	**25.0**	**38**	**35**	**SPT**	**NO**	SC	3450	2514	1.0	38.0
B1	**30.0**	**18**	**34**	**SPT**	**NO**	SC	4140	2892	1.0	18.0
B1	**35.0**	**21**	**33**	**SPT**	**NO**	SC	4830	3270	1.0	21.0
B2	**10.0**	**29**	**50**	**MC**	**NO**	SC	1380	1380	0.6	17.4
B2	**15.0**	**35**	**35**	**MC**	**NO**	SC	2070	1758	0.6	21.0

Sample	Depth (ft)	C_E	C_B	C_R	C_S	C_N	N_{60}, bpf	$(N_1)_{60}$, bpf	$\Delta(N_1)_{60}$, bpf	$(N_1)_{60CS}$, bpf	$CRR_{M=7.5,\sigma'=1}$
B1	10.0	1.33	1.05	0.80	1.00	1.13	**30.7**	**29.1**	**5.6**	**35**	**1.05**
B1	15.0	1.33	1.05	0.85	1.00	1.05	**22.7**	**21.3**	**5.5**	**27**	**0.34**
B1	20.0	1.33	1.05	0.95	1.00	0.98	**34.7**	**33.9**	**5.5**	**39**	**3.42**
B1	25.0	1.33	1.05	0.95	1.00	0.95	**50.7**	**48.0**	**5.5**	**54**	**1.04E+04**
B1	30.0	1.33	1.05	0.95	1.00	0.87	**24.0**	**20.7**	**5.5**	**26**	**0.32**
B1	35.0	1.33	1.05	1.00	1.00	0.84	**28.0**	**24.6**	**5.5**	**30**	**0.49**
B2	10.0	1.33	1.05	0.80	1.00	1.15	**23.2**	**22.4**	**5.6**	**28**	**0.38**
B2	15.0	1.33	1.05	0.85	1.00	1.05	**28.0**	**26.2**	**5.5**	**32**	**0.61**

Cyclic Stress Ratio Induced by Earthquake (CSR)

Sample	Depth, ft	σ, psf	σ', psf	r_d	C_σ	K_σ	MSF	$CSR_{M,\sigma'}$	$CSR_{M=7.5,\sigma'=1}$	F.S.
B1	10	1380	1380	0.99	0.26	1.10	0.88	**0.32**	**0.33**	**3.00+**
B1	15	2070	1758	0.98	0.18	1.02	0.88	**0.37**	**0.42**	**0.81**
B1	20	2760	2136	0.97	0.30	0.98	0.88	**0.41**	**0.47**	**3.00+**
B1	25	3450	2514	0.95	0.30	0.93	0.88	**0.43**	**0.52**	**3.00+**
B1	30	4140	2892	0.94	0.17	0.94	0.88	**0.44**	**0.53**	**0.60**
B1	35	4830	3270	0.92	0.20	0.90	0.88	**0.44**	**0.56**	**0.87**
B2	10	1380	1380	0.99	0.19	1.07	0.88	**0.32**	**0.34**	**1.12**
B2	15	2070	1758	0.98	0.22	1.03	0.88	**0.37**	**0.42**	**1.47**

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LIQUEFACTION CALCULATIONS

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Novato, California

Project No. 769.183 Date: 12/12/2018

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7
FIGURE

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LIQUEFACTION RESULTS

San Marin High School
Performing Arts Center
Novato, California

Project No. 769.183 Date: 12/12/2018

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8
FIGURE

NOTES:

1. Wall drainage should consist of clean, free draining 3/4 inch crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material. Alternatively, pre-fabricated drainage panels (Miradrain G100N or equivalent), installed per the manufacturers recommendations, may be used in lieu of drain rock and fabric.

2. All retaining walls adjacent to interior living spaces shall be water/vapor proofed as specified by the project architect or structural engineer.

3. Perforated pipe shall be SCH 40 or SDR 35 for depths less than 20 feet. Use SCH 80 or SDR 23.5 perforated pipe for depths greater than 20 feet. Place pipe perforations down and slope at 1% to a gravity outlet. Alternatively, drainage can be outlet through 3" diameter weep holes spaced approximately 20' apart.

4. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe. Additionally, all angled connectors shall be long bend sweep connections.

5. During compaction, the contractor should use appropriate methods (such as temporary bracing and/or light compaction equipment) to avoid over-stressing the walls. Walls shall be completely backfilled prior to construction in front of or above the retaining wall.

6. Refer to the geotechnical report for lateral soil pressures.

7. All work and materials shall conform with Section 68, of the latest edition of the Caltrans Standard Specifications.

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TYPICAL RETAINING WALL BACKDRAIN

San Marin High School
Performing Arts Center
Novato, California

Project No. 769.183 Date: 11/16/2017

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9
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION (BORINGS) AND LABORATORY TESTING

1.0 Subsurface Exploration

We explored subsurface conditions at the site by drilling 2-test borings using a truck-mounted Mobile B-53 drill rig equipped with 6-inch hollow stem augers on November 10th, 2018. The approximate boring locations are shown on Figure 2. The borings were drilled to maximum explored depths of 26.0- and 51.5-feet below the ground surface.

The soils encountered were logged and identified in the field by our Geologist in general accordance with ASTM Standard D 2487, "Field Identification and Description of Soils (Visual-Manual Procedure)." This standard is briefly explained on Soil and Rock Classification Charts. Figures A-1 and A-2, respectively. The boring logs are presented on Figures A-3 through A-7.

We obtained “undisturbed” samples using a 3-inch diameter, split-barrel modified California sampler with 2.5 by 6-inch brass tube liners or with a 2-inch diameter, split-barrel Standard Penetration Test (SPT) sampler. The sampler was driven with a 140-pound hammer falling 30 inches. The number of blows required to drive the samplers 18 inches was recorded and is reported on the boring logs as blows per foot for the last 12 inches of driving. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory.

2.0 Laboratory Testing

We conducted laboratory tests on selected intact samples to verify field identifications and to evaluate engineering properties. The following laboratory tests were conducted in accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216;
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937;
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166;
- Amount of Material in Soils Finer Than the No. 200 (75 µm) Sieve, ASTM D 1140;
- Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis, ASTM D6913;
- Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D4318;
- pH in soil, EPA 9040;
- Resistivity in Soil, SM 2510; and
- Anions in soil (sulfate and chloride), EPA 300.

The moisture content, dry density, unconfined compressive strength, percent passing the #200 sieve results, and plasticity index results are shown on the exploratory Boring Logs, Figures A-3 through A-7. Gradation results are shown on Figure A-8 and a Plasticity Chart is presented on Figure A-9. Corrosion test results are shown on Figure A-10. The exploratory boring logs, description of soils encountered, and the laboratory test data reflect conditions only at the location of the boring at the time they were excavated or retrieved. 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.

MAJOR DIVISIONS		SYMBOL	DESCRIPTION
COARSE GRAINED SOILS over 50% sand and gravel	CLEAN GRAVEL	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GRAVEL with fines	GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	CLEAN SAND	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
	SAND with fines	SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS over 50% silt and clay	SILT AND CLAY liquid limit <50%	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	SILT AND CLAY liquid limit >50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, muck, and other highly organic soils
ROCK			Undifferentiated as to type or composition

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

PI	PLASTICITY INDEX
LL	LIQUID LIMIT
SA	SIEVE ANALYSIS
HYD	HYDROMETER ANALYSIS
P200	PERCENT PASSING NO. 200 SIEVE
P4	PERCENT PASSING NO. 4 SIEVE

SAMPLER TYPE

- MODIFIED CALIFORNIA
- STANDARD PENETRATION TEST
- THIN-WALLED / FIXED PISTON
- HAND SAMPLER
- ROCK CORE
- X DISTURBED OR BULK SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.

STRENGTH TESTS

TV	FIELD TORVANE (UNDRAINED SHEAR)
UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress

SAMPLER DRIVING RESISTANCE

Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:

25	sampler driven 12 inches with 25 blows after initial 6-inch drive
85/7"	sampler driven 7 inches with 85 blows after initial 6-inch drive
50/3"	sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive

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SOIL CLASSIFICATION CHART

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A-1
FIGURE

FRACTURING AND BEDDING

Fracture Classification	Spacing	Bedding Classification
Crushed	less than 3/4 inch	Laminated
Intensely fractured	3/4 to 2-1/2 inches	Very thinly bedded
Closely fractured	2-1/2 to 8 inches	Thinly bedded
Moderately fractured	8 to 24 inches	Medium bedded
Widely fractured	2 to 6 feet	Thickly bedded
Very widely fractured	greater than 6 feet	Very thickly bedded

HARDNESS

Low	Carved or gouged with a knife
Moderate	Easily scratched with a knife, friable
Hard	Difficult to scratch, knife scratch leaves dust trace
Very hard	Rock scratches metal

STRENGTH

Friable	Crumbles by rubbing with fingers
Weak	Crumbles under light hammer blows
Moderate	Indentations <1/8 inch with moderate blow with pick end of rock hammer
Strong	Withstands few heavy hammer blows, yields large fragments
Very strong	Withstands many heavy hammer blows, yields dust, small fragments

WEATHERING

Complete	Minerals decomposed to soil, but fabric and structure preserved
High	Rock decomposition, thorough discoloration, all fractures are extensively coated with clay, oxides or carbonates
Moderate	Fracture surfaces coated with weathering minerals, moderate or localized discoloration
Slight	A few stained fractures, slight discoloration, no mineral decomposition, no affect on cementation
Fresh	Rock unaffected by weathering, no change with depth, rings under hammer impact

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in other locations and with the passage of time.

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ROCK CLASSIFICATION CHART

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A-2
FIGURE

BORING 1

EQUIPMENT: Truck-Mounted Mobile B-53 with 6.0-inch Solid Flight Auger
DATE: 11/10/18
ELEVATION: 108 - feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			Clayey SAND with Gravel (SC) Light yellow brown, dry, medium dense, fine to coarse grained sand, ~20-25% subangular to subrounded gravel, ~10-15% low plasticity clay. [Alluvium]	34	108	8.0			
1	5			Grades dense.	61	122	13.6	UC 9250		
2				Sandy CLAY with Gravel (CL) Light yellow brown, moist, very stiff, medium plasticity clay, ~30-35% fine to coarse grained sand, ~15-20% subangular to subrounded gravel. [Alluvium]	23				P200 50.7%	SA
3	10									
4	15			Clayey SAND with Gravel (SC) Light yellow brown, dry, medium dense, fine to coarse grained sand, ~35-40% medium plasticity clay, ~20-25% subangular to subrounded gravel. [Alluvium]	17				P200 37.0%	
5										
6	20									

Water level encountered during drilling
Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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A-3
FIGURE

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	BORING 1 (CONTINUED)	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	20			Clayey SAND with Gravel (SC) Light yellow brown, dry, medium dense, fine to coarse grained sand, ~35-40% medium plasticity clay, ~20-25% subangular to subrounded gravel. [Alluvium]	26		12.0		LL:48 PI:28	
7										
	25			Grades dense.	38				P200 35.2%	
8										
9	30			Grades wet, free water present, medium dense, shale and sandstone rock fragments present in front of sampler.	18		18.7			
10										
	35				21				P200 33.1%	
11										
12	40			Clayey SAND (SC) Light orange, red, and brown, wet, dense, fine to coarse grained sand, ~35-45% medium to high plasticity clay, trace subangular gravel to 1"Ø. [Residual Soil]						

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
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A-4
FIGURE

DEPTH meters / feet	SAMPLE	SYMBOL (4)	BORING 1 (CONTINUED)	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
40			Clayey SAND (SC) Light orange, red, and brown, wet, dense, fine to coarse grained sand, ~35-45% medium to high plasticity clay, trace subangular gravel to 1"Ø. [Residual Soil]	36		26.9			
13									
45				80/11"		23.4			
14									
			Shale Light grey, green, low hardness, weak, pervasively sheared, thinly bedded/laminated. [Bedrock]						
15									
50				47		7.8			
			Boring terminated at 51.5 feet. Ground water encountered at 21.5 feet during exploration.						
16									
55									
17									
18									
60									

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
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A-5
FIGURE

BORING 2

EQUIPMENT: Truck-Mounted Mobile B-53 with 6.0-inch Solid Flight Auger
DATE: 11/10/18
ELEVATION: 107 - feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			1.5 inches Asphalt Concrete						
				16 inches Aggregate Baserock						
				Clayey SAND with Gravel (SC) Medium brown, moist, medium dense, fine to coarse grained sand, ~30-40% gravel to ¾"Ø, ~20-30% medium plasticity clay. [Fill]	27		10.3			
1	5				15					
2					18				P200 50.0%	
				Sandy CLAY (CL) Medium brown, moist, very stiff, medium plasticity clay, ~40% fine to coarse grained sand, ~5-15% subangular gravel to ½"Ø. [Alluvium]	15					
3	10				29	119	16.3	UC 3150		
4										
5	15			As above, slight variations in gravel/sand/clay percentages as typical of meandering stream facies.	35	115 121	18.4 15.1			
				Sandstone Light tan to buff, moderately hard, moderately strong, moderately to highly weathered, fine to medium grained sand. [Bedrock]						
6	20									

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
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A-6
FIGURE

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	BORING 2 (CONTINUED)	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	20			Sandstone Light tan to buff, moderately hard, moderately strong, moderately to highly weathered, fine to medium grain sand. [Bedrock]	54		16.2			
7										
	25				72/10"		13.2			
8				Boring terminated at 26.0 feet. No groundwater encountered during exploration.						
9	30									
10										
	35									
11										
12	40									

Water level encountered during drilling

Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
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BORING LOG

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A-7
FIGURE

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PARTICLE SIZE ANALYSIS - ASTM D 6913 & ASTM D 1140

SYMBOL	SAMPLE SOURCE	CLASSIFICATION
—◆—	B1 @ 10.0	Sandy CLAY with Gravel (CL)

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SIEVE ANALYSIS RESULTS

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A-8
FIGURE

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ATTERBERG LIMITS TEST (ASTM D 4318)

Sample	Classification	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Boring: B1 @ 20.0 ft	Clayey SAND with Gravel (SC)	48	20	28

PI = 0-3: Non-Plastic
PI = 3-15: Slightly Plastic
PI = 15-30: Medium Plasticity
PI = >30: High Plasticity

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PLASTICITY INDEX TEST RESULTS

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A-9
FIGURE

E T S

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ATTN: Ben Pappas	DATE of	S. Santos	D. Jacobson
JOB NAME: San Marin High School, 15 San Marin Drive,	DATE RECEIVED	COMPLETION	L. Quijano	LAB DIRECTOR
JOB #: 759.176 Novato, California	1/18/2018	1/26/2018		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
07625-1	SMHS1-SMD/N	Native Soil B1-B5 @ 0.0'-4.0'	**6.74**	**1,253**	**798**	**183**	**66**
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 %
Method	Detection	Limits --->	---	0.1	0.1	1	0.1

**********COMMENTS**********

Resistivity is >1,250 ohm-cm, i.e., in the low range; soil reaction (i.e., pH) is very mildly acidic; sulfate is low enough (@ <200 ppm), and chloride also is low enough (@ <100 ppm). The CalTrans (CT) times to perforation of galvanized steel and full depth pitting times (following Uhlig) for otherwise unprotected steel in this soil are determined based on pertinent parameters [see the table at left below below]. Sulfate should have no significant adverse impact on concrete, cement, mortar or grout; and chloride should not have a measureable adverse impact on rebar or buried steel. Based on the tested parameters, a point range for this soil can be determined (see table below on right). In principle, lime or mild cement (@ 1%-2%) treatment of this soil could be of benefit in that rasising soil pH to the 7.5-7.5 range would increase perf and pitting depth times as indicated in the table below. This type of treatment may or may not be practical depending on the specifics of the situation. To improve metals longevity any more in this soil would require steel upgrading or other actions. At times, structural strength considerations may require heavier gauge steel than is used in the presented examples such that perforation and pitting times can be beyond the specified life span. Where this is not the case, cathodic protection along with coating or wrapping steel assets is one potential solution. Some of the other options include increased and/or specialized engineering fill, use of a polymer coating, or use of plastic, fiberglass or concrete assets. Based on these results, standard concrete mixes and rebar should be acceptable in this soil, although other testing could be needed to verify this.

SAMPLE ID	CT 18 ga	CT 12 ga	2 mm (Uhlig)	PARAMETER/ID	SMHS1-SMD/N
SMHS1-SMD/N	>15 yrs	>36 yrs	~10 yrs	pH	Ø
treated	>27 yrs	~60 yrs	~20 yrs	Rs	2-10
				SO4	Ø
				Cl	Ø
				Redox	–
				TOTAL POINTS	2-10

\\\\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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APPENDIX B
PREVIOUS SUBSURFACE EXPLORATION

BORING 2

EQUIPMENT: Simco with 6-inch Solid Flight Auger

DATE: 8/5/11
ELEVATION: 105-Feet*
*REFERENCE: Google Earth, 2011

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	
	19% P200	1000 UC	10	9.1	114	0	0			SILTY SAND (SM) medium brown, dry to moist, loose to medium dense, with rootlets and trace gravels [Fill]
	34% P200		8	15.6	105	1	5			
			52	18.7	110	2				
						3	10			CLAY with SAND (CL) gray and light brown, moist, very hard, medium plasticity, [Alluvium]
						4				
							15			
						5				
						6	20			CLAYEY SAND with GRAVEL (SC) medium to dark brown, moist, hard, low plasticity, 1/4" subrounded gravels [Alluvium]

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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FILE: 769.13 BL.dwg

BORING LOG

NUSD - San Marin High School
Cafeteria
Novato, California

Project No. 769.13 Date: 8/8/11

Drawn EDT
Checked

A-5
FIGURE

BORING 2 (CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters / feet	SAMPLE	SYMBOL (3)	
	17% P200		67	11.9	131	20			CLAYEY SAND with GRAVEL (SC) medium to dark brown, moist, hard, low plasticity, 1/4" subrounded gravels [Alluvium]
						7			CLAYEY SILT (ML) light gray, dry to moist, low to medium plasticity, [Alluvium]
						25			
						8			
						9			
						30			
			50/5"						*Sampler broke at bottom of borehole. Unable to retrieve sampler.
						10			Bottom of boring at 31.5-feet No groundwater observed during exploration
						35			
						11			
						12			
						40			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
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NUSD - San Marin High School Cafeteria
Novato, California

Project No. 769.13 Date: 8/8/11

Drawn EDT
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A-6
FIGURE

BORING 2

EQUIPMENT: Truck Mounted Drill Rig with 4.0-inch Solid Flight Auger
DATE: 1/4/18
ELEVATION: 114 - feet*
*REFERENCE: Google Earth, 2018

DEPTH meters / feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0 / 0			6" Topsoil over 12" Aggregate Base						
			Gravelly SAND (SW) Light brown, dry to moist medium dense, ~25% sub-angular gravel, fine to coarse grained sand, trace fines [Fill/Alluvium]	33	80	7.6			
1 / 5				30	107	14.5			
2									
3 / 10			Clayey SAND with Gravel (SC) Red brown, moist to wet, very dense, fine to coarse grained sand, ~10-15% fine sub-rounded gravel, ~30-40% low plasticity clay (varies slightly throughout sample), moderate to well cemented [Alluvium]	64	111	14.1		38.3% P200	
4									
15			Sandy Gravelly CLAY (CL) Red brown, moist to wet, very dense, moderately plastic, 30-35% fine to coarse sand, ~20-25% small to medium sub-rounded gravel [Alluvium]	89	116	18.9		51.8% P200	
5									
6 / 20			Clayey SAND with Gravel (SC) Red brown to yellow brown, medium dense, ~20% sub-rounded to sub-angular gravel, fine to coarse sand, ~40% moderate plasticity clay, well-lithified						

Water level encountered during drilling
Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

San Marin High School
STEM Building
Novato, California

Project No. 769.176 Date: 1/22/2018

Drawn ZMS
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A-5
FIGURE

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	BORING 2 (CONTINUED)	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	20			Clayey SAND with Gravel (SC) Red brown to yellow brown, medium dense, ~10-15% sub-rounded to sub-angular gravel, fine to coarse sand, ~40% moderate plasticity clay, well cemented [Alluvium]	43		19.0		41.6% P200	
7										
	25				43		13.6			
8				Stiffer drilling at 28 feet						
9	30			Sandstone Mottled orange and gray, moderately hard, moderately strong, highly to completely weathered [Franciscan Bedrock]	59		21.3			
10										
	35									
11										
12	40									

▽ Water level encountered during drilling

▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
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Novato, California

Project No. 769.176 Date: 1/22/2018

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A-6
FIGURE

DEPTH meters / feet	SAMPLE	SYMBOL (4)	BORING 2 (CONTINUED)	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
40			Sandstone Mottled orange and gray, moderately hard, moderately strong, highly to completely weathered, fine grained, massive [Franciscan Bedrock]	86/11"		21.4			
13			End of boring at 41.5-feet Groundwater encountered at 14.0 feet						
45									
14									
15									
50									
16									
55									
17									
18									
60									

Water level encountered during drilling

Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
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A-7
FIGURE