Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, California 94947
T 415 / 382-3444
F 415 / 382-3450

GEOTECHNICAL INVESTIGATION
MUIR BEACH OVERLOOK WATER TANK
MUIR BEACH, CALIFORNIA

November 17, 2009

Project 1615.01

Prepared For:
Muir Beach Community Services District
16 Miller Avenue
Suite 203
Mill Valley, California 94941

Attn: Mr. Leighton Hills

CERTIFICATION

This document is an instrument of service, prepared by or under the direction of the undersigned professionals, in accordance with the current ordinary standard of care. The service specifically excludes the investigation of radon, asbestos or other hazardous materials. The document is for the sole use of the client and consultants on this project. No other use is authorized. If the project changes, or more than two years have passed since issuance of this report, the findings and recommendations must be reviewed by the undersigned.

MILLER PACIFIC ENGINEERING GROUP
(a California corporation)

REVIEWED BY:

REGISTERED PROFESSIONAL ENGINEER
SCOTT A. STEPHENS
2398
Exp. 6/30/11
GEOTECHNICAL
STATE OF CALIFORNIA

Mike Jewett
Staff Geologist

Scott Stephens
Geotechnical Engineer No. 2398
(Expires 6/30/11)

Miller Pacific
ENGINEERING GROUP

GEOTECHNICAL INVESTIGATION
MUIR BEACH OVERLOOK WATER TANK
MUIR BEACH, CALIFORNIA

TABLE OF CONTENTS

FIGURES

APPENDIX A - SUBSURFACE EXPLORATION AND LABORATORY TESTING

GEOTECHNICAL INVESTIGATION
MUIR BEACH OVERLOOK WATER TANK
MUIR BEACH, CALIFORNIA

I. INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed water storage tank located adjacent to an existing tank near Muir Beach Overlook in Muir Beach, California. The site location is shown on Figure 1. This report is intended for the exclusive use of the Muir Beach Community Services District. No other use is authorized without the express written consent of Miller Pacific Engineering Group.

The purpose of our services is to conduct a geotechnical investigation, evaluate geologic hazards, identify constraints and develop recommendations to aid in the design and construction of the proposed water storage tank. The scope of our Phase 1 services is described in our proposal dated October 1, 2009 and includes the following geotechnical services:

- Summary of the geologic setting and seismicity;
- Exploration of subsurface soil conditions with 2 soil borings;
- Laboratory testing on select soil samples;
- Geologic hazards evaluation;
- Geotechnical recommendations and design criteria for the project; and,
- Preparation of this report.

Supplemental services are expected to include consultation during design, geotechnical plan review, and construction inspection and testing.

II. PROJECT DESCRIPTION

The project involves construction of a 45-foot diameter 200,000 gallon water tank adjacent to an existing 38-foot diameter 150,000 gallon water tank. The locations of the existing and proposed tank are shown on Figure 2. The existing tank is founded on a graded pad at the top of a topographic knoll. We understand this graded pad will be extended to create a level pad for the proposed tank. Grading for the project is anticipated to include maximum cuts of approximately 10 feet and minimal fill, if any. Based on the preliminary layout, portions of cut slopes are inclined steeper than 1:1 (horizontal:vertical).

The project owner is Muir Beach Community Services District. The tank designer and contractor is Natgun of Wakefield, Massachusetts. The project civil and structural engineers are not known at this date.

III. SITE CONDITIONS

A. Regional Geology

The project site is located within the Coast Range Geomorphic Province of California. The regional topography of this province is characterized by northwest-southeast trending mountain ridges and intervening valleys formed by tectonic activity between the Pacific and the North American Plates. Extensive faulting during the Pliocene Age (1.8 to 7 million years ago) formed the uneven depression that is now San Francisco Bay. More recent tectonic activity is concentrated along the San Andreas Fault Zone, a complex group of generally parallel northwesterly trending faults.

The regional bedrock geology consists of the Jurassic-Cretaceous (65-190 million years ago) Franciscan Assemblage which is made up of sedimentary, igneous, and metamorphic rocks that have been complexly folded, sheared, and altered through tectonic activity. Regional geologic mapping by the United States Geological Survey (2000) indicates the site is underlain by Franciscan Melange, which is typically composed of sandstone and shale with lesser amounts of volcanic and metamorphic rock. A regional geologic map is shown on Figure 3.

B. Seismicity

The project 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 single cracks in the earth's crust but typically are braids of breaks that comprise shatter zones which link to form networks of major and minor faults. Within the Bay Area, faults are concentrated along the San Andreas Fault zone. The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination and is radiated outward in the form of energy waves. The amplitude and frequency of earthquake ground motions 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, high-amplitude motion when moving through soft ground materials, such as bay mud.

1. Active Faults in the Region - The project site is located within a seismically active San Francisco Bay region and will therefore experience the effects of future earthquakes. Such earthquakes could occur on any of several active faults within the region. An "active" fault is one

that shows displacement within the last 11,000 years (i.e. Holocene) and has a reported average slip rate greater than 0.1 mm per year. 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 4.

2. Historic Fault Activity - Numerous earthquakes have occurred in the region within historic times. The results of our computer database search indicate that 29 earthquakes (Richter Magnitude 5.0 or larger) have occurred within 100 kilometers (62 miles) of the site area between 1735 and 2009. The five most significant historic earthquakes to affect the project site are summarized in Table A.

TABLE A
SIGNIFICANT EARTHQUAKE ACTIVITY
Muir Beach Overlook Water Tank
Muir Beach, California

Epicenter (Latitude, Longitude)	Historic Richter Magnitude	Year	Distance
37.80, -122.20	6.8	1836	34 km
37.60, -122.40	7.0	1838	33 km
37.70, -122.10	6.8	1868	46 km
38.20, -122.40	6.2	1898	40 km
37.70, -122.50	8.2	1906	19 km

References: Sources: USGS (2008)

3. Probability of Future Earthquakes - The historical records does not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probability in this region, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" (2007) to estimate the probabilities of earthquakes on active faults. Potential sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, and micro-seismicity, to arrive at estimates of probabilities of earthquakes with a Moment Magnitude greater than or equal to 6.7 by 2037.

The probability studies focus on seven "fault systems" within the Bay Area. Fault systems are composed of different, interacting fault segments capable of producing earthquakes within the individual segment or in combination with other segments of the same fault system. The probabilities for the individual fault segments in the San Francisco Bay Area are presented on Figure 4.

In addition to the seven fault systems, the studies included probabilities of “background earthquakes.” These earthquakes are not associated with the identified fault systems and may occur on lesser faults (i.e., West Napa) or previously unknown faults (i.e., the 1989 Loma Prieta and 2000 Mt. Veeder - Napa earthquakes). When the probabilities on all seven fault systems and the background earthquakes are combined mathematically, there is a 63 percent chance for a magnitude 6.7 or larger earthquake to occur in the Bay Area by the year 2037. Smaller earthquakes (between magnitudes 6.0 and 6.7), capable of considerable damage depending on proximity to urban areas, have about a 92 percent chance of occurring in the Bay Area by 2037 (USGS, 2007).

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.

C. <u>Surface Conditions</u>

The site sits on gently sloping ground at the top of a prominent knoll north of Muir Beach and is adjacent to an existing 150,000 gallon redwood tank constructed in 1965. The north side of the proposed tank pad slopes to the east at an approximate inclination of 1:5 (vertical:horizontal). The center of the proposed pad is currently occupied by an artificial berm which rises about 3 feet above the north side of the site. The south side of the berm slopes between 1.5:1 and 2:1 to the existing tank pad to the south. Much of the northern and central portions of the proposed pad are covered in heavy scrub brush and broom. Access to the existing pad is provided by a small, mulch-covered access road which rises from Seacape Drive at the southeast end of the artificial berm.

D. <u>Field Exploration and Laboratory Testing</u>

Subsurface conditions at the site were explored with two borings performed on October 30 2009. The borings ranged from 21.5 to 22.5 feet deep and were performed at the locations shown on Figure 2. The soils encountered were logged by our Field Geologist and samples were obtained for laboratory testing. The subsurface exploration and laboratory testing program are discussed in more detail in Appendix A. Descriptions of the soil and rock classification terms are presented on Figures A-1 and A-2. Boring Logs are presented on Figures A-3 through A-5.

We conducted laboratory testing of selected samples from the test borings in order to determine their relevant engineering properties. Laboratory tests included determination of moisture content, dry density, and unconfined compressive strength. The results of the laboratory tests are presented on the boring logs.

The boring logs, description of soils encountered and the laboratory test data reflect conditions only at the location of the sampling or testing at the time they were excavated or conducted. 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.

E. Subsurface Conditions

Boring 1, located on the north side of the proposed tank pad, was drilled to a depth of 22.5 feet. Approximately 7 feet of fill and residual soil consisting of silty sand and sand with clay overlies bedrock. The bedrock is moderately to highly weathered Franciscan sandstone and shale mélange to the depth of the boring at 22.5 feet. The rock varies in hardness and strength, generally increasing with depth, though some highly weathered and crushed strata were observed between layers of more competent rock. Groundwater was observed in a fracture zone at a depth of 18.5 feet.

Boring 2 was located on the existing access road at the south end of the proposed tank pad. This boring encountered approximately 7 feet of medium-dense fill and residual soil consisting of clayey sand and clay with sand. Beneath the soil cover, slightly to moderately weathered Franciscan sandstone was encountered to the boring depth of 21 feet, 7 inches, and was observed to increase in hardness and strength with depth. No groundwater was observed in Boring 2.

Because the borings were not left open for an extended period of time, groundwater observations may not accurately reflect stabilized groundwater levels. However, given the topography and subsurface conditions at the site, shallow groundwater is not expected.

IV. GEOLOGIC HAZARDS

A. General

This section identifies potential geologic hazards at the project site, their significant adverse impacts, and recommended mitigation measures. The significant geologic hazard at the project site is strong seismic ground shaking.

B. Fault Surface Rupture

Under the Alquist-Priolo Special Studies Zone Act, the California Geological Survey (CGS) produced 1:24,000 scale maps showing all active faults. The project site is about 4.0 kilometers from the San Andreas Fault. However, the site is not located within an Alquist-Priolo Special Studies Zone. The potential for fault surface rupture at the site is low.

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

C. Seismic Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 4, could cause moderate to strong ground shaking at the site. Estimates of peak bedrock accelerations are based on either deterministic or probabilistic methods.

1. Deterministic Methods – Deterministic methods use empirical relations developed from data collected during previous earthquakes to provide estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the site, their maximum credible magnitude, closest distance to the project area, and probable peak accelerations is provided in Table B.

TABLE B
ESTIMATED SEISMIC GROUND MOTIONS
Muir Beach Overlook Water Tank
Muir beach, California

Deterministic Hazard Analysis Fault	Moment Magnitude for Characteristic Earthquake	Closest Estimated Distance (kilometers)	Median Peak Ground Acceleration (g)
San Andreas	6.9	4.0	0.39
San Gregorio	6.9	10.9	0.23
Point Reyes	7.0	23.5	0.13
Hayward	6.9	24.8	0.12
Rodgers Creek	7.0	28.1	0.11

Sources: USGS (2009), Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Borzognia (2008), Chiou and Youngs (2008), Idriss (2008)

2. Probabilistic Methods – Probabilistic methods for determining peak bedrock accelerations estimate the probability of exceeding various levels of peak horizontal acceleration (i.e., earthquake ground motion) within a specified time period. The methodology has been developed in recent years by recognized seismologists, earthquake engineers, and scientists. The seismic hazard evaluation involves combining the following: the probability that an earthquake will occur within a specified time period (commonly termed recurrence relationship); the probability that a given earthquake rupture surface is within a specified distance from the site; and, the probability that the peak horizontal acceleration at the project site will exceed a specified level.

In evaluating the seismic hazards associated with the subject site, we have considered both a PGA that has a 2 percent probability of being exceeded in 50 years and a PGA that has a 10 percent probability of being exceeded in 50 years (design basis earthquake PGA_{DBE}). For this analysis, we used the USGS Seismic Hazard Curves and Uniform Hazard Response Spectra, Version 5.0.9a. The estimated PGA_{DBE} for the site was calculated as 1.1 g for a 2% probability of exceedance and 0.66 g for a 10% probability of exceedance, both for 5 percent damping.

The potential for strong seismic shaking at the project site is high. The San Andreas Fault is the closest source for future earthquakes, and an earthquake in the area would most likely originate from the Rodgers Creek or the Hayward Fault. The most significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: Less than significant with mitigation

Mitigation: *Mitigation for seismic shaking includes designing the structures in accordance with the most recent version of the California Building Code (CBC, 2007) or the American Water Works Association (AWWA, 2006). Recommended seismic coefficients are provided in Section V of this report.*

D. Liquefaction Potential

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. This phenomenon can occur where there are saturated, loose, granular deposits subjected to seismic shaking. Liquefaction-related phenomena include settlement, flow failure, and lateral spreading. We did not encounter soils susceptible to liquefaction during our exploration. Therefore, the probability of damage due to liquefaction is low.

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

E. Seismic Induced Ground Settlement

Ground shaking can induce settlement of loose granular soils above the water table. Considering the relatively shallow bedrock at the project site, the probability of seismic induced settlement is low.

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

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 channel banks. Since these conditions do not exist at the site, the probability of lurching and ground cracking is low.

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

G. Settlement

New surface loads can cause consolidation of soft clays or compression of loose soils. The foundation of the new tank will bear on dense sandstone bedrock and stiff clayey fill, hence the probability of damage due to significant settlement of the ground surface is low.

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

H. <u>Erosion</u>

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flows. The potential for erosion on the tank pad is low, but the potential for minor erosion of the cut slope above the tank is moderate.

Evaluation: *Less than significant with mitigation*
Mitigation: *Site grading should be performed in accordance with the recommendations and criteria presented in Section V of this report. The project Civil Engineer should design site drainage to collect surface water into a storm drain system and discharge water at an appropriate location. Re-establishing vegetation on disturbed areas will also be required to minimize erosion. Erosion control measures during and after construction should conform to the most recent version of the Erosion and Sediment Control Field Manual (California Regional Water Quality Control Board, 2002).*

I. <u>Seiche and Tsunami</u>

Seiche and tsunamis are short duration earthquake-generated water waves in 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 in the San Pablo and San Francisco Bays. Considering the elevation of the project site, the likelihood of inundation or damage from a seiche or tsunami wave is remote.

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

J. <u>Flooding</u>

The adverse impact from flooding is water damage to structures. The site is located on a knoll nearly 500 feet above sea level, thus the probability of damage due to large-scale flooding is remote.

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

K. Expansive Soil

Expansive soil occurs when clay particles interact with water causing volume changes in the clay soil. The clay soil may swell when saturated and shrink when dried. This phenomenon generally decreases in magnitude with increasing confinement pressure at depth. These volume changes may damage lightly loaded foundations, flatwork, and pavement. Expansive soil also causes soil creep on sloping ground. We did not observe expansive soil conditions during our subsurface exploration; therefore the potential for expansive soil damage is low.

Evaluation: *No significant impact*

Mitigation: *No mitigation measures are required.*

L. Slope Stability

Available published maps do not show any active or dormant landslides on the site, nor were any observed during our field reconnaissance in the immediate area of the proposed tank site. The new tank pad is set back sufficiently from the bluffs above the Pacific Ocean such that calving or collapse of the cliffs is not expected to affect the tank.

Evaluation: *Less than significant with mitigation*

Mitigation: *Site grading and allowable slope inclination recommendations presented in Section V of this report should be incorporated into the project planning and design.*

V. CONCLUSIONS AND RECOMMENDATIONS

A. General

Based on the results of our investigation, we conclude that the project is feasible and the site is suitable for the planned water storage tank. The primary geotechnical issues are strong seismic ground shaking and providing uniform foundation support for the tank foundation. Recommendations to address these and other geotechnical issues are presented in the subsequent sections of this report.

B. Site Grading

Site grading is expected to consist primarily of cuts up to 10 feet tall. Site preparation and grading to protect the tank pad should conform to the following recommendations and criteria.

1. Surface Preparation – Clear all trees, brush, roots, over-sized debris, and organic material from areas to be graded. Any loose soil or rock at subgrade will need to be excavated to expose firm natural soils or bedrock. For the tank pad and access road, the exposed subgrade surface should be moisture conditioned to near the optimum moisture content and compacted to at least 90% relative compaction (ASTM D-1557) to produce a firm and unyielding surface. Subgrade areas exposing bedrock need not be recompacted.

2. Excavation – Much of the excavation for the tank pad will be in weathered sandstone bedrock with some areas of weaker shale mélange. It is our opinion that most of this bedrock can be excavated with conventional equipment (large dozer or excavator). It is possible that locally hard rock will be encountered and the use of hard rock excavation equipment or methods may be required.

3. Slopes – Cut and fill slope inclinations should not be steeper than 2:1 (horizontal:vertical). The project does not currently include any fill slopes. However, the cut slopes are planned much steeper. Retaining walls may need to be incorporated into the design to keep slopes at 2:1 or shallower. All graded slope surfaces should be trimmed to remove loose soil. All graded slopes should be covered with straw mats or similar erosion-resistant material and planted as soon as possible upon completion of grading and prior to the start of rains.

   For temporary slopes, the Federal Occupational Safety and Health Administration (OSHA) has promulgated rules for Excavations, 29 CFR Part 1926, October 31, 1989. OSHA dictates allowable slope configurations and minimum shoring requirements based on categorized soil types. In conformance with OSHA's categorization, the fill soils (up to 8 feet thick) are "Type B" and the bedrock below is characterized as "Type A." The Contractor may elect to use a variety of shoring and temporary slope configurations, but his operations must conform to Federal and State OSHA regulations. Additionally, it should be made clear that the safety of excavations, slopes, construction operations, and personnel are the sole responsibility of the Contractor.

Performance of the temporary cut slopes will be influenced by the length of time the cut is unsupported, seepage and surface runoff over the cut face, bedding planes of rock and soil materials and other factors. Temporary unsupported vertical cuts shall not exceed 5 feet and may experience sloughing, especially during wet weather conditions, and cleanup of debris at the base of the cut may be required. Permanent and temporary cut slopes should be inspected by a Geotechnical Engineer during construction.

4. Compacted Fill – Fill, backfill, and scarified subgrades should be conditioned to a moisture content within 3 percent of the optimum moisture content. Properly moisture conditioned soils should be placed in loose horizontal lifts of 8 inches thick or less and uniformly compacted to at least 90 percent relative compaction.

   The fill material shall consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40, (3) have a Plasticity Index less than 20, and (4) have a maximum particle size of 6 inches. We judge that most of the soil and rock mixtures generated from on-site excavations are suitable for use as fill provided the maximum particle sizes are less than 6 inches. Any imported fill material needs to be tested to determine its suitability for use as fill material.

C. Seismic Design and Site Specific Response Spectrum

Minimum mitigation of ground shaking includes seismic design of the structure in conformance with the provisions of the most recent version (2007) of the California Building Code. The most significant effects of earthquake shaking can be mitigated by close adherence to the seismic provisions of the current (2007) edition of the CBC or the current AWWA Standard for Welded Carbon Steel Tanks for Water Storage (2006). However, since the goal of the building code is protection of life safety, some tank damage may still occur during strong ground shaking.

The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and close proximity to the San Andreas Fault, we recommend the CBC / AWWA coefficients and site values shown in Table C below for use in equations 16-37[(1)] and 16-38 to calculate the design base shear of the new construction. To determine site seismic coefficients, we used the USGS Seismic Hazard Curves and Uniform Hazard Response Spectra, Version 5.0.9a, using the latitude and longitude shown on Figure 4.

TABLE C
SEISMIC FACTORS
Muir Beach Overlook Water Tank
Muir Beach, California

Factor Name	Coefficient	CBC Table	AWWA Table/ Figure[A]	Site Specific Value
Site Class[1]	$S_{A,B,C,D,E,\text{ or }F}$	1613.5.2	Table 25	S_B
Site Coefficient	F_a	1613.5.3 (1)	Figure 7	1.00
Site Coefficient	F_v	1613.5.3 (2)	Figure 8	1.00
Spectral Acc. (short)	S_s	1613.5.1	Table 26	1.925 g
Spectral Acc. (1-sec)	S_1	1613.5.1	Table 27	1.01 g

(1) Soil Profile Type S_B Description: Rock with a shear wave velocity greater than 2,500 ft/s and less than 5,000 ft/s

Site specific probabilistic bedrock accelerations with a 2% chance of exceedence in 50 years (2500 year return period) were evaluated using the USGS Seismic Hazard Curves and Uniform Hazard Response Spectra, Version 5.0.9a. We also evaluated the deterministic bedrock accelerations at the site. The probabilistic would be more appropriate for critical structures and the CBC / AWWA design is most often used for residential, commercial and industrial projects.

The calculated probabilistic, deterministic and CBC / AWWA design response spectrums are shown on Figure 5. The greater of the deterministic or CBC / AWWA response should be used as the lower bound response spectra as shown on Figure 5. For a more critical structure or to reduce the amount of damage during a strong seismic event, the probabilistic spectra would be recommended.

D. Foundation Design

The results of our subsurface exploration suggest that the tank will be founded on a contact between dense sandstone bedrock to the north and stiff clayey fill to the south and east. Due to deeper bedrock along the southern and eastern portions of the tank, differential settlement of the tank should be anticipated unless uniform bedrock support is provided. For design purposes, differential settlement of a shallow ring footing is estimated at 0.5 to 1 inch.

To minimize differential settlement, we recommend the tank foundations bear on bedrock. We recommend a deepened ring footing or drilled, cast-in-place piers where bedrock will not be encountered at foundation subgrade. Geotechnical design criteria for the tank foundation are presented in Table D.

TABLE D
FOUNDATION DESIGN CRITERIA
Muir Beach Overlook Water Tank
Muir Beach, California

Shallow Spread Footings	
Minimum depth:	18 inches
Allowable bearing capacity: [1, 2]	
Fill/Residual Soils:	3,000 psf
Bedrock:	5,000 psf
Base friction coefficient:	0.35
Lateral passive resistance: [3]	
Bedrock	450 pcf
Drilled Piers	
Minimum embedment in weathered bedrock:	3 feet
Skin Friction: [3, 4]	
Fill/Residual Soils (up to 7 feet):	1,000 psf
Weathered Bedrock:	2,500 psf
Lateral Passive Resistance: [3]	
Fill/Residual Soils (up to 7 feet):	350 psf
Weathered Bedrock:	450 psf

Notes:
(1) Foundation to bear on competent bedrock.
(2) Dead plus live loads. Can increase values by 1/3 for total loads including seismic.
(3) Equivalent fluid pressure. Ignore upper 12 inches unless confined by concrete or asphalt pavements. For piers, apply values over effective width of two pier diameters.
(4) Uniform pressure distribution. Uplift resistance equals 80% of the downward skin friction.

E. Pipeline

Excavations for utilities will be in hard sandstone bedrock and stiff clayey fill. Trench excavations having a depth of 5 feet or more must be excavated and shored in accordance with OSHA regulations. Pursuant to OSHA classifications, on-site fill soils area Type C, while the bedrock is a Type A.

A minimum of 4 inches of bedding material shall be placed in the bottom of the trench excavation for pipe bedding. The bedding material shall be continuous around the pipe and extend at least 6 inches above the top of pipe. The bedding material shall be compacted to at least 90 percent relative compaction (R.C.). The bedding material and compaction requirements shall meet the criteria presented in the Muir Beach Community Services District standard specifications.

F. Access Road Design

Site grading for the paved areas that will be located around the tank should be performed as described in Section V.B., including over excavation of loose soils. Given that the tank site traffic will consist of infrequent light to moderately heavy trucks we recommend the following light pavement sections. The following pavement section is based on a Traffic Index of 3.0, and a minimum subgrade R-Value of 20. The assumed subgrade soil conditions should be confirmed during construction when the subgrade is exposed in the pavement areas

TABLE E
RECOMMENDED PAVEMENT SECTION
Muir Beach Overlook Water Tank
Muir Beach, California

T.I.	Subgrade Conditions	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
3	Soil (R-value = 20)	2.0	6.0

The upper 6 inches of subgrade in pavement areas must be scarified, moisture conditioned to near the optimum water content, and then compacted to a minimum 95 percent relative compaction. The compacted surface must also be non-yielding when proof-rolled with heavy construction equipment.

The base rock should consist of compacted Class 2 Aggregate Base (Caltrans 2000) compacted to achieve at least 95 percent relative compaction and a non-yielding surface when proof-rolled with heavy construction equipment.

G. Site Drainage

Storm water runoff should be carefully controlled to reduce erosion of the slopes below the tank. We understand that the current surface drainage pattern will not be significantly modified. To prevent water ponding near the tank, slope the adjacent paved areas downward at least 0.1 feet for 5 feet (2 percent). Unpaved areas should be sloped downward at least 0.25 feet for 5 feet (5 percent) from the tanks.

VI. SUPPLEMENTAL SERVICES

We should review the plans and specifications when they near 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 the site grading, compaction of fill material, and foundation excavations to confirm that subsurface conditions are as expected and adjust foundation depths and other elements of the design, if warranted.

LIST OF REFERENCES

Abrahamson, N. and Silva, W., Summary of the Abrahamson and Silva NGA Ground-Motion Relations," EERI Earthquake Spectra, Volume 24, Number 1 Feb. 2008

American Society for Testing and Materials, "1994 Annual Book of ASTM Standards, Section 4, Construction, Volume 4.08, Soil and Rock; Dimension Stone; Geosynthetics," ASTM, Philadelphia, 1992.

American Water Works Association, Manual of Water Supply Practices, "Welded Carbon Steel Tanks for Water Storage", AWWA Manual D100-05, 2006.

Blake, T.F., "A Computer Program for the Probabilistic Estimation of Peak Acceleration and Uniform Hazard Spectra Using 3-D Faults as Earthquake Sources", FRISKSP (Version 4.00), 2001.

Boore, D.M. and Atkinson, G.M., "Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA a Spectral Periods between 0.01 s and 10.0 s," EERI Earthquake Spectra, Volume 24, Number 1, February 2008

California Building Code, California Building Standards Commission, Sacramento, California, California Code of Regulations, Title 24, Part 2, Volume 2, Chapter 16, 2007.

California Department of Conservation, Division of Mines and Geology, "Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Central Coast Region", DMG CD 2000-004, 2000.

California Department of Conservation, Division of Mines and Geology, "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada to be Used with the 1997 Uniform Building Code", International Conference of Building Officials, Whittier, California, February 1998.

California Department of Conservation, Division of Mines and Geology, "Probabilistic Seismic Hazard Assessment for the State of California", Open-File Report 96-08, 1996.

California Department of Transportation (Caltrans), Standard Specifications, 2000.

Campbell, K., Bozorgnia, Y., "NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s," EERI Earthquakes Spectra, Volume 24, Number 1, February 2008

Chiou, B. and Youngs, R., "An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra," EERI Earthquakes Spectra, Volume 24, Number 1, February 2008

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C. J., "The revised 2002 California Probabilistic Seismic Hazard Maps, California", USGS/CGS: dated June 2003.

Idriss, I., "An NGA Empirical Model for Estimating the Horizontal Spectral Values Generated By Shallow Crustal Earthquakes," EERI Earthquakes Spectra, Number 1, February 2008

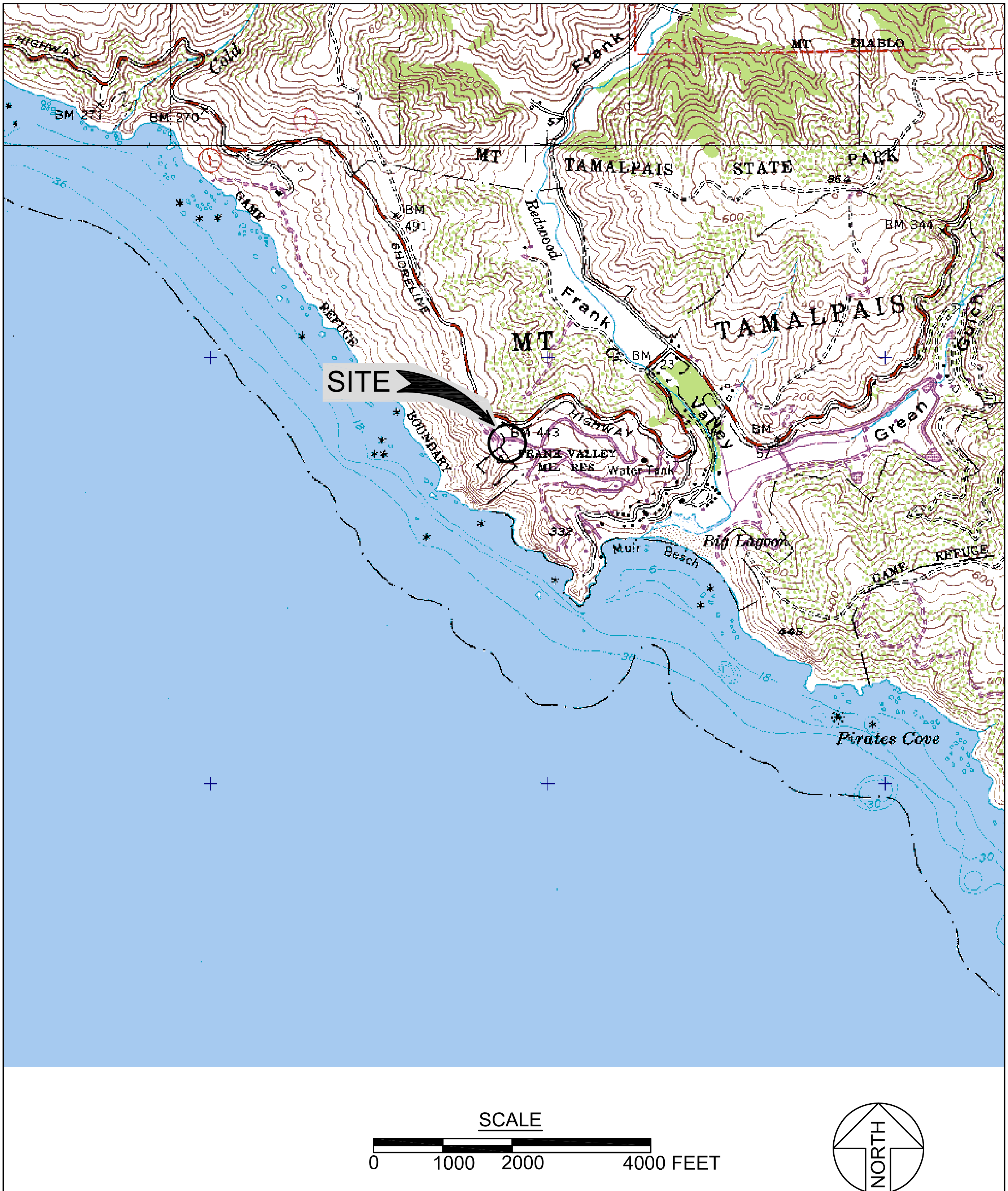
Occupational Safety and Health Administration (OSHA), Title 29 Code of Federal Regulations, Part 1926, 2005.

Uniform Building Code, International Conference of Building Officials, Volume 2, Chapter 16, 1997.

United States Geological Survey (USGS), "Earthquake Probabilities in the San Francisco Bay Region, 2002 to 2031 – A Summary of Finding," The Working Group on California Earthquake Probabilities, Open File Report 99-517, 2003.

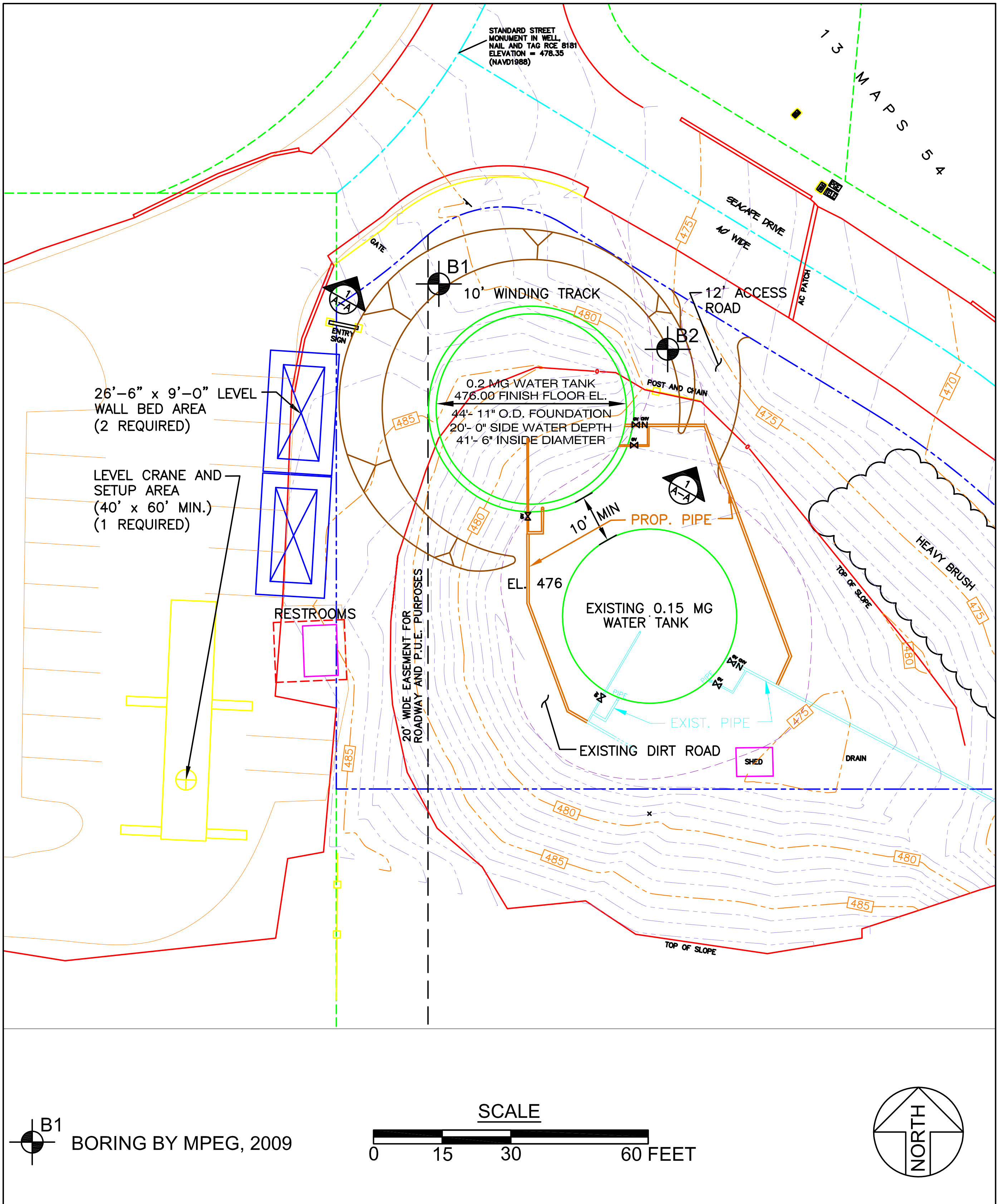
United States Geological Survey (USGS), Earthquake Hazards Program , Earthquake Circular Area Search http://neic.usgs.gov/neis/epic/epic_circ.html, 2009

United States Geological Survey (USGS), "National Seismic Hazards Map Program," Version 5.0.9, October 06, 2008.



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
		Muir Beach CSD Muir Beach Water Tank Muir Beach, California Project No. 1615.01 Date: 10/23/09
		Drawn MFJ Checked
		1 FIGURE

FILE: 1615.01SM.dwg



Miller Pacific
ENGINEERING GROUP

FILE: 1615.01SP.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

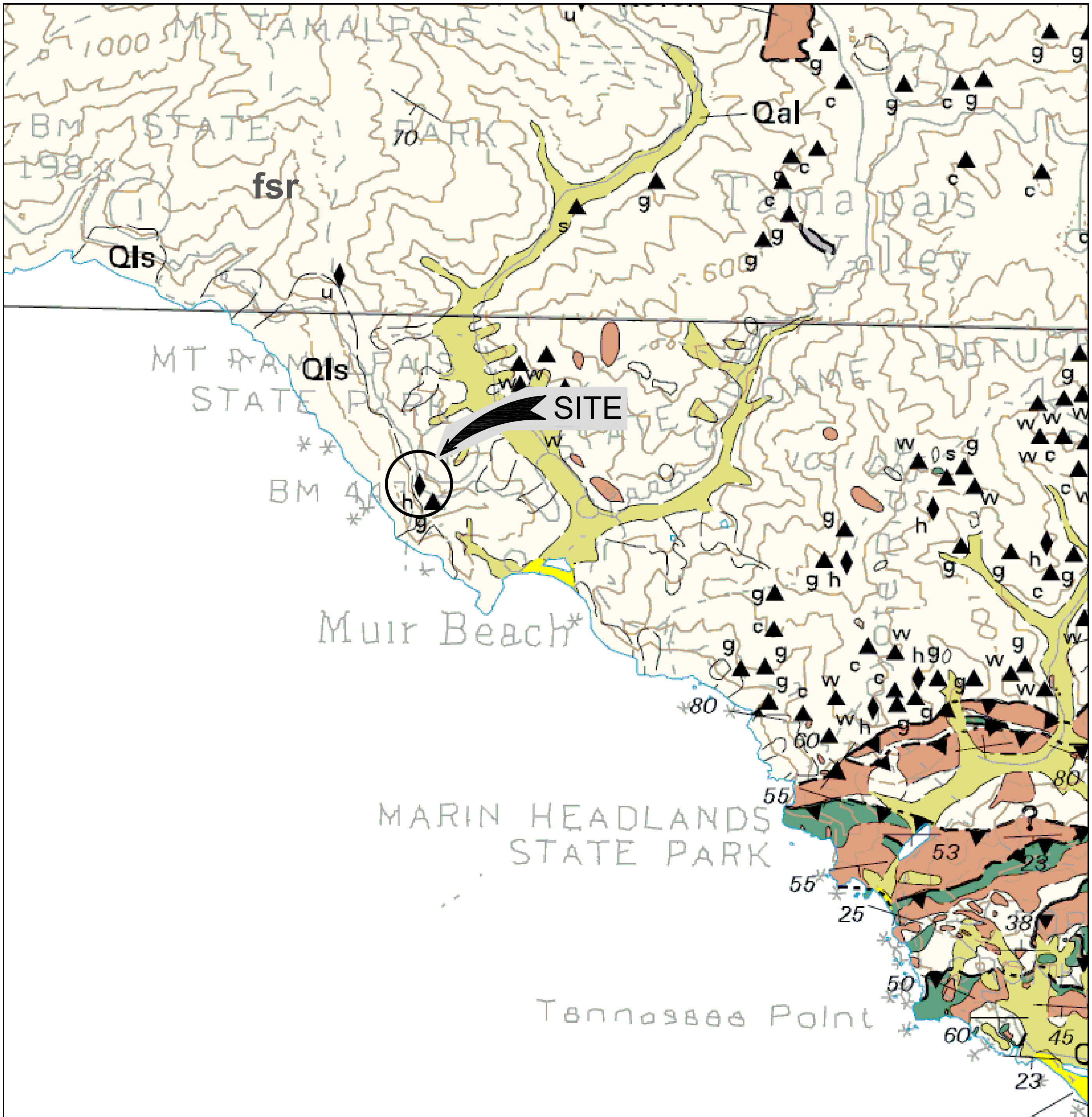
SITE PLAN

Muir Beach CSD
Muir Beach Water Tank
Muir Beach, California

Project No. 1615.01 Date: 10/23/09

Drawn MFJ
Checked

2
FIGURE



LEGEND

Qls - Quarternary landslide debris

Qal - Quarternary alluvium, typically stream and channel deposits consisting of poorly sorted silts, sands and gravels

fsr - Franciscan Melange, typically sandstone and shale with lesser amounts of volcanic and metamorphic rocks

NORTH

NOT TO SCALE

Miller Pacific
ENGINEERING GROUP

FILE: 1615.01GM.DWG

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

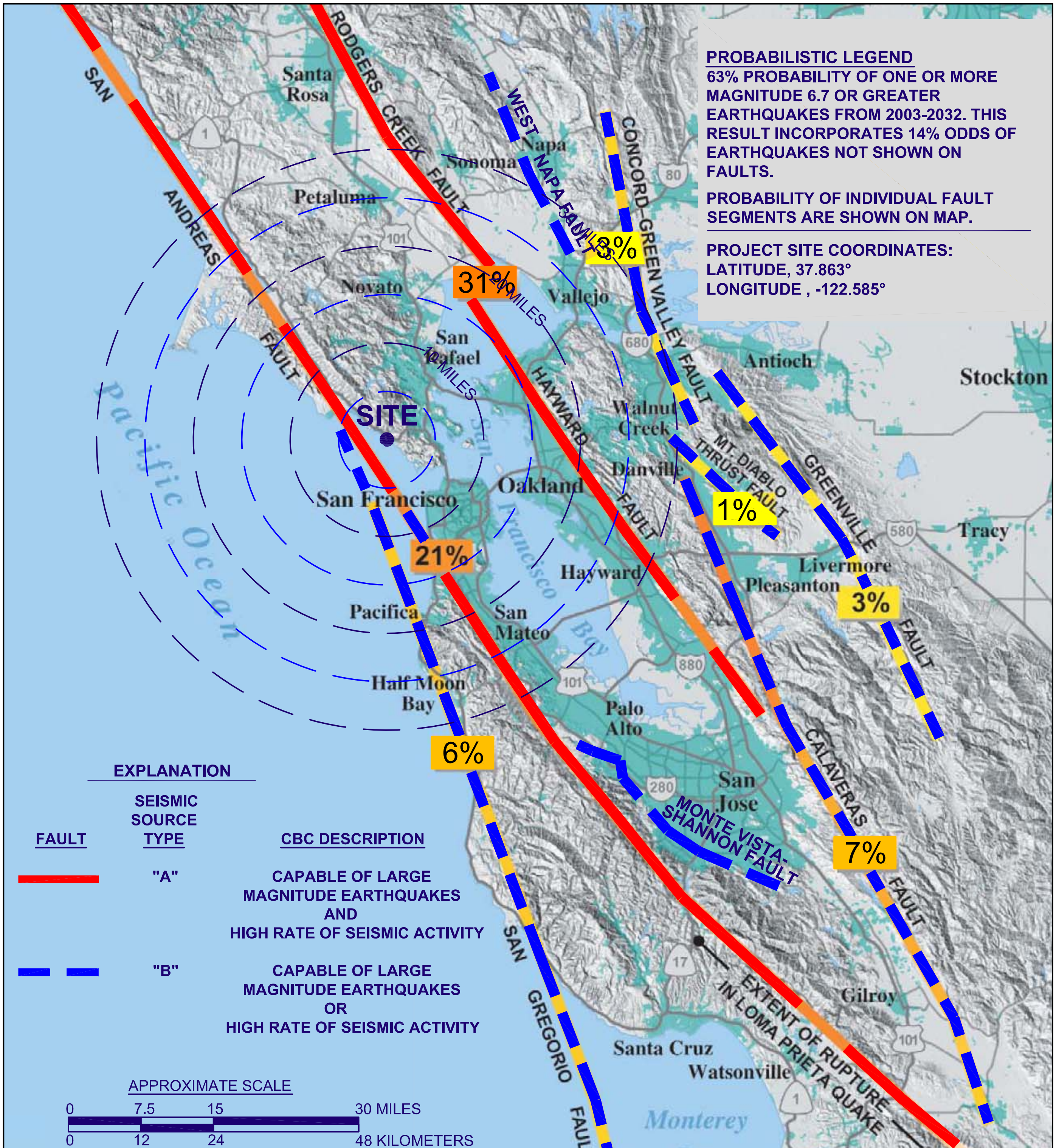
GEOLOGIC MAP

Muir Beach CSD
Muir Beach Water Tank
Muir Beach, California

Project No. 1615.01 Date: 10/23/09

Drawn MFJ
Checked

3
FIGURE



REFERENCES:

1) REVISED 2002 CALIFORNIA PROBABILISTIC SEISMIC HAZARD MAPS, CGS, 2003.

2) ACTIVE FAULT MAP MODIFIED FROM SUMMARY OF EARTHQUAKE PROBABILITIES IN THE S.F. BAY REGION, 2002 - 2031, THE WORKING GROUP ON CALIFORNIA EARTHQUAKE PROBABILITIES, 2003.

N

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1615.01FM.dwg

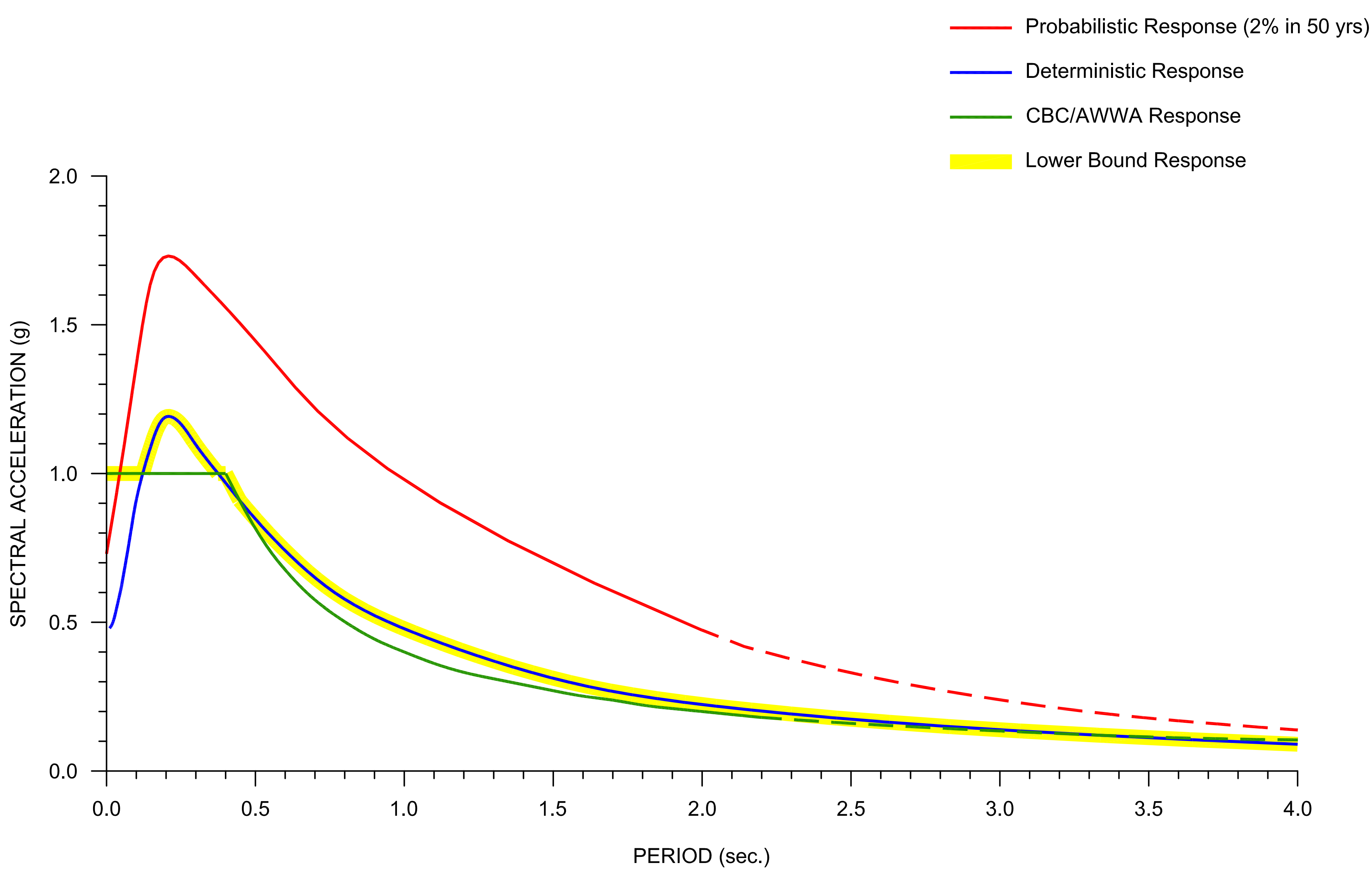
ACTIVE FAULT MAP

Muir Beach CSD
Muir Beach Water Tank
Muir Beach, California

Project No. 1615.01 Date: 10/23/09

Drawn MFJ
Checked

4
FIGURE



Miller Pacific
ENGINEERING GROUP

FILE: 1615.01SPECTRA.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

SITE SPECIFIC DESIGN SPECTRA

Muir Beach CSD
Muir Beach Water Tank
Muir Beach, California

Project No. 1615.01 Date: 11/11/09

Drawn MFJ
Checked

5
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. Soil and Rock Classification Systems

We explored subsurface conditions at the site with two exploratory borings drilled on October 30, 2009. Borings were excavated to a depth between 21.5 and 22.5 feet using truck-mounted equipment.

The soils encountered were logged and identified by our field geologist in general accordance with ASTM Standard D 2487, "Field Identification and Description of Soils (Visual-Manual Procedure)." This standard is briefly explained on Figure A-1, Soil Classification Chart and Key to Log Symbols and Figure A-2, Rock Classification Chart. The boring logs are presented on Figures A-3 to A-5.

B. 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; and

- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166;

The moisture content, dry density, and unconfined compressive strength test results are shown on the exploratory Boring Logs, Figures A-3 through A-5.

The exploratory boring logs, description of soils encountered and the laboratory test data reflect conditions only at the location of the excavation 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
- HAND SAMPLER
- STANDARD PENETRATION TEST
- ROCK CORE
- THIN-WALLED / FIXED PISTON
- 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

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1615.01BL.dwg

SOIL CLASSIFICATION CHART

Muir Beach CSD
Muir Beach Water Tank
Muir Beach, California

Project No. 1615.01 Date: 11/2/09

Drawn MFJ
Checked

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.

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1615.01BL.dwg

ROCK CLASSIFICATION CHART

Muir Beach CSD
Muir Beach Water Tank
Muir Beach, California

Project No. 1615.01 Date: 11/2/09

Drawn MFJ
Checked

A-2
FIGURE

BORING 1

EQUIPMENT: Mobile B-53 with 6" Solid Flight Augers

DATE: 10/30/09

ELEVATION: 481 Feet*

*REFERENCE: MBCSD Topo Survey, 2009

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)
			24	19.5	99
		2300 UC	29	19.3	112
			50/3"	7.2	
		1100 UC	37	12.3	124
		900 UC	18	15.8	116
			49	8.4	134

SANDY SILT WITH GRAVEL (ML)
Dark brown, moist, medium stiff, low plasticity, ~30% very fine to coarse sand, ~20% fine to medium-grained, angular to subrounded gravel [FILL]

CLAY WITH SAND (CL)
Medium gray with orange mottling, moist, stiff, low plasticity, ~20% very fine to fine sand, occasional angular gravel [FILL]

SAND WITH CLAY (SP)
Light brown, moist, dense, very fine to medium-grained, ~15% low plasticity clay [RESIDUAL SOIL]

SANDSTONE
Medium brown, moderately weathered, crushed, friable, hard, fine to medium-grained [BEDROCK]

grades to medium gray-brown, highly weathered, crushed, weak to friable, moderately hard

MELANGE
Medium gray-brown, highly weathered, crushed, weak to friable, low to moderate hardness, ~50% each sandstone and shale [BEDROCK]

SANDSTONE
Medium gray, moderately weathered, intensely fractured, weak, hard [BEDROCK]

Encountered groundwater at 18.5 feet

MELANGE

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

Miller Pacific
ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1615.01BL.dwg

BORING LOG

Muir Beach CSD
Muir Beach Water Tank
Muir Beach, California

Project No. 1615.01 Date: 11/2/09

Drawn MFJ
Checked

A-3
FIGURE

BORING 1 (CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH feet	DEPTH meters	SAMPLE	SYMBOL (3)	DESCRIPTION
						20				SANDSTONE
										MELANGE Medium gray, highly to completely weathered, crushed, friable, moderately hard [BEDROCK]
			42	6.7	138		7			
										Bottom of boring at 22.5 feet. Groundwater measured at 20.0 feet immediately after drilling.
						25	8			
							9			
						30				
							10			
						35	11			
							12			
						40				

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

Miller Pacific
ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1615.01BL.dwg

BORING LOG

Muir Beach CSD
Muir Beach Water Tank
Muir Beach, California

Project No. 1615.01 Date: 11/2/09

Drawn MFJ
Checked

A-4
FIGURE

BORING 2

EQUIPMENT: Mobile B-53 with 6" Solid Flight Augers

DATE: 10/30/09

ELEVATION: 477 Feet*

*REFERENCE: MBCSD Topo Survey, 2009

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH (meters / feet)	DESCRIPTION
						0 / 0	SANDY CLAY (CL) Medium to dark brown, moist, medium stiff, low plasticity, ~30-40% very fine to fine sand, occasional angular gravel [FILL]
		1800 UC	19	29.0	99	1 / 5	CLAY WITH SAND (CL) Medium gray with orange mottling, moist, stiff, low plasticity, ~20% very fine to fine sand [FILL]
		1400 UC	49/9"	13.6	123	2	SANDSTONE Medium brown, moderately weathered, closely fractured, weak to moderately strong, moderately hard [BEDROCK]
			50/4"	8.4		3 / 10	grades to crushed, moderately strong at 9.0 feet; grades slightly weathered, strong, hard at 9.5 feet
			25/2"	5.1		4	
			25/1"	6.3		15 / 5	noted very hard, slow drilling at 16.0 feet; grades light brown, fresh at 17.0 feet
			25/1"	5.3		6 / 20	Bottom of boring at 20 feet 7 inches. No groundwater encountered during drilling.

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

Miller Pacific
ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1615.01BL.dwg

BORING LOG

Muir Beach CSD
Muir Beach Water Tank
Muir Beach, California

Project No. 1615.01 Date: 11/2/09

Drawn MFJ
Checked

A-5
FIGURE