Miller Pacific
ENGINEERING GROUP

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

GEOTECHNICAL INVESTIGATION
FAIR-ANSELM PLAZA
FAIRFAX, CALIFORNIA

September 28, 2009

Project No. 1598.01

Prepared For:
Dan and Mathew Friedman, Owners
Fair-Anselm Plaza
771B Center Boulevard
Fairfax, California 94930

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, toxic mold and other biological pollutants, and other hazardous materials. The document is for the sole use of the client and consultants on this project. Use by third parties or others is expressly prohibited without written permission. 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

Mike Jewett
Staff Geologist

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

GEOTECHNICAL INVESTIGATION
FAIR-ANSELM PLAZA
FAIRFAX, CALIFORNIA

TABLE OF CONTENTS

I.	INTRODUCTION	Page 1
II.	PROJECT DESCRIPTION	2
III.	SITE CONDITIONS	3
	A. Regional Geology	3
	B. Seismicity	3
	C. Surface Conditions	5
	D. Field Exploration	5
	E. Subsurface Conditions	6
	F. Sonic Echo Testing	7
IV.	GEOLOGIC HAZARDS	8
	A. General	8
	B. Fault Surface Rupture	8
	C. Seismic Shaking	8
	D. Liquefaction Potential	9
	E. Seismic Induced Ground Settlement	10
	F. Erosion	10
	G. Expansive Soil	11
	H. Flooding	11
	I. Settlement	11
	J. Lurching, Lateral Spreading and Ground Cracking	12
	K. Seiche & Tsunami	12
	L. Slope Instability	12
V.	CONCLUSIONS AND RECOMMENDATIONS	14
	A. Conclusions	14
	B. Existing Improvements	14
	C. Bank Stabilization	14
	D. Seismic Design	15
	E. Foundation Design Criteria	16
	F. Retaining Walls	18
	G Site Grading	19
VI.	SUPPLEMENTAL GEOTECHNICAL SERVICES	21
LIST OF REFERENCES		22

FIGURES

Site Location Map	Figure 1
Site Plan	2
Geologic Map	3
Active Fault Map	4
Topographic Survey	5
Liquefaction Susceptibility Map	6
Schematic Rip-Rap Buttress	7
Schematic Retaining Wall	8

APPENDIX A – SUBSURFACE EXPLORATION AND LABORATORY TESTING

Soil Classification Chart	Figure A-1
Rock Classification Chart	A-2
Boring Logs	A-3 to A-8
Sieve Analyses Results	A-9
Boring Log by Neil O. Anderson and Associates	A-10

APPENDIX B – SONIC ECHO TESTING

Sonic Echo Testing Report, ABE Engineering	Figure B-1 to B-2
Sonic Echo Test Plots, ABE Engineering	B-3 to B-5

GEOTECHNICAL INVESTIGATION
FAIR-ANSELM PLAZA
FAIRFAX, CALIFORNIA

I. INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed foundation and retaining wall improvements at the Fair-Anselm Plaza in Fairfax, California. The project site location is shown on Figure 1.

This report is intended for the exclusive use of Fair-Anselm Plaza. No other use is authorized without the written consent of Miller Pacific Engineering Group.

The scope of our Phase 1 services is described in our proposal dated June 25, 2009 and includes the following geotechnical services:

- Review of available published geologic and geotechnical reference data;
- Exploration of subsurface conditions with 4 borings at the project site;
- Laboratory testing of selected samples to determine the pertinent engineering properties of the soil layers;
- Performance of sonic echo testing on several existing piers to evaluate their depth and potential continued use;
- Evaluation of geologic hazards relevant to site development and discussion of potential mitigative measures where appropriate;
- Development of geotechnical recommendations for the project; and
- Preparation of this report summarizing our geotechnical recommendations and design criteria.

This report completes our Phase 1 services. Supplemental services are expected to include geotechnical consultation/plan review and construction observation and testing.

II. PROJECT DESCRIPTION

The project site is located at the southwest corner of Pastori Avenue and Center Boulevard in Fairfax, California. The existing structure sits adjacent to the northern bank of San Anselmo Creek, and the creek has incised a channel which has eroded the northern bank approximately fifty feet beneath the structure, exposing two rows of structural piers. Erosion along the northern bank of the creek has also undermined and caused failure of a retaining wall adjacent to the west end of the building.

The project consists of mitigating creek bank erosion adjacent to and beneath the building and evaluating the structural integrity and continued usefulness of the existing foundation piers. Site conditions in the vicinity of the planned improvement area are shown on Figure 2.

At this time, the project design team includes Dan and Mathew Friedman (Owners), Scott Hostrosser of IPA (Planner), and Oberkamper and Associates (Civil Engineers). We understand a structural engineer will be retained.

III. SITE CONDITIONS

A. Regional Geology

The site is located within the Coast Ranges Geomorphic Province of California. The regional bedrock geology consists of complexly folded, faulted, sheared, and altered sedimentary, igneous, and metamorphic rock of the Franciscan Complex of Jurassic-Cretaceous age (65-190 million years ago). The Franciscan Complex is characterized by a diverse assemblage of greenstone, sandstone, shale, chert, and mélange, with lesser amounts of conglomerate, calc-silicate rock, schist and other metamorphic rocks.

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

Regional geologic mapping (Rice 1976, USGS 2000) shows the site is located near a geologic contact between alluvial soils and bedrock comprised of sandstone and shale of the Franciscan Complex. Alluvial soils consist of gravel, sand and silt that are poorly to moderately sorted. The mapped geologic conditions at the project site are shown on Figure 3.

B. Seismicity

1. Active Faults in the Region - The project site is located within a seismically active area and will therefore experience the effects of future earthquakes. 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.

An “active” fault is one that shows displacement within the last 11,000 years and, therefore, is considered more likely to generate a future earthquake than a fault that shows no sign of recent rupture. The California Geologic Survey (2003) 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. The San Andreas Fault is the nearest known active fault, approximately 12 kilometers (7 miles) to the southwest of the project site.

2. Historic Fault Activity - Numerous earthquakes have occurred in the region within historic times. The results of our computer database search indicate that 36 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
Fair-Anselm Plaza
Fairfax, California

Epicenter (Latitude, Longitude)	Historic Richter Magnitude	Year	Distance
37.80, -122.20	6.8	1836	39 km
37.60, -122.40	7.0	1838	45 km
37.70, -122.10	6.8	1868	53 km
38.20, -122.40	6.2	1898	28 km
37.70, -122.50	8.25	1906	32 km

References: Sources: USGS (2009)

3. Probability of Future Earthquakes - The historical records do 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. Surface Conditions

The Fair-Anselm Plaza sits on relatively level ground along the northern bank of San Anselmo Creek. The creek has incised a channel approximately 15 feet deep. The southern side of the building extends over the creek by as much as 30 feet and is supported by three rows of piers. The southernmost row of piers currently lies in the flow line of the channel. Bank erosion has occurred under the building and around the existing piers. The creek has also undermined a tied-back wooden retaining wall at the western end of the building, and the wall itself no longer provides any support to the creek bank.

In the vicinity of the project site, surface conditions along San Anselmo Creek consist of a deeply incised channel with steep to very steep bank inclinations. The northern bank of the creek varies from approximately a 2:1 (horizontal:vertical) inclination at the building's east end to approximately 1:1 at the west end. The southern bank is slightly less inclined, and meanders in the stream channel have produced small gravel bars along the south side of the channel. Flow occurs year-round, though water levels are relatively low in the summer and fall months.

D. Field Exploration

We drilled 4 exploratory borings ranging in depth from approximately 14 to 39 feet at the site on July 16, 2009 using track-mounted hydraulic drilling equipment. The approximate boring locations are shown on Figure 2. Borings 1 and 2 were drilled near the top of the creek bank at the west end of the building, in the vicinity of the existing failed retaining wall. Borings 3 and 4 were drilled at the top of the creek bank at the east end of the building. The soils encountered were logged by our Field Geologist and samples were obtained for laboratory testing. The subsurface exploration program is discussed in more detail in Appendix A. A Soil Classification Chart and Rock Classification Chart are shown on Figures A-1 and A-2, respectively. The boring logs are presented on Figures A-3 through A-8.

Laboratory testing of select samples included determination of moisture content, dry density, unconfined compressive strength, percentage of particles passing a number 200 sieve, and

gradation. Results of laboratory testing are shown on the Boring Logs, Figures A-3 through A-8, with the exception of gradation results, which are shown on Figure A-9.

A previous geotechnical report was prepared in the project vicinity by Neil O. Anderson and Associates (2008). Boring 5 from this report is located near the Fair-Anselm Plaza as shown on Figure 2. The boring log is included for reference on Figure A-10. The laboratory testing program is discussed in more detail in Appendix A.

E. Subsurface Conditions

Our subsurface exploration generally confirms the mapped local geologic conditions. Boring 1 was excavated at the top of the creek bank and adjacent to the west end of the building. Minor concrete debris was encountered in the upper two feet. Fill material consisting of sandy silt with gravel was encountered to a depth of approximately 3.5 feet and was underlain by approximately 33 feet of alluvial soils generally comprised of variable proportions of sands, silts, and clays. These alluvial soils are underlain by relatively hard sandstone bedrock, which was encountered at a depth of 36 feet. Boring 1 was terminated at 37.5 feet due to drilling refusal.

Boring 2 was excavated approximately 20 feet west of Boring 1, near the trash enclosure in the building's west parking lot. Boring 2 encountered approximately 3 feet of fill material similar to that observed in Boring 1. We encountered approximately 10 feet of alluvial soils consistent with those found in Boring 1, and Boring 2 was terminated at a depth of 13.5 feet.

Boring 3 was excavated at the southern edge of the east parking lot approximately 30 feet from the building's eastern end. Boring 3 encountered approximately 4.5 feet of sandy silt and silty sand with gravel, consistent with fill materials observed on the west side of the building. These fill materials were underlain by moderately hard mélange and claystone rock, believed to be a large boulder along the northern edge of the creek. Boring 3 was terminated at 14.0 feet when it was determined that samples and observations would not be representative of overall subsurface conditions.

Boring 4 was excavated approximately eight feet from the top of the creek bank adjacent to the building's eastern side. Boring 4 encountered approximately 4 feet of fill similar to that observed in the other borings. The fill was underlain by approximately 24 feet of alluvial soils consisting largely of silty clay and clayey silt to a depth of approximately 28 feet. Soft to medium stiff clay was encountered between 28 and 39 feet and was underlain by hard sandstone bedrock, encountered at a depth of 39 feet. Boring 4 was terminated at 40.25 feet due to sampler and drilling refusal.

Groundwater was encountered in Borings 1 and 4 at depths of 29 and 30 feet, respectively. However, because we were unable to leave the boreholes open for an extended period of time

and due to the immediate proximity of San Anselmo Creek, it is unlikely that these measurements accurately reflect stabilized groundwater levels. It should be anticipated that groundwater levels will roughly coincide with the water level in the creek, and may be higher during the winter months and following significant rainfall events.

F. Sonic Echo Testing

Sonic echo testing was performed by ABE Engineering on ten foundation piers on July 30, 2009. Sonic echo testing consists of measuring the time taken for a stress wave, or S-wave, to reach the bottom of the pier and return to the top. The S-wave is generated by striking the top of the pier with a small hammer, and an accelerometer attached to the top of the pier records its response. It is assumed that concrete piers have relatively consistent impedance, and that therefore two piers, identical in length, will produce the same travel time for an S-wave generated at the top of the pier to reach the bottom and return to the top.

The results of the sonic echo testing and our subsurface exploration generally indicate that the majority of the foundation piers are embedded in bedrock below any potentially liquefiable soil layers. Based on the sonic echo results and our boring logs, three of the piers appear to be bearing on softer alluvial soil. It should be noted that two of the three piers in question appear to have been installed in connection with the bank vault some time after the main foundation piers were constructed and are not part of the original building support system. ABE Engineering's sonic echo testing report and survey plots are presented as Appendix B while pile locations and identifications are shown on Figure 5.

IV. GEOLOGIC HAZARDS

A. General

The principal geologic hazards which could potentially affect the site are strong seismic shaking and erosion. Other hazards, such as fault rupture, expansive soil, and settlement are not considered significant at the site. Geologic hazards, their impacts and recommended mitigation measures are discussed below.

B. Fault Surface Rupture

Under the Alquist-Priolo Special Studies Zone Act, the California Division of Mines and Geology (CDMG) produced 1:2000 scale maps showing all active faults. The site is not located within an Alquist-Priolo Special Studies Zone and is not near any of the known active faults. The potential for fault surface rupture at the site is low.

Mitigation: No mitigation measures are required.

C. Seismic Shaking

The site will 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. The intensity of earthquake motion will depend on the characteristics of the generating fault, distance to the fault and rupture zone, earthquake magnitude, earthquake duration, and site-specific geologic conditions.

Estimates of peak ground accelerations are based on either deterministic or probabilistic methods. Deterministic methods are typically used for residential, commercial and industrial projects and use empirical relations to provide approximate estimates of median peak ground accelerations. Critical facilities, such as hospitals and schools, typically use probabilistic methods.

Our borings encountered approximately 35 to 40 feet of relatively stiff soils underlain by hard bedrock. Empirical relations developed for stiff soil sites (Boore and Atkinson, 2008) provide approximate estimates of median peak site accelerations. A summary of the active faults that could most significantly affect the planning area, their maximum credible magnitude, closest distance to the project site, and probable peak ground accelerations are summarized in Table B.

TABLE B
ESTIMATED DETERMINISTIC PEAK GROUND ACCELERATION
Fair-Anselm Plaza
Fairfax, California

Fault	Moment Magnitude for Characteristic Earthquake[1]	Closest Estimated Distance[2] (kilometers)	Median Peak Ground Acceleration (g)[3]
San Andreas	7.8	12	0.31
Hayward	6.9	17	0.19
Rodgers Creek	7.0	17	0.20
San Gregorio	7.2	24	0.16
West Napa	6.5	36	0.10

(1) United States Geological Survey, 2008.
(2) Distance estimated using FRISKSP using site coordinates shown on Figure 4
(3) Determined from attenuation relationship by Boore and Atkinson (2008)

The calculated accelerations should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

The potential for strong seismic shaking at the project site is high. The San Andreas Fault and Rodgers Creek Fault present the highest potential for severe ground shaking. The significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Mitigation: *Mitigation measures include designing the new improvements in accordance with the most recent provisions of the California Building Code (2007 CBC). Recommended, site specific seismic coefficients are presented in Section V of this report.*

D. Liquefaction Potential

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena can include ground settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. Mapping provided by USGS (2009) indicates liquefaction susceptibility varies from moderate to very high along the creek channel near the project site, as shown on Figure 6.

Our exploration did encounter thin deposits of loose to medium-dense granular deposits below the groundwater table. Based on our analysis, there are sand and gravel layers within the alluvial soil that are susceptible to liquefaction in the immediate vicinity of the project site. A potentially liquefiable layer exists in Boring 1 at a depth of 20 to 25 feet (elevation 63 to 68 feet MSL). Therefore, we judge the risk of liquefaction occurrence is moderate. If liquefaction were to occur, it could cause localized loss of foundation support within or above the liquefiable layer. However, the existing drilled piers appear to be embedded well below the liquefiable layer

Mitigation: *Mitigation measures include structural evaluation of the existing foundation piers to ensure adequate support for the structure should localized liquefaction occur during a seismic event. Retaining walls should also be supported on deep foundations, such as drilled cast-in-place piers. Foundation recommendations and design criteria are discussed in more detail in Section V of this report.*

E. Seismic 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. Loose granular soils above the water table were not encountered in our borings and were not observed in the creek banks. The existing structure is supported on a drilled pier foundation system. Therefore, we judge the risk of seismic-induced settlement is low.

Mitigation: *No mitigation measures are required.*

F. Erosion

Flow conditions of the San Anselmo Creek must be considered while assessing erosion at the project site. Intense rainfall and increased surface run-off during strong storm events will increase the quantity and velocity of flow, increasing potential for erosion and scour along the creek banks in the vicinity of the existing structure and associated improvements.

Sandy soils on moderate slopes or clayey soils on steep slopes are also susceptible to erosion when exposed to concentrated water runoff. The steep creek banks at the project site are composed of alluvial soils which are susceptible to erosion when subjected to concentrated water runoff, as evidenced by the current alignment of the creek channel beneath the building. Given the project site location, the risk of future erosion is high.

Mitigation: *Implementation of bank stabilization measures is needed to improve stability and mitigate erosion in the vicinity of the existing building The project Civil Engineer should be responsible for evaluation and design of any erosion mitigation measures upstream or downstream from the project site. Specific bank stabilization options are discussed in detail in Section V of this report. Any*

erosion that does occur should be promptly repaired to prevent enlargement. Erosion control measures during and after construction, at a minimum, should conform to the most recent version of the Erosion and Sediment Control Field Manual (California Regional Water Quality Control Board, 2002).

G. Expansive Soil

Clays and silts of moderate to high plasticity, when located near the ground surface, can exhibit expansive characteristics (shrinking and swelling with seasonal drying and wetting cycles) that can be detrimental to lightly-loaded structures and flatwork. Shrink/swell cycles can also result in soil creep along moderate to steep slopes. We did not observe expansive soil conditions during our exploration. Therefore, damage from expansive soils is not considered to be a significant risk at the site.

Mitigation: No mitigation measures are required.

H. Flooding

The project site is located along San Anselmo Creek and is mapped near a FEMA 100-year flood zone. Existing and planned improvements may experience future flooding events. Risks associated with flooding are low to moderate. Cleanup of debris or damage to the structures from floating debris may be necessary. The project may alter the hydraulic flow in the creek and could change the upstream or downstream flooding potential. Loading of the new structures due to flooding events should be considered in the structural design of the improvements.

Mitigation: Structures should be designed to withstand loading associated with peak flow and velocities. The project Civil Engineer responsible for site drainage should also evaluate the planned projects' impact on flooding potential and provide appropriate mitigation.

I. Settlement

New surface loads can cause consolidation of soft clays or compression of loose soils. While loose soils were not encountered during our exploration, isolated lenses of soft clays were observed at the west end of the existing building, and thicker deposits of soft clays were encountered at the east end. These soft clays were encountered at depths of over 20 feet, and were found to be approximately 10 feet thick at the building's west end. Some of the existing piers are embedded in the soft clay layer. Therefore, the risk for significant settlement at the site during a seismic event is considered moderate.

Mitigation: Mitigation measures include structural evaluation of the existing piers to ensure adequate support for the structure should settlement occur during a seismic event.

Foundation recommendations for retaining walls are presented in Section V of this report.

J. Lurching, Lateral Spreading, and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. Ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep slopes or channel banks. The channel banks near the project site have the potential for lurching and ground cracking during strong seismic shaking. Provided the new and existing structures are stable during seismic events, the potential for significant damage is low. Some clean-up and removal of debris from the creek banks should be expected.

Lateral spreading involves large blocks of earth sliding towards a free face on an inclined, continuous, liquefied soil layer. Continuous liquefiable soil layers are not present at the project site. Thus the potential for damage from lateral spreading is considered low.

Mitigation: No mitigation measures are required.

K. Seiche and Tsunami

Seiche and tsunamis are short duration, earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche would be dependent upon ground motions and fault offset from nearby active faults. The project is located approximately 88 feet above sea level and is located approximately 5 miles west of San Pablo Bay. Therefore, the risk of seiche or tsunami at the site is remote.

Mitigation: No mitigation measures are required.

L. Slope Instability

Weak soils and bedrock on moderate to steep slopes can move downslope due to gravity. Slope instability is often initiated or accelerated from soil saturation and groundwater pressure.
Our geologic site inspections did not reveal signs of recent instability of the steep creek banks. Considering the steep topography and relatively shallow alluvial soils along the creek, some shallow sloughing or raveling of the creek banks may occur. The risk of “global” slope failure that would significantly impact the planned improvements is low. Therefore, we judge the risk of slope instability at the site is low. The addition of the new retaining wall and rip-rap placement planned along the banks beneath the existing building will improve slope stability in the vicinity of the project site.

Mitigation: Implementation of bank stabilization measures as indicated by conceptual plans will increase stability in the vicinity of the project site. Specific options for bank and slope

stabilization is discussed in detail in Section V of this report. No other mitigation measures are required.

V. CONCLUSIONS AND RECOMMENDATIONS

A. Conclusions

Based on the results of our investigation and experience with similar sites and projects, we conclude that the planned seismic and/or erosion-control improvements along the creek are feasible from a geotechnical standpoint. The primary geotechnical concerns related to the project are evaluation of the foundation system for competence during a seismic event and reducing erosion along the northern creek bank. Geotechnical recommendations and criteria for use in project planning and design are provided in the following sections.

B. Existing Improvements

Existing structures along the north bank of the creek are limited to the main building and its associated foundation improvements (concrete piers and exposed steel beams) as well as a failing retaining wall immediately to the west. The retaining wall consists of steel I-beam soldier piles with timber lagging and tiebacks. Most of the timber lagging has fallen into the creek and the tiebacks are exposed along the upper creek bank in many places. A variety of utility conduits serving the building traverse the creek bank as well. Asphalt-paved parking lots exist adjacent to the top of the creek bank at either end of the building.

C. Bank Stabilization

The creek bank beneath the existing building should be stabilized to prevent further erosion and potential undermining of the building's foundation system. Slopes immediately east and west of the building should also be stabilized to prevent encroachment of the creek channel upon the building foundations, utility services, parking lots, and other associated improvements. Slope stability options are variable and are chiefly dependent upon slope inclination, design life, and cost.

Beneath the building, stabilization may be accomplished by placement of stabilization fabric and rip-rap armoring as schematically shown on Figure 7. Alternatively, structural stabilization such as retaining walls or shotcrete could be utilized. The preferred method will be partially dependent upon the permitting requirements. It has been our experience that it is more difficult to obtain permits for placement of foreign materials (such as shotcrete) in a stream channel. Therefore, we recommend a rip-rap buttress be constructed beneath the building.

Where the creek banks are more steeply inclined, as at the east and west ends of the building, options for mitigation of slope instability are limited to shallowing of the slopes or construction of retaining walls. Should the slopes be laid back as a means of increasing bank stability, they should be cut to a maximum inclination of 2:1 (horizontal:vertical), and would likely require removal of some existing parking stalls. Based on the potential for liquefiable soils and erosion

along the creek banks, retaining walls should be constructed on deep foundations such as drilled, cast-in-place piers, helical piers, or mini-piles. Because the bank is currently over-steepened, care should be taken during construction to avoid overexcavating or otherwise undermining the bank.

D. Seismic Design

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 (CBC). The intent of the CBC is protection of life safety. Adherence to the seismic provisions of the current code will reduce structural damage during strong ground shaking, but may not eliminate damage. 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 and the San Gregorio Fault, we recommend the CBC 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 5.0.9, using the latitude and longitude shown on Figure 4. Per the California Building Code (2007), soil profiles containing liquefiable soils are classified as Site Class S_F. However, since thin horizons of liquefiable soils were encountered well above the bottoms of the shallowest foundation piers, liquefaction in an earthquake event should have negligible effects on the structure. Therefore, we have used Site Class S_D for Site-Specific values shown below, as the foundation piers bear on stiff soil or rock.

TABLE C
SEISMIC DESIGN FACTORS
Fair-Anselm Plaza
Fairfax, California

2007 California Building Code

Factor Name	Coefficient	CBC Table[1]	Site Specific Value
Site Class[2]	$S_{A,B,C,D,E,\text{ or }F}$	1613.5.2	S_D[2]
Spectral Acc. (short)	S_s	1613.5.1	1.50 g
Spectral Acc. (1-sec)	S_1	1613.5.1	0.65 g
Site Coefficient[2]	F_a	1613.5.3 (1)	1.0
Site Coefficient	F_v	1613.5.3 (2)	1.5

(1) For facilities regulated by the Division of the State Architect – Structural Safety (DSA-SS), the Office of Statewide Health Planning and Development (OSHPD), or other agencies (e.g. schools, hospitals, etc.) use the "A" equations and tables in lieu of the equations and tables noted above. "Site specific" values in the table apply to all structures.

(2) Soil Profile Type S_D Description: Stiff Soil, Shear Wave Velocity between 600 (180) and 1200 (365) feet per second (m/s), Standard Penetration Test N value between 15 and 50, and Undrained Shear Strength between 1000 (50) and 2000 (100) psf (kPa).

E. Foundation Design Criteria

The alluvial soils encountered during our exploration generally consist of stiff silts and loose to medium stiff sands containing variable amounts of clay, sand, and gravel. The alluvial soil layer is approximately 40 feet thick and bedrock was encountered between 36 and 40 feet below the existing parking lot elevation. The underlying Franciscan bedrock is slightly to moderately weathered, strong, and relatively hard.

We understand that seismic evaluation and potential retrofitting of the existing structure is planned. Based on ABE Engineering's Sonic Echo Testing of selected foundation piers, it appears that the existing piers bear on firm alluvial soils or bedrock below potentially liquefiable layers. Following a structural evaluation, these piers may be considered for continued support during a seismic event. For the structural evaluation of the vertical capacity at existing piers, we recommend using the criteria shown in Table D. Load tests should be performed on individual piers to confirm the existing foundation can provide the required vertical capacity. If required, we can provide supplemental analysis for the lateral capacity of the foundation piers or piles. For foundation retrofit or new piers, we recommend one of the deep foundation systems discussed below.

Drilled Piers – Piers may be drilled through the loose saturated alluvial soils and "socketed" into the underlying weathered sandstone bedrock. However, due to the presence of loose and saturated sands, the drilled piers may need to be either cased or excavated utilizing drilling mud to prevent hole collapse. Design criteria for existing and new drilled piers are presented below in Table D.

TABLE D
DRILLED PIER FOUNDATION DESIGN CRITERIA
Fair-Anselm Plaza
Fairfax, California

Minimum diameter:		18 inches
Minimum penetration into firm bedrock:		5 feet
	Static[1]	Seismic[1]
Skin Friction (dead plus live loads)[2,3,4]:		
0-20 Feet Below Parking Lot (85-105msl)	750 psf	1000 psf
20-30 Feet Below Parking Lot (75-85msl)	1000 psf	0 psf
30-40 Feet Below Parking Lot (65-75msl)	750 psf	1000 psf
Firm Franciscan Rock (Below 65msl)	3000 psf	4000 psf
End Bearing	0 psf	10,000 psf
Lateral passive resistance [4,5,6]:		
0-20 Feet Below Parking Lot (85-105msl)	300 pcf	500 pcf
20-30 Feet Below Parking Lot (75-85msl)	350 pcf	100 pcf
30-40 Feet Below Parking Lot (65-75msl)	300 pcf	500 pcf
Firm Franciscan Rock (Below 65msl)	600 pcf	1000 pcf

Notes:
(1) Static criteria are long term design loads not including seismic loading. Seismic criteria are for short term seismic loading.
(2) Uniform pressure distribution.
(3) Uplift resistance is equal to 80% the downward skin friction.
(4) Neglect the upper 3 feet
(5) Apply values over effective width of two pier diameters.
(6) Equivalent fluid pressure. Maximum passive not to exceed 10 times the design criteria. Include overburden for starting pressure. Can provide more detailed lateral pile analysis upon request.

Driven Mini Piles – Driven mini-piles may be utilized to support the proposed retaining wall or supplement the existing building piers. As with drilled piers, the mini-piles should be driven through the loose saturated alluvial soils to bear on the weathered sandstone bedrock. Based on our experience with similar projects and site conditions, full size

driven piles may not be feasible or cost effective for a project of this size. However, 4 to 6 inch mini-piles may be practical. We can provide vertical and lateral pile capacities for a given pile type if requested, though we anticipate 20 to 40 kips per mini-pile is a reasonable design estimate.

Helical Anchors – Helical anchors may be utilized to support the proposed retaining wall and supplement the existing building piers. The helical anchors should be installed through the underlying loose saturated alluvial soils and bear on the weathered sandstone bedrock. Depending on the number and size of the helices on each anchor, vertical design capacities up to 20 to 30 kips per anchor can be reasonably achieved. Actual capacities are ultimately dependent on the amount of torque applied during installation.

F. Retaining Walls

A new retaining wall is recommended to support the north bank adjacent to the west end of the building, just south of the existing trash enclosure. The retaining wall could be used in combination with a rip-rap buttress to reduce the overall wall height. Specific wall types may include soldier-pile and timber-lagging, concrete blocks, or shotcrete and soil nails. Pier embedment into the underlying bedrock layer will provide the necessary vertical and lateral support for the new wall.

Retaining walls that can deflect at the top can be designed using the unrestrained criteria shown in Table E. Walls that are tied-back and not allowed to deflect are 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 or through soil nails and/or tiebacks. In addition to the soil loads, the retaining walls should be designed to resist temporary seismic loads.

TABLE E
RETAINING WALL DESIGN CRITERIA
Fair-Anselm Plaza
Fairfax, California

Foundation		
Refer to the foundation design criteria in Section E		
Lateral Earth Pressure[1]	Unrestrained	Restrained[2]
Level Ground	40 pcf	25*H psf
2:1 Slope	60 pcf	40*H psf
Seismic Surcharge[2]	10*H psf	
Tiebacks[3]		
Minimum Unbonded Length	½ wall height	
Minimum Diameter	4 inches	
Skin Friction	750 psf	

Notes:
(1) Interpolate earth pressures for intermediate slopes.
(2) Rectangular uniform pressure distribution (H = height of wall).
(3) Factor of safety should be reduced to 1.0.

Retaining walls should be drained and backfilled in accordance with details provided on Figure 8. Wall drainage should be collected in a 4-inch perforated, Schedule 40 PVC drain line placed at the base of the wall as shown on Figure 8. The upper 12-inches of the backdrain should consist of compacted on-site soils. Seepage collected in the drain line should be conveyed in a closed pipe to a suitable discharge outlet or through weep holes in the wall. To maintain the wall drainage system, a clean-out must be provided for the perforated pipe at the upstream end. Sweep fittings should be used at all major changes in direction.

G. Site Grading

1. Anticipated Excavation Conditions. Construction of foundation reinforcements and the new retaining wall will require removal of debris associated with the old, failed retaining wall. The observed surface and subsurface conditions suggest the presence of potentially large debris and rubble in the vicinity of the old retaining wall. Potholing and debris removal may be required prior to constructing foundations for the retaining wall. Removal of such debris may result in deeper excavations than those shown in the conceptual plans. Any overexcavation should be backfilled to the plan elevations as described below. Equipment capable of breaking large concrete debris, such as a hoe-ram or jackhammers, may be required during excavation of the old wall and associated rubble.

2. Subgrade Preparation. Upon completion of excavations and prior to placement of structural fill, the exposed ground surface should be scarified to a depth of 8-inches, moisture conditioned to near the optimum moisture content, and re-compacted to a minimum of 90 percent relative compaction (ASTM D-1557). The finished subgrade surface should be uniform, firm and unyielding under loaded construction equipment.

3. Fill Materials. Some of the on-site alluvial soil does appear suitable for use as compacted fill. Fill materials should consist of non-expansive materials spread uniformly and continuously prior to compaction. Structural fill shall be free of organic matter, have a Liquid Limit of less than 40, a Plasticity Index of less than 15, and a minimum R-value of 20. Oversized material (greater than 4 inches) shall be removed from the fill soils.

4. Compaction. All on-site and imported fill should be uniformly conditioned to near the optimum moisture content. Properly moisture conditioned and cured materials should subsequently be placed in loose horizontal lifts of 8 inches thick or less, and uniformly compacted to a minimum of 92 percent relative compaction to produce a firm, non-yielding surface.

5. Temporary Cut Slope Construction. Temporary (steeper) cut slopes may be required during construction of foundation reinforcements and the new retaining wall. Temporary cut slopes into the alluvial soils (creek bed materials) should conform to OSHA requirements for Type "C" soils. Temporary cut slopes into firm cohesive (clayey) soil or weak rock (creek banks above water level) should conform to OSHA requirements for Type "B" soils. Geologic inspection will be required during excavation to verify that the anticipated soil types are consistent with the conditions encountered. Because the Contractor is responsible for site safety and operations, the grading contractor will have the ultimate responsibility for the design and performance of temporary cut slopes.

Performance of the temporary cut slopes will be heavily influenced by the length of time the cut is unsupported, cut height, inclination, seepage, surface runoff over the cut face, soil materials and other factors. The steeper (temporary) cut slopes may exhibit some sloughing, especially during wet weather conditions, and cleanup of debris at the base of the cut may be required.

VI. SUPPLEMENTAL GEOTECHNICAL SERVICES

During project design, we can provide supplemental consultation, design and analysis for the preferred foundation system. 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/or test; temporary excavations, site grading, foundations, retaining wall drainage and backfill to confirm that the soils encountered during construction are consistent with the design criteria and recommend any needed field modification to suit the actual site conditions.

LIST OF REFERENCES

American Society for Testing and Materials, "2001 Annual book of ASTM Standards, Section 4, Construction, Volume 4.08, Soil and Rock; Dimension Stone; Geosynthetics," ASTM, Philadelphia, 1997.

Association of Bay Area Governments (ABAG), "Bay Area Liquefaction Susceptibility Maps", http://gis.abag.ca.gov/website/liq/viewer.htm, Accessed August 19, 2009.

Blake, T.F., FRISKSP (Version 4.00) A Computer Program for the Probabilistic Estimation of Peak Acceleration and Uniform Hazard Spectra Using 3-D Faults as Earthquake Sources, 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, 2007 Edition, California Building Standards Commission/International Code Council, Sacramento, California.

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 Division of Mines and Geology, Special Publication 42, "Alquist-Priolo Special Studies Zone Act," 1972 (Revised 1988).

Neil O. Anderson and Associates, "Preliminary Geotechnical Investigation, Fair-Anselm Property, Center Boulevard, Fairfax, California", Project Number WGG0051, January 2008.

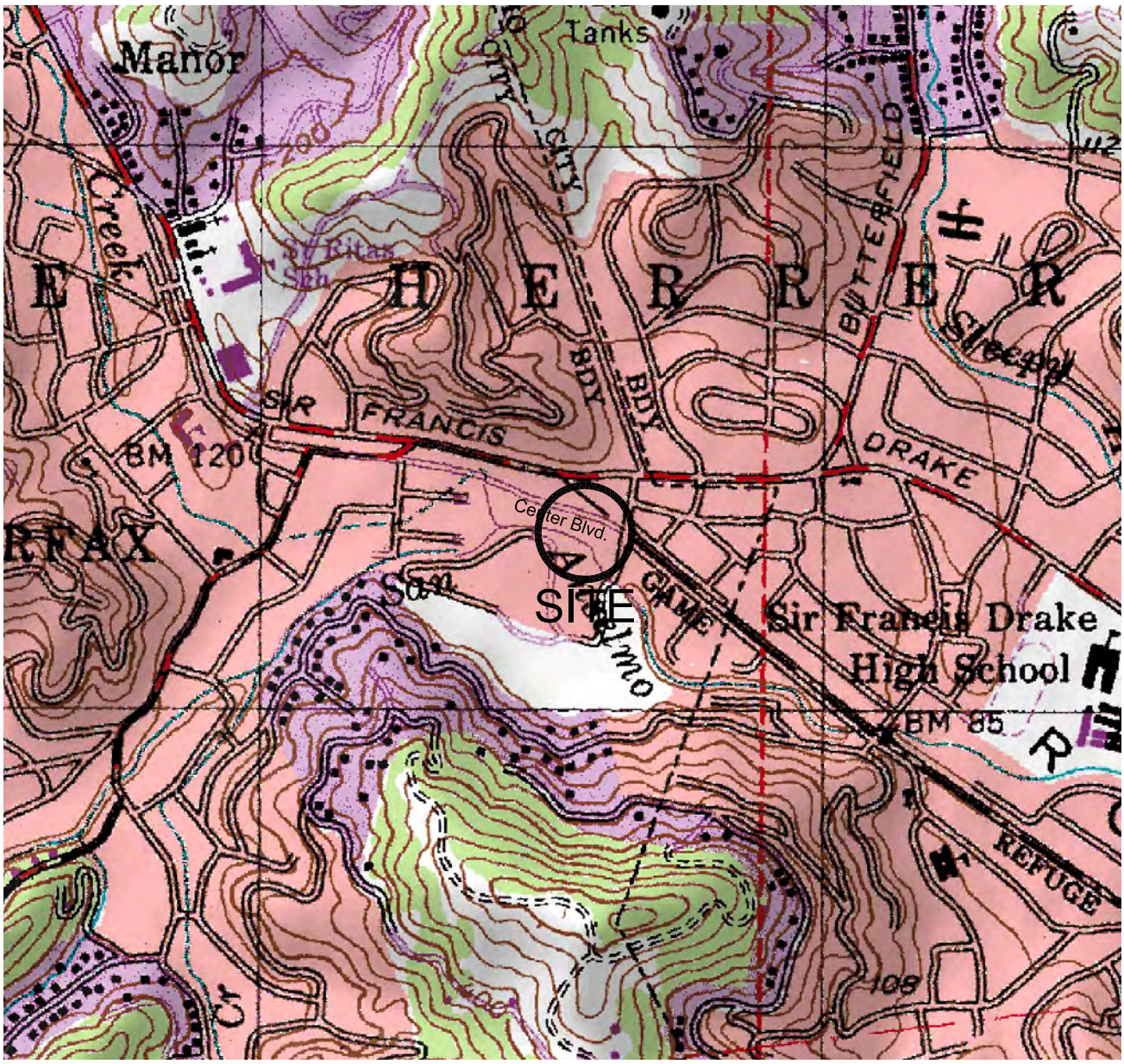
Rice, S.J., Smith, T.C., and Strand, R.G., California Department of Conservation, Division of Mines and Geology, "Geology for Planning: Central and Southeast Marin County, California," Open-File Report 76-2 S.F. Plate 1E, 1976.

Southern California Earthquake Center, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California," University of Southern California, March 1999.

U.S. Geological Survey, "Geologic Map and Map Database of parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California,", Misc. Field Studies MF-2337, 2000.

U.S. Geological Survey, "(Liquefaction) Susceptibility Map of the San Francisco Bay Area", http://geomaps.wr.usgs.gov/sfgeo/liquefaction/susceptibility.html Accessed July 21, 2009.

U.S. Geological Survey, "The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2)," The 2007 Working Group on California Earthquake Probabilities, USGS Open File Report 2007-1437, CGS Special Report 203, SCEC Contribution #1138, 2008.



SITE LOCATION

SCALE

0 1000 2000 4000 FEET

0 500 1000 METERS

NORTH

REFERENCE: DeLorme 3D TopoQuads, 1999

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		
		Fair-Anselm Plaza Fairfax, California	Drawn JBG Checked	1 FIGURE
A CALIFORNIA CORPORATION, © 2009, ALL RIGHTS RESERVED FILE: 1598.01 SLM.dwg		Project No. 1598.01 Date: 07/12/2009		

REFERENCE: Google Earth 2008, Neil O. Anderson and Associates 2008

Test Boring by Miller Pacific Engineering Group, 2009

Test Boring by Neil O. Anderson and Associates, 2007

APPROXIMATE SCALE

0 25 50 100FEET

NORTH

Miller Pacific
ENGINEERING GROUP

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED
FILE: 1598.01SP.dwg

1333 N. McDowell Blvd.
Suite C
Petaluma, CA 94947
T 707 / 765-6140
F 707 / 765-6222
www.millerpac.com

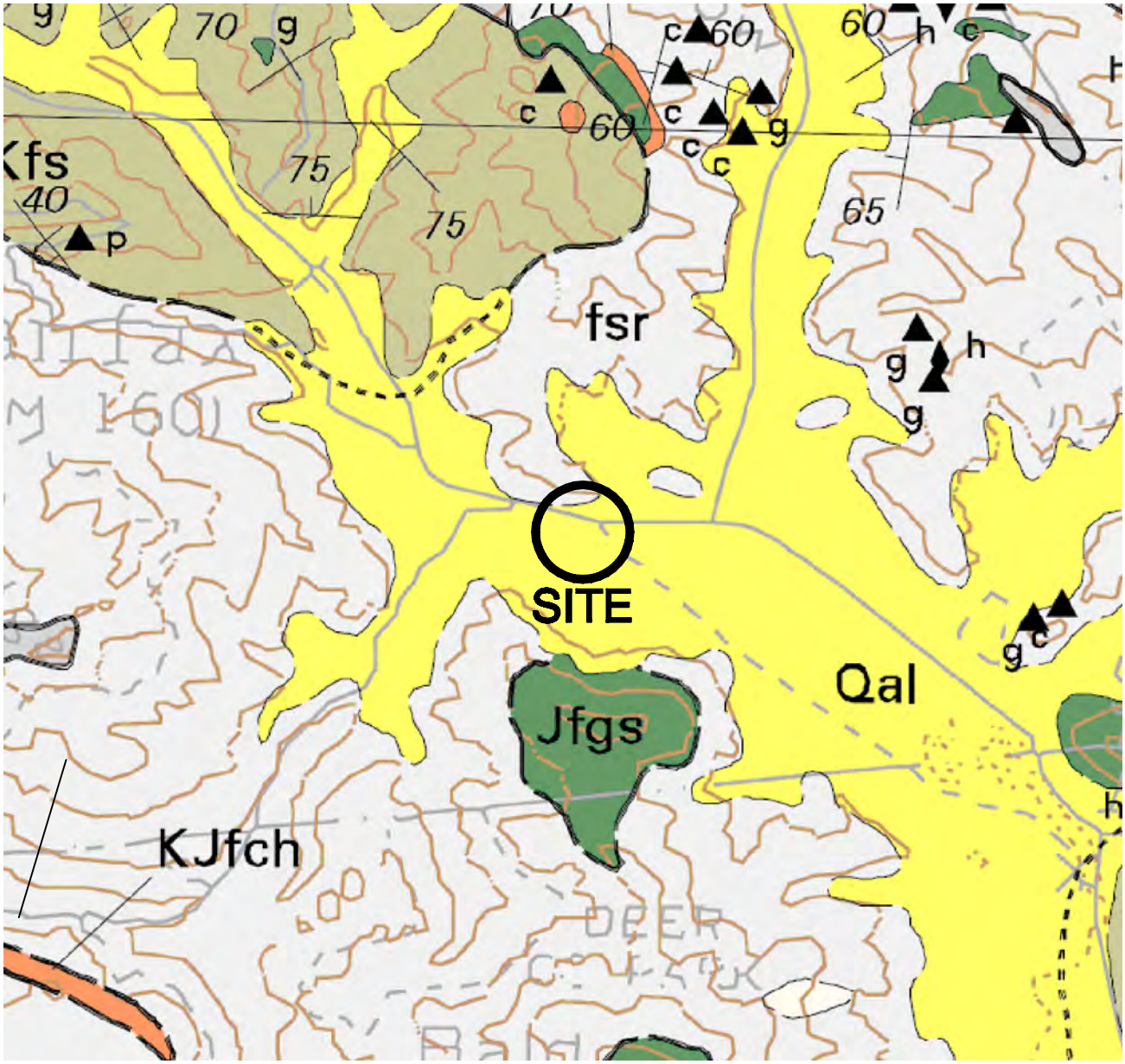
SITE PLAN

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 9/14/09

Drawn MFJ
Checked

2
FIGURE



Reference: U.S. Geological Survey, "Geologic Map and Map Database of Parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California," Version 1, 2000,

SCALE

0 1000 2000 4000 FEET

NORTH

Explanation

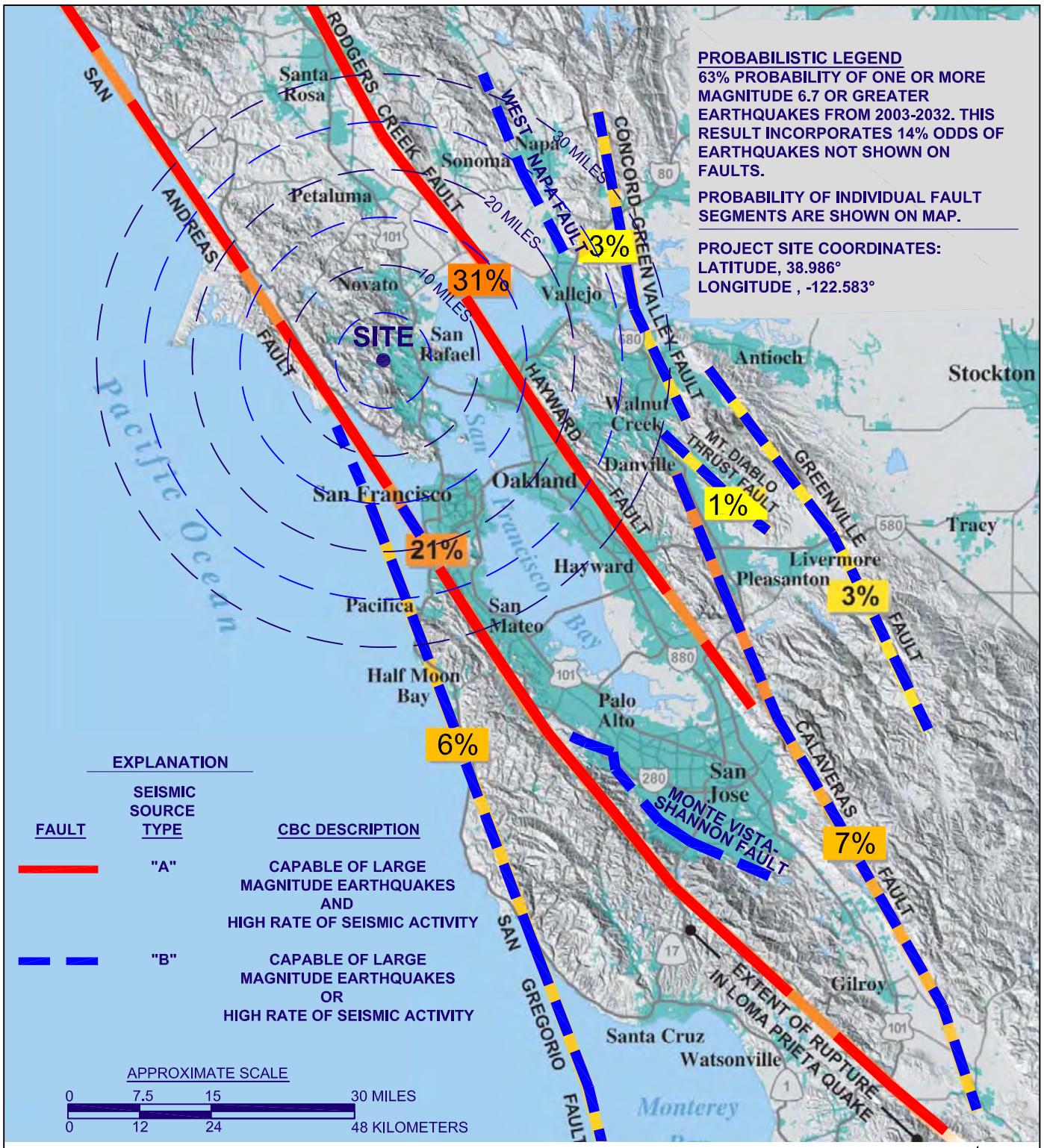
Qal: Alluvium - sand, gravel, silt and clay; loose to soft and friable

Kfs: Cretaceous Sandstone - sandstone and shale with minor conglomerate

Jfgs: Jurassic Greenstone - outcrops as pillow lavas and intrussive basalt

fsr: Franciscan Complex Melange - tectonic mixture of variably sheared shale and sandstone containing hard tectonic inclusions

Miller Pacific ENGINEERING GROUP	504 Redwood Drive Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	GEOLOGIC MAP		
		Fair-Anselm Plaza Fairfax, California	Designed N/A Drawn DFS Checked	3 FIGURE
A CALIFORNIA CORPORATION, © 2009, ALL RIGHTS RESERVED FILE: 1598.01GM		Project No. 1598.01 Date: 8/17/09		



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.

Miller Pacific
ENGINEERING GROUP

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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01FM.dwg

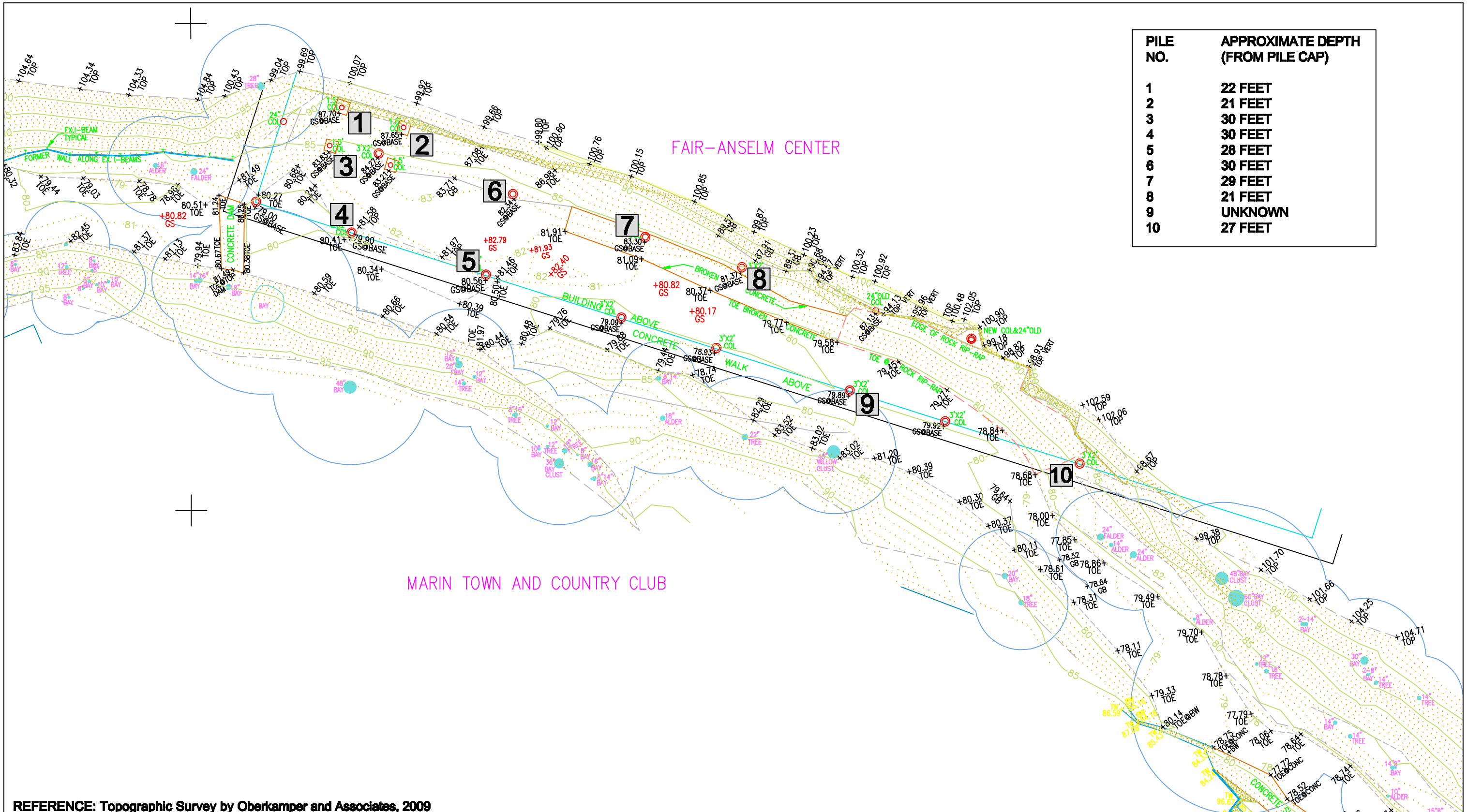
ACTIVE FAULT MAP

Fair-Anselm Plaza
Farifax, California

Project No. 1598.01 Date: 7/21/09

Drawn MFJ
Checked

4
FIGURE



PILE NO.	APPROXIMATE DEPTH (FROM PILE CAP)
1	22 FEET
2	21 FEET
3	30 FEET
4	30 FEET
5	28 FEET
6	30 FEET
7	29 FEET
8	21 FEET
9	UNKNOWN
10	27 FEET

REFERENCE: Topographic Survey by Oberkamper and Associates, 2009

Existing Foundation Piers

3 ABE Engineering Pier ID Number; See Appendix B

NORTH

SCALE

0 15 30 60 FEET

Miller Pacific
ENGINEERING GROUP

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED
FILE: 1598.01TOPO

1333 N. McDowell Blvd.
Suite C
Petaluma, CA 94947
T 707 / 765-6140
F 707 / 765-6222
www.millerpac.com

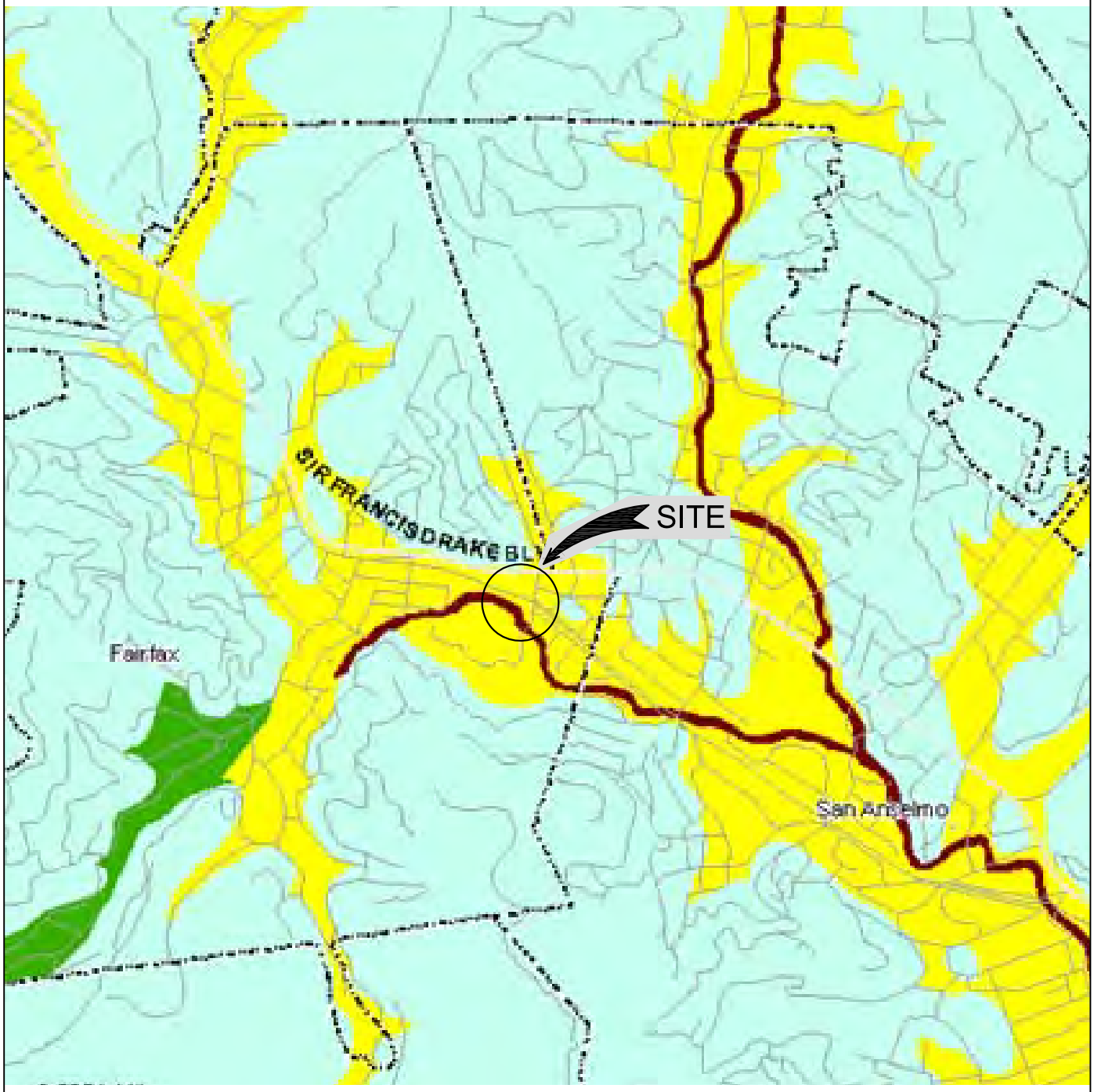
TOPOGRAPHIC SURVEY

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 9/17/09

Drawn MFJ
Checked

5
FIGURE



LIQUEFACTION SUSCEPTIBILITY
- VERY HIGH
- HIGH
- MODERATE
- LOW

NOT TO SCALE

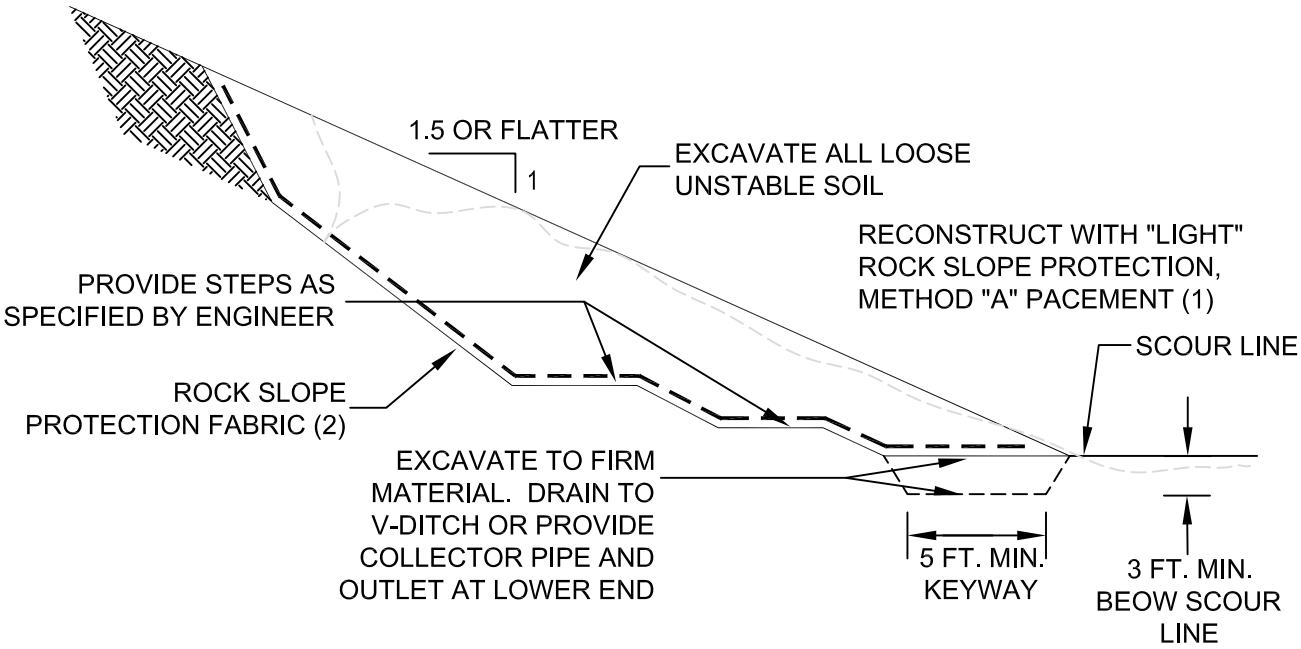
NORTH

REFERENCE: Association of Bay Area Governments, http://www.abag.ca.gov/?go=liq, accessed August 19, 2009

Miller Pacific ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	LIQUEFACTION SUSCEPTIBILITY MAP	Drawn MFJ Checked	6 FIGURE
		Fair-Anselm Plaza Fairfax, California		
		Project No. 1598.01 Date: 8/19/09		

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01LIQ



RIP-RAP BUTTRESS DETAIL

(NO SCALE)

NOTES:

(1.) ROCK SLOPE PROTECTION PER SECTION 72 OF CALTRANS STANDARD SPECIFICATIONS (1997).

(2.) TYPE A ROCK SLOPE PROTECTION FABRIC PER CALTRANS SECTION 88 OF CALTRANS STANDARD SPECIFICATION (1997).

(3.) SCOUR LINE TO BE DETERMINED BY CIVIL ENGINEER.

Miller Pacific
ENGINEERING GROUP

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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01RIPRAP.dwg

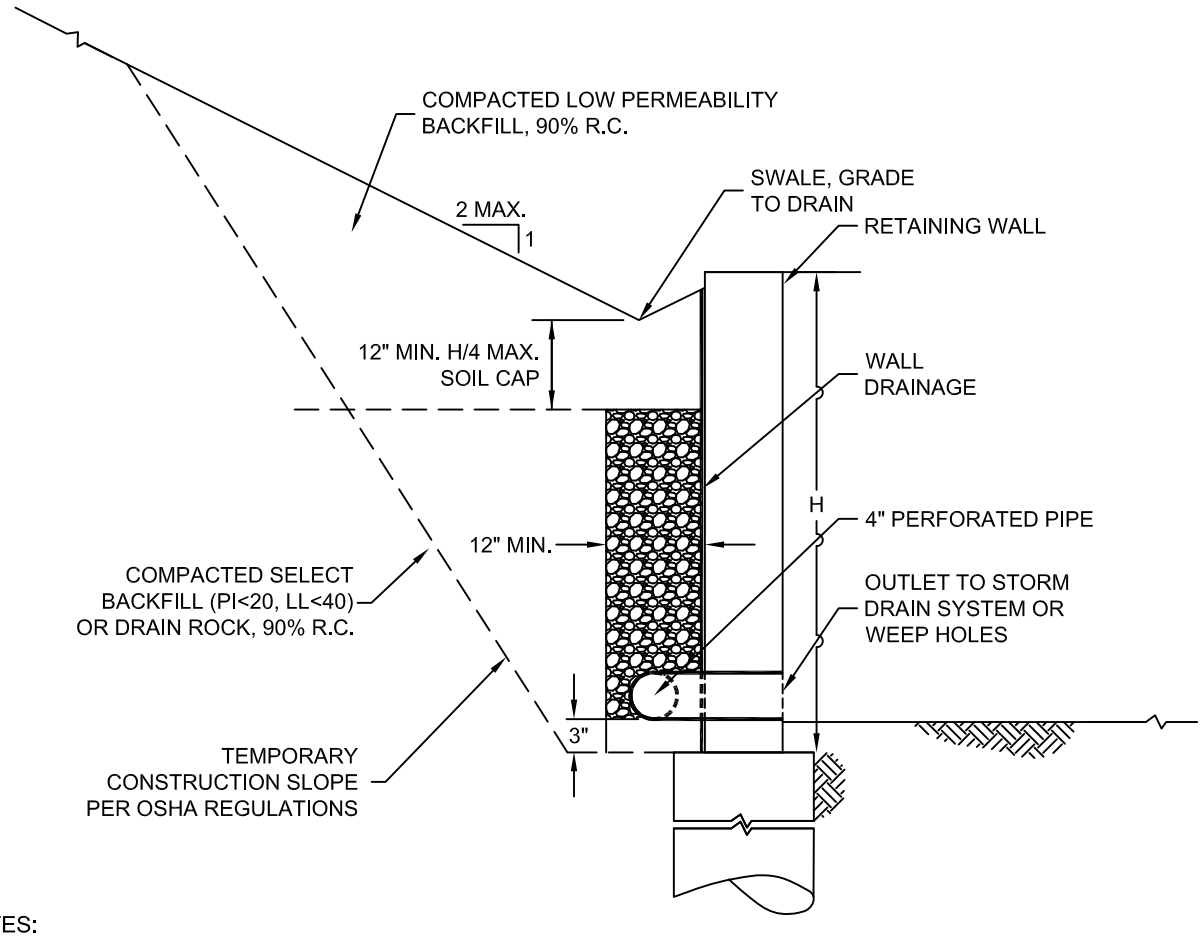
CONCEPTUAL RIP-RAP BUTTRESS

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 9/21/09

Designed SAS
Drawn MFJ
Checked

7
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, a pre-fabricatd drainage panels (Miradrain G100N or equivalent) installed per the manufactures recommendations, may be used in lieu of drain rock and fabric.

2. 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 sloped at 1% to a gravity outlet. Alternatively, drainage can be outlet through 3" diameter weep holes spaced approximately 20' apart.

3. 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.

4. 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.

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

6. All work and materials shall conform with Section 68, of the latest edition of the State of California Standard Specifications (Caltrans).

Miller Pacific ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SCHEMATIC RETAINING WALL DETAIL Fair-Anselm Plaza Fairfax, California Project No. 1598.01 Date: 9/21/09	Designed SAS Drawn MFJ Checked	8 FIGURE

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED
FILE: 1598.01WALL.dwg

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

1.0 Subsurface Exploration

We explored subsurface conditions at the site by drilling 4 test borings on July 16th, 2009 at the locations shown on Figure 2. Test borings were drilled to depths between 14 and 39 feet using 6-inch solid augers with hydraulic track-mounted equipment.

The soils encountered were logged and identified by our Engineer 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-8. Additionally, a boring log by Neil O. Anderson and Associates is provided for reference as Figure A-10.

We obtained relatively “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;
- Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis, ASTM D 6913

The moisture content, dry density, and compressive strength test results are shown on the exploratory boring logs, Figures A-3 through A-8. The gradation test results are shown on Figure A-9.

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

Miller Pacific
ENGINEERING GROUP

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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01BL.dwg

SOIL CLASSIFICATION CHART

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 7/21/09

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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01.dwg

ROCK CLASSIFICATION CHART

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 7/21/09

Drawn MFJ
Checked

A-2
FIGURE

BORING 1

EQUIPMENT: Simco 4000 with 6" Solid Flight Augers

DATE: 7/16/09

ELEVATION: 88 Feet +/-*

*REFERENCE: Google Earth 2009

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)	
						0	0			SANDY SILT WITH GRAVEL (ML) Medium brown, moist, very stiff, ~30% fine to medium sand, ~15% fine to medium angular gravel, occasional large sandstone cobbles [FILL]
			60	12.2	114					
						1				
							5			SANDY SILT (ML) Medium brown, moist, stiff, ~30-40% very fine to fine sand, clean medium dense sand seam from 5.75 feet to 6.25 feet [ALLUVIUM]
			16	9.6	109					
						2				
						3	10			SILT WITH SAND (ML) Medium brown, moist, stiff, ~10-15% very fine sand [ALLUVIUM]
		900 UC	21	17.5	103					
										CLAYEY SILT (ML) Dark brown, moist, medium stiff to stiff, ~20-30% medium-plasticity clay [ALLUVIUM]
						4				
							15			
						5				
										SILTY CLAY (CL) Dark brown, moist, soft to medium stiff, medium plasticity, ~30-40% silt [ALLUVIUM]
						6	20			
										SAND WITH CLAY AND GRAVEL (SW)

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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01BL.dwg

BORING 1

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 7/21/09

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 meters / feet	SAMPLE	SYMBOL (3)	
	12.4% P200	250 UC	14	12.4	112	20			SAND WITH GRAVEL (SW) Medium gray to brown, moist to wet, loose, very fine to medium-grained, ~20% fine to medium gravel [ALLUVIUM]
						7			
						25			CLAYEY SAND (SW-SC) Medium brown, moist, medium dense, fine to coarse-grained, well-rounded, ~40% low-plasticity clay [ALLUVIUM]
						8			
									SILTY CLAY (CL) Medium brown, moist, very stiff, low-plasticity, ~30% silt [ALLUVIUM]
						9			
		2000 UC	28	19.5	112	30			Groundwater encountered at 29.0 feet during drilling Grades to ~50% each clay and silt Stiffer drilling noted at 32.5 feet
						10			
						35			
						11			
									SANDSTONE Dark gray to medium brown, closely fractured, strong, hard, slightly to moderately weathered [BEDROCK]
						12			Bottom of boring at 37.6 feet. Groundwater measured at 30.0 feet 1 hour after drilling.
						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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01BL.dwg

BORING LOG

Fair-Anselm Plaza
Fairfax, California

Drawn MFJ
Checked

Project No. 1598.01 Date: 7/21/09

A-4
FIGURE

BORING 2

EQUIPMENT: Simco 4000 with 6" Solid Flight Augers

DATE: 7/16/09

ELEVATION: 89 Feet+/-*

*REFERENCE: Google Earth 2009

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)	
						0	0			3" AC 4" AB
		800 UC	23	18.6	103					SANDY CLAY WITH GRAVEL (CL) Medium brown, stiff, moist, low plasticity, ~30% very fine to medium sand, ~15% fine to medium gravel [FILL]
			13	21.0	104	1				SANDY CLAY WITH GRAVEL (CL) Medium brown, medium stiff to stiff, moist, low plasticity, ~30% very fine to medium sand, ~15% fine to medium gravel occasional sandstone fragments [ALLUVIUM]
							5			
		1000 UC	16	21.2	103	2				
	38.6% P200 6.9% P200	900 UC		23.8	102					SILTY GRAVEL (GW) Medium to dark brown to olive gray, moist, medium dense [ALLUVIUM]
			41	6.1	125	3	10			
		1300 UC	19	23.1	104	4				SANDY GRAVEL (GW) Medium gray, moist, dense, ~35-40% fine to coarse sand, ~60% fine to coarse gravel [ALLUVIUM]
										SANDY SILT (ML) Medium brown, moist, stiff, ~40% fine sand [ALLUVIUM]
							15			
						5				Bottom of boring at 13.5 feet. No groundwater encountered during drilling.
						6	20			

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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01BL.dwg

BORING 2

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 7/21/09

Drawn MFJ
Checked

A-5
FIGURE

BORING 3

EQUIPMENT: Simco 4000 with 6" Solid Flight Augers

DATE: 7/16/09

ELEVATION: 85 Feet+/-*

*REFERENCE: Google Earth 2009

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)	
						0	0			2" AC 4" AB
			29	12.2	104					SANDY SILT (ML) Light brown, moist, very stiff, medium plasticity, ~40% very fine to medium sand [FILL]
						1				SILTY SAND WITH GRAVEL (SW-SM) Light brown, moist, medium dense to dense, ~20% silt, ~20% fine to medium gravel [FILL]
			91	13.5	114		5			MELANGE Light brown with orange mottling, intensely fractured to crushed, moderately strong, moderately hard, moderately to highly weathered, ~50% each sandstone and claystone [ALLUVIUM/BOULDER]
						2				
			61	16.6	111					
						3	10			
			90	19.6	110	4				CLAYSTONE LIght gray, closely fractured, massive, weak, low hardness, slightly to moderately weatehred [ALLUVIUM/BOULDER]
							15			
						5				Bottom of boring at 14.5 feet. No groundwater encountered during drilling.
						6	20			

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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01BL.dwg

BORING 3

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 7/21/09

Drawn MFJ
Checked

A-6
FIGURE

BORING 4

EQUIPMENT: Simco 4000 with 6" Solid Flight Augers

DATE: 7/16/09

ELEVATION: 86 Feet+/-*

*REFERENCE: Google Earth 2009

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)	
						0	0			3" AC 4" AB
			23	13.8	104	1				SILTY SAND WITH GRAVEL (SW-SM) LIght brown, moist, medium dense, ~15% silt, ~15% fine to medium gravel [FILL]
		2500 UC	37	22.2	105	2	5			CLAYEY SILT (ML) Medium brown, moist, very stiff, medium plasticity, ~30-40% clay [ALLUVIUM] Grades stiff Grades to ~15% clay, minor very fine sand, occasional organics
		1200 UC	46	18.7	105	3 4	10			
	16.8% P200		70	13.8	121	5	15			GRAVELLY SAND WITH SILT (SW) Medium brown with orange mottling, moist, dense , ~20% fine to coarse rounded gravels [ALLUVIUM] Stiffer drilling noted at 17.0-18.0 feet.
						6	20			SILTY CLAY (CL)

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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01BL.dwg

BORING 4

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 7/21/09

Drawn MFJ
Checked

A-7
FIGURE

BORING 4 (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)	
	1200 UC	93	43	17.1	101	20			SILTY CLAY (CL) Medium brown, moist, stiff to very stiff, low to medium plasticity, ~50% each silt and clay [ALLUVIUM]
						7			
						25			
						8			
						9			CLAY (CL) Dark brown, moist, soft to medium stiff, low plasticity [ALLUVIUM]
						30			Groundwater encountered at 30.0 feet during drilling
						10			Grades dark gray
						35			
						11			Grades dark brown
						12			SANDSTONE Medium brown, closely fractured, strong, hard, slightly to moderately weathered [BEDROCK]
			60/3"	24.9	93	40			Bottom of boring at 40.25 feet. Groundwater measured at 30.0 feet 10 minutes after 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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01BL.dwg

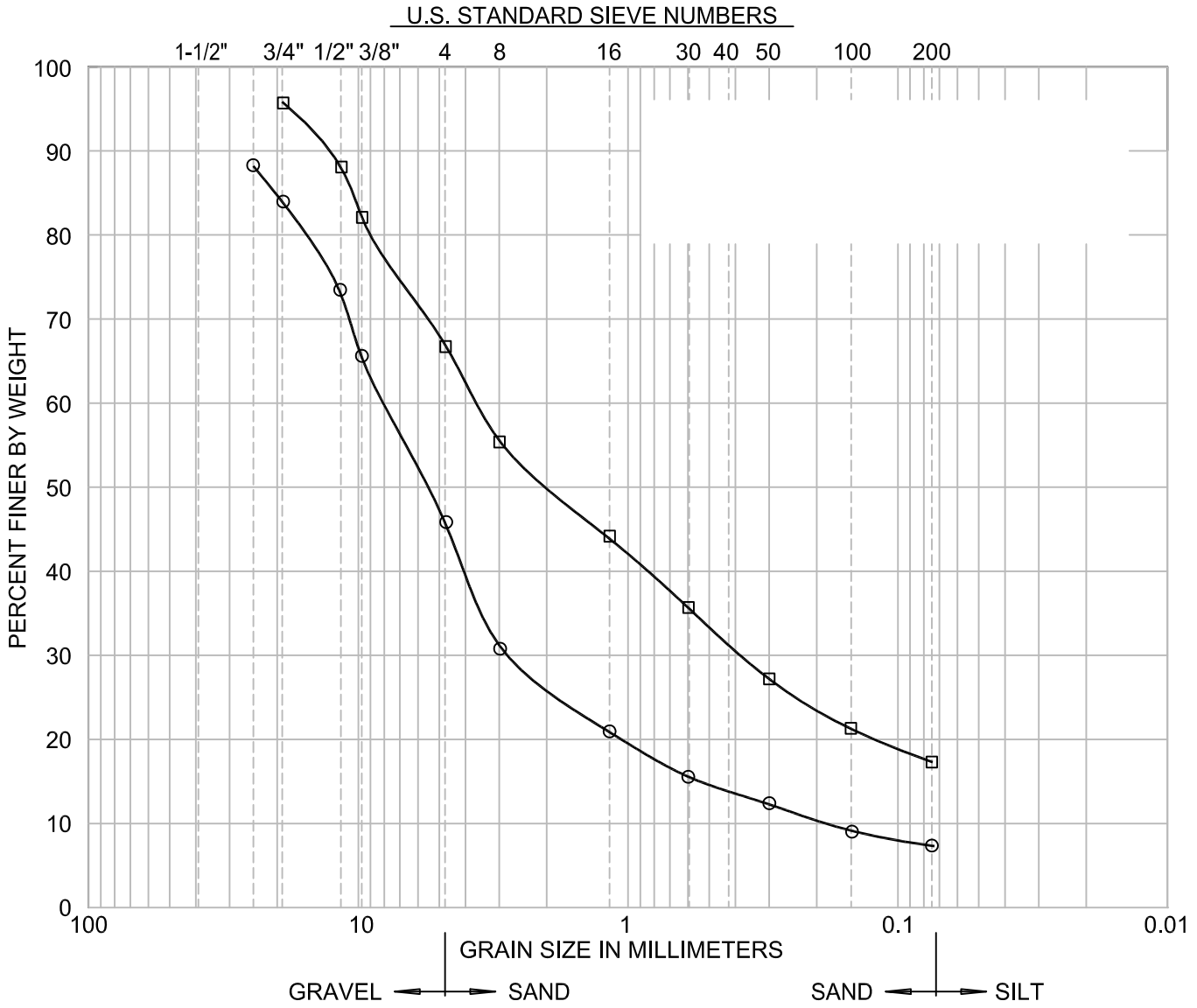
BORING LOG

Fair-Anselm Plaza
Fairfax, California

Drawn MFJ
Checked

Project No. 1598.01 Date: 7/21/09

A-8
FIGURE



SYMBOL	SAMPLE SOURCE	CLASSIFICATION
—o—	B2@10.0'	Sandy Gravel (GW)
—□—	B4@16.0'	Gravelly Sand with Silt (SW)

REFERENCE: CALTRANS STANDARD SPECIFICATIONS, JULY 1995

Miller Pacific
ENGINEERING GROUP

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

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01SIEVE

SIEVE ANALYSIS

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 8/17/09

Drawn MFJ
Checked

A-9
FIGURE

Neil O. Anderson & Assoc., Inc.
1190 Burnett Ave, Suite A, Concord, CA 94520
(925)609-7224 Fax (925)609-6324

LOG OF TEST BORING

BOREHOLE NUMBER

B5

PROJECT NUMBER: **WGG0051**
PROJECT NAME: **FAIR-ANSELM PROPERTY**
LOCATION: **CENTER ROAD, FAIRFAX, CA**
DRILLING EQUIP.: **B-24 TRUCK MOUNTED DRILL RIG**

DATE DRILLED: **12/20/07**
GROUND SURFACE ELEVATION: **0.0** Feet

PLATE NO. 6

Depth, ft.	Sample	Sampling Method	Moisture, %	Dry Density, pcf	Blow Counts	Blow Count Histogram	Ground Water	Soil Lithology	Soil Lithology Description	Notes
0									ASPHALT: (AC) Asphaltic Concrete, 2"	
-1									SANDY CLAY: (CL) Clay, sandy, very stiff, moist, brown, rust mottling	
-2										
-3										
-4										
-5										
-6		CM	10	102	28					
-7										
-8										
-9										
-10										
-11		CM	11	99	36					
-12										
-13									SANDY CLAY: (CL) Clay, sandy, stiff, moist, brown, grey mottling	
-14										
-15										
-16		CM	18	101	29					pi (ll = 37, pi = 34) -200 = 83%
-17										
-18										
-19										
-20										
-21		CM	14	106	16					
-22									SAND: (SP) Sand, dense, very moist to saturated	
-23							▼			
-24										
-25									GRAVEL: (GP) Gravel, dense, saturated	
-26		CM	11	117	25					Test hole terminated

Miller Pacific
ENGINEERING GROUP

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED
FILE: 1598.01NOABoring

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

BORING LOG

Fair-Anselm Plaza
Fairfax, California

Project No. 1598.01 Date: 8/25/09

Drawn MFJ
Checked

A-10
FIGURE

APPENDIX B

SONIC ECHO TESTING

ABE Engineering

2230 Lariat Lane, Walnut Creek, CA 94596
PHONE: 925-944-6363 FAX: 925-476-1588
EMAIL: SA@AbeEngineering.com

Pile Test Results

Company:	Miller Pacific Engineering	**Date:**	August 13, 2009
Attn:	Scott Stephens	**From:**	Steve Abe
Re:	Sonic Pulse Echo Tests 711 Center Blvd. Fairfax, CA July 30, 2009		Job No. 9051

This report summarizes the results from non-destructive sonic pulse echo testing performed on July 30, 2009 for nine drilled piles located underneath the building at the above referenced address. The purpose of the testing was to estimate the lengths of the existing piles.

PIT Test Method and Background

PIT employs the sonic pulse echo method to estimate pile length and detect anomalies or pile damage by measuring changes in pile impedance below the pile top, based on stress-wave reflections. The test is normally performed by striking the pile top (free pile end) with a small hammer and measuring the pile top response with an accelerometer. A Pile Integrity Tester was used to process and analyze the acceleration signals. The PIT Tester digitizes and conditions the acceleration signal and integrates it to a velocity record in the field and analysis and test output was done in the office later.

The PIT test results consist of pile velocity records. For piles with uniform cross sectional impedance throughout their lengths (such as pre-cast piles), velocity reflections should occur only at the free pile ends (top and bottom). The free end of the pile at the toe produces a positive tension wave reflection. Measurement of the compression wave travel time is used to estimate the pile length based on the pile wavespeed. If the pile length is known, the wavespeed can be measured. However, if the pile length is not known accurately, an assumed wavespeed must be used. Intermediate velocity reflections can occur before the toe reflection due to anomalies in the pile, extensive pile damage, abrupt changes in soil shaft resistance, or secondary reflections from a grade beam or superstructure. If a pile is completely broken, a pile toe reflection will not appear in the record and only the damage reflection will occur.

The velocity reflections occur from a change in pile impedance which is defined as (EA/c), where E is the elastic modulus of concrete, A is the cross sectional area of the pile, and c is the wave speed of concrete. The length of the pile or distance to a reflection (L) is determined by multiplying one half the wave travel time t by the wavespeed c ($L = c\,t / 2$). For this test an assumed pile wavespeed of 14,000 ft/sec was used which is a typical average value concrete piles. However, because the actual wavespeed could vary by about 10%, the actual pile lengths could also vary by plus or minus 10% from the reported length.

Discussion of P.I.T. Results

The PIT records are attached and indicate the apparent pile lengths based on the assumed wave speed and the time to the 2L/c pile toe reflection. The piles were labeled with marking paint at the site for reference to this report with horizontal lines in the pile showing the test locations. Thus, the reported pile lengths are referenced below these horizontal lines for each pile.

Note that a major defect or abrupt change in pile cross sectional area will produce a reflection similar in appearance to a pile toe reflection, and there is usually know way to discern between the two. Thus for these results it is assumed that the first major reflection indicates the pile toe or pile length. Also, pile toes

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

FILE: 1598.01APPENDIXB.dwg

that are drilled into rock exhibit a negative toe reflection while pile toes that are founded in softer soils exhibit positive toe reflections. Therefore, some speculation regarding the nature of the material at the pile toe can be drawn from the appearance of the PIT records. The PIT results are summarized in the following table.

Pile No.	Pile Length (ft)	Comments
1	22	Toe appears to be in soft soil
2	21	Toe appears to be in soft soil
3	30	Toe appears to be in rock or high end bearing layer
4	30	Toe appears to be in rock or high end bearing layer
5	28	Toe appears to be in rock or high end bearing layer
6	30	Toe appears to be in rock or high end bearing layer
7	29	Toe appears to be in rock or high end bearing layer
8	21	Toe appears to be in soft soil
9	?	Inconclusive record due to pile top condition
10	27	Toe appears to be in rock or high end bearing layer

Please note that the P.I.T. results do not evaluate pile capacity or the load transfer characteristics of the piles. Furthermore, PIT records will not detect all defects or damage that may exist on a pile. Cracks that are held together by reinforcement will typically not be detected and often, effects of high soil friction can mask or hide anomalies in the records. The results are by no means intended to verify the adequacy or structural load capacities of the piles. Rather they are intended to provide additional supplemental information to aide the engineer of record in evaluating the in-situ foundation conditions.

I appreciate the opportunity to be of assistance to you on this project. Please do not hesitate to contact me if you have any questions regarding the PIT results.

Very truly yours,
ABE Engineering

Steve Abe, P.E.

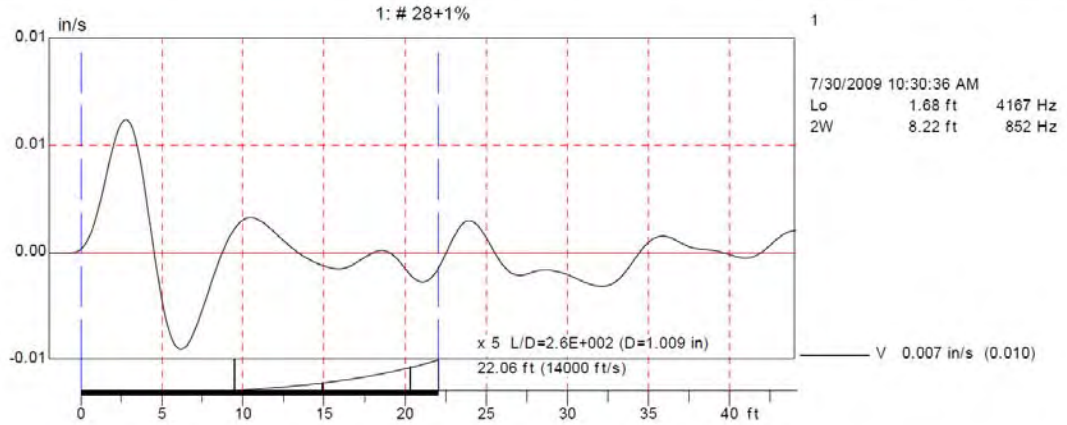
REGISTERED PROFESSIONAL ENGINEER
STEVEN K. ABE
No. C 055277
Exp. 12/2008
CIVIL
STATE OF CALIFORNIA

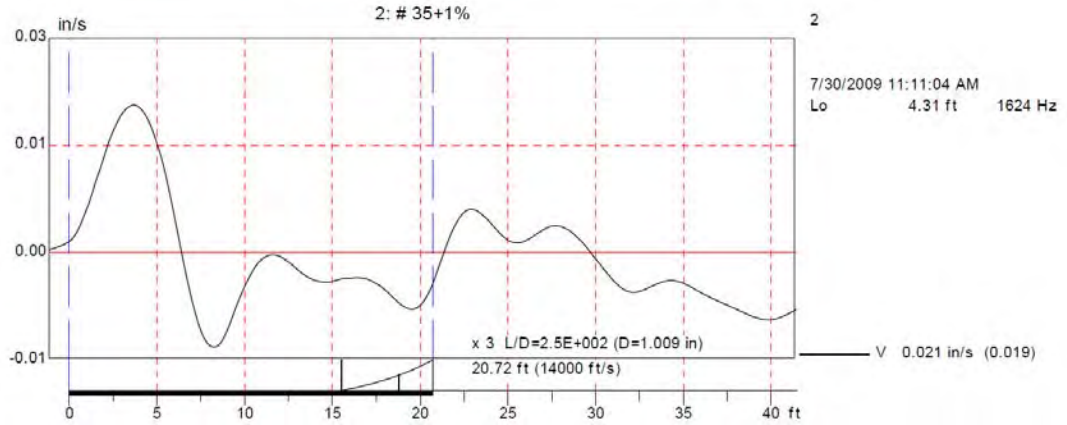
A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED

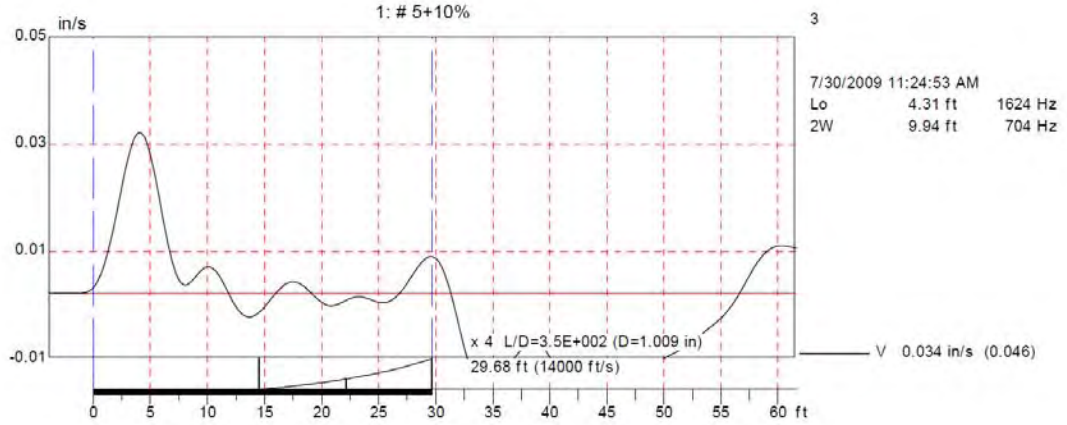
ABE Engineering
771 CENTER
C:\Documents and Settings\Steve Abe\My Documents\abe-docs\771 center pit.PIT

8/13/2009

PIT-W™ Ver. 2001-1



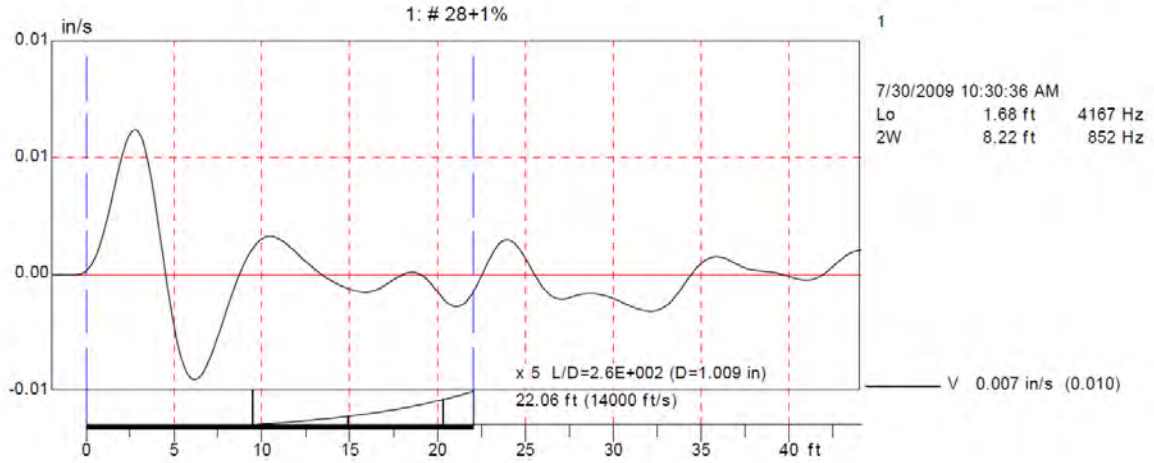


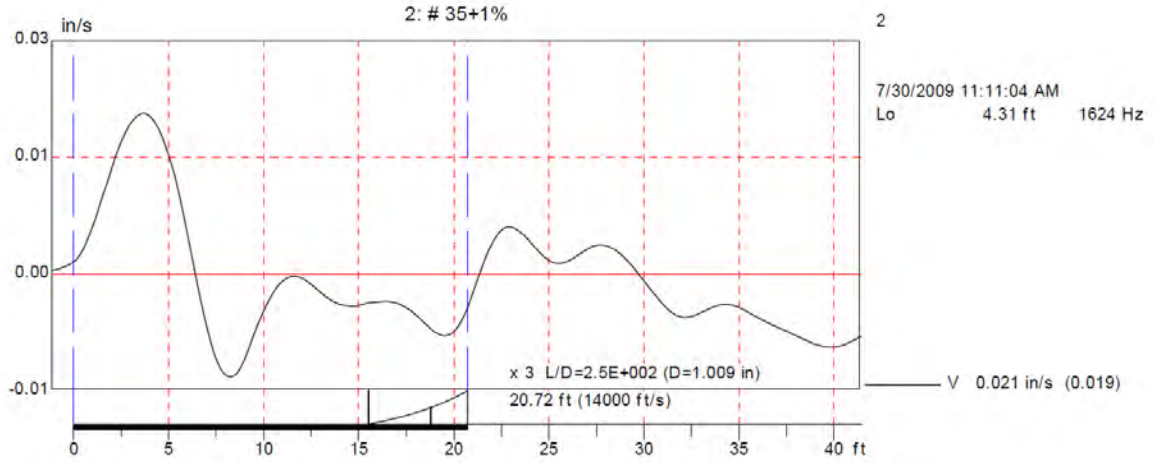


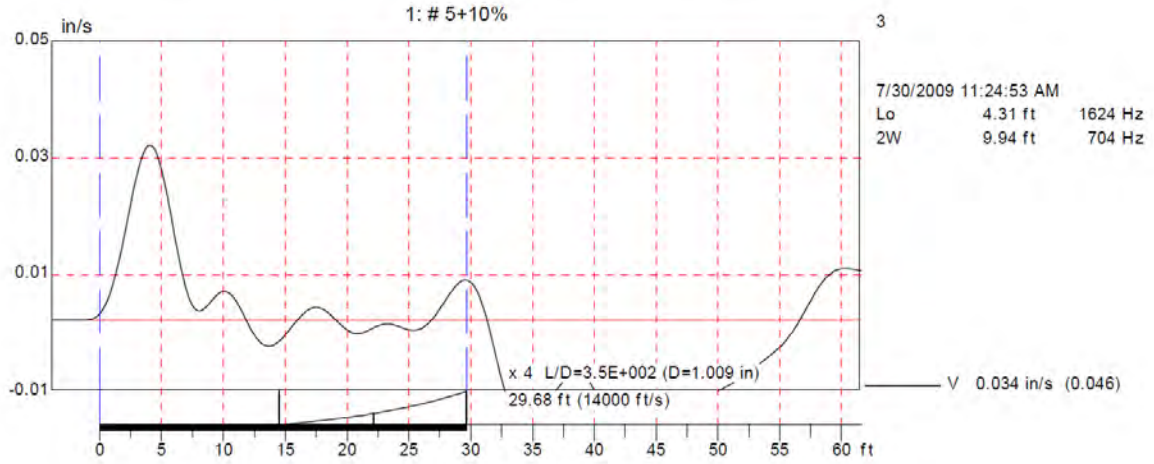
Miller Pacific ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SONIC ECHO TEST REPORT		
		Fair-Anselm Plaza Fairfax, California	Drawn MFJ Checked	B-3 FIGURE
A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED FILE: 1598.01APPENDIXB.dwg		Project No. 1598.01 Date: 8/14/09		

ABE Engineering
771 CENTER
C:\Documents and Settings\Steve Abe\My Documents\abe-docs\771 center pit.PIT

8/13/2009
PIT-W™ Ver. 2001-1





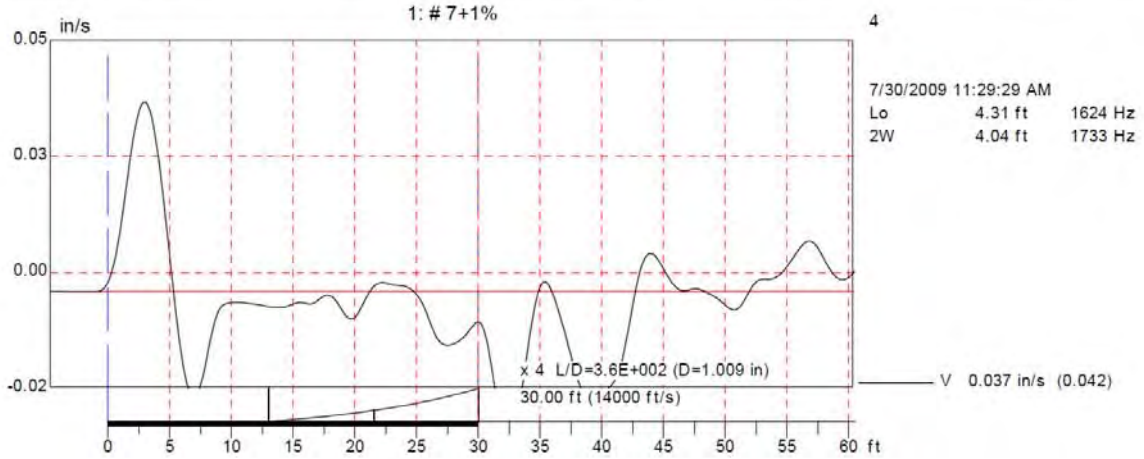


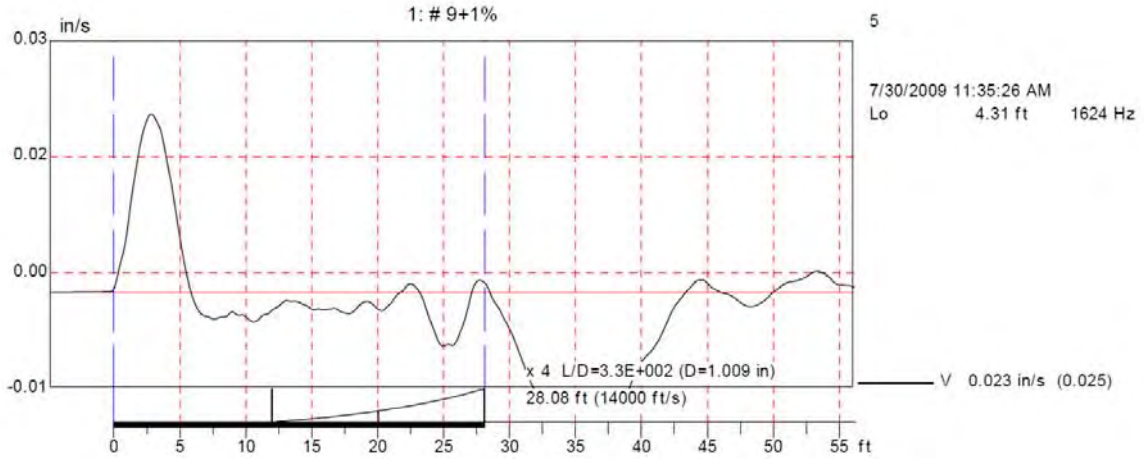
Miller Pacific ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SONIC ECHO TEST REPORT		
		Fair-Anselm Plaza Fairfax, California	Drawn MFJ Checked	B-4 FIGURE
A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED FILE: 1598.01APPENDIXB.dwg		Project No. 1598.01 Date: 8/14/09		

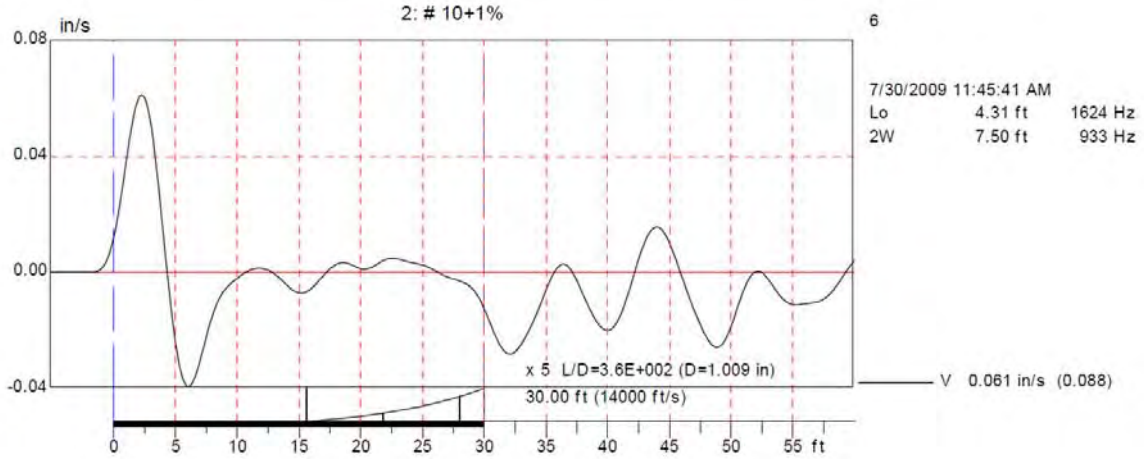
ABE Engineering
771 CENTER
C:\Documents and Settings\Steve Abe\My Documents\abe-docs\771 center pit.PIT

8/13/2009

PIT-W™ Ver. 2001-1







Miller Pacific ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SONIC ECHO TEST REPORT	Drawn MFJ Checked	B-5 FIGURE
		Fair-Anselm Plaza Fairfax, California		
		Project No. 1598.01 Date: 8/14/09		

A CALIFORNIA CORPORATION, © 2008, ALL RIGHTS RESERVED
FILE: 1598.01APPENDIXB.dwg