GEOTECHNICAL INVESTIGATION
VIVARIUM REPLACEMENT AND EXPANSION PROJECT
SAN FRANCISCO VA MEDICAL CENTER (SFVAMC)
San Francisco, California

HDR Architects
San Francisco, California

22 September 2010
Project No. 5106.01

Treadwell&Rollo
Environmental & Geotechnical Consultants

Treadwell&Rollo

22 September 2010
Project 5106.01

Ms. Anne Gluch
HDR Architects
560 Mission Street, Suite 900
San Francisco, California 94105

Subject: Geotechnical Investigation
Vivarium Replacement and Expansion Project
San Francisco VA Medical Center
San Francisco, California

Dear Ms. Gluch:

We are pleased to present our geotechnical investigation report for the proposed Vivarium replacement and expansion project at the San Francisco VA Medical Center (SFVAMC) at 4150 Clement Street in San Francisco, California. Our investigation was performed in accordance with our proposal dated 30 June 2010. This summary omits the detailed recommendations; therefore, anyone relying on the report must read it in its entirety.

The site is roughly triangular in shape with plan dimensions of approximately 150 feet by 170 feet by 230 feet. The site slopes downward in the southeast direction. It is bound by existing Buildings 6, 14 and 26 to the northeast, Building 12 to the south, and Buildings 21 and 205 and a water tower to the west. Currently, the site is occupied by Building 17, a concrete pathway, and landscaping.

We understand the current plans include constructing a new 11,800 square foot Vivarium east of the existing water tower. The new Vivarium will be two stories in height and will have a finished floor elevation near the lowest existing site grade in the northeast portion of the site. To maintain a constant top of slab elevation, the first floor of the buildings will be about 12 feet below grade in its northwest portion. The project may involve the demolition of Building 17, constructing a covered walkway linking the Vivarium to Building 12, and relocation of utilities.

The site is blanketed by 2 to 5 feet of fill. Native soil, consisting of very stiff clay with bedrock fragments, underlies some of the fill beneath the southwest portion of the site. Bedrock of the Franciscan Complex was encountered below the fill and/or native soil at depths ranging from 2 to 6 feet below the existing ground surface.

The recommendations contained in the report are based on a limited subsurface exploration program. Consequently, variations between expected and actual soil conditions may be found during construction. We should therefore be retained to observe shoring construction, installation of foundation, site grading, and compaction of utility trench backfill.

We appreciate the opportunity to assist you with this project and look forward to working with you during final design and construction.

Sincerely yours,
TREADWELL & ROLLO, INC.

Richard D. Rodgers

Richard D. Rodgers, G.E.
Principal

REGISTERED PROFESSIONAL ENGINEER
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GEOTECHNICAL INVESTIGATION
Vivarium Replacement and Expansion Project
San Francisco VA Medical Center (SFVAMC)
San Francisco, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Treadwell & Rollo, Inc. for the proposed replacement and expansion of the San Francisco VA Medical Center (SFVAMC) Vivarium at 4150 Clement Street in San Francisco, California, located as shown on Figure 1. The site for the proposed building is within the SFVA Hospital campus east of an existing water tower.

The site is roughly triangular in shape with plan dimensions of approximately 150 feet by 170 feet by 230 feet. It is bound by existing Buildings 6, 14, and 26 to the northeast, Building 12 to the south, and Buildings 21 and 205, and the water tower to the west, as shown on Figure 2. Currently, the site is occupied by Building 17, a concrete pathway, and landscaping. Building 205 houses the central plant for the Hospital and there are numerous underground utilities in the vicinity associated with it.

We understand the current plans include constructing a new 11,800 square foot Vivarium two stories in height that will have a finished floor elevation near the lowest existing site grade in the northeast portion of the site, approximately Elevation 340 feet[1]. The site slopes up from the northeast reaching about Elevation 352 feet in the northwest portion of the site near the water tower. Therefore construction of the first floor may include cuts of up to 12 feet for the first floor slab, plus a few feet for the foundation, at the high point of the site. The project may involve the demolition of Building 17, constructing a covered walkway that links the Vivarium to Building 12, and the relocation of utilities.

2.0 SCOPE OF SERVICES

As outlined in our proposal dated 30 June 2010, our geotechnical services included exploring the subsurface conditions by drilling four test borings and performing engineering analyses to develop conclusions and recommendations regarding:

- Soil, bedrock and groundwater conditions at the site
- site seismicity and seismic hazards, including potential for fault rupture, ground shaking, liquefaction, landsliding, lateral spreading, and seismically induced settlements

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1 Reference: Site survey map by Sandis, e-mailed to Treadwell & Rollo, Inc. on 20 July 2010. All elevations reference City and County of San Francisco Datum.

- mitigation of liquefaction potential, if appropriate
- allowable bearing capacity and lateral resistance parameters for existing foundations (including vertical and horizontal load deformation curves), as appropriate
- appropriate foundation type(s)
- design criteria for the new foundation type(s), including lateral load resistance, uplift and bearing capacities, as appropriate
- friction coefficients for the resistance of lateral loads by footings
- anchors for the resistance of seismic uplift
- lateral earth pressures (static and seismic) for basement and cantilever retaining walls
- influence of construction on nearby structures, including the water tower (from a geotechnical standpoint)
- seismic factors of the 2007 California Building Code
- soil corrosion potential
- construction considerations

3.0 FIELD EXPLORATION AND LABORATORY TESTING

To evaluate the subsurface conditions at the site, we drilled four test borings, the approximate locations of which are shown on the Site Plan, Figure 2. Prior to performing our field investigation, we:

- notified Underground Service Alert (USA)
- checked the boring locations for utilities using an independent private utility locator.

Details of the field investigation activities and laboratory testing are described in the remainder of this section.

3.1 Borings

On 23 July 2010, four borings were drilled by Access Soil Drilling using limited-access drilling equipment. The borings were drilled to depths from approximately 2.5 to 18.3 feet below the existing ground surface (bgs). The borings were drilled under the direction of our field engineer who logged the soil and rock encountered and obtained representative samples for visual classification and laboratory testing. Logs of the borings are presented on Figures A-1 through A-4 in Appendix A. The soil and rock encountered in the borings are classified in accordance with the charts presented on Figures A-5 and A-6, respectively.

Soil and bedrock samples were obtained using two different types of split-barrel samplers. The sampler types are as follows:

- Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel tubes with an inside diameter of 2.43 inches
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter, without liners.

The sampler types were chosen on the basis of soil or rock type being sampled and desired sample quality for laboratory testing.

The SPT and S&H samplers were driven with a 140-pound, above-ground, rope and pulley safety hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy and are shown on the boring logs. The blow counts used for this conversion were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and 3) the only blow count if the sampler was driven six inches or less.

Upon completion, the boreholes were backfilled with grout consisting of cement, bentonite, and water. Few soil cuttings were generated, less than about 5 gallons for all four borings; therefore, the soil at each location was spread near each hole in the landscape areas.

3.2 Laboratory Testing

The soil and rock samples recovered from the field exploration program were re-examined in the office for classifications, and representative samples were selected for laboratory testing. The laboratory testing program was designed to correlate and evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, dry density, plasticity (Atterberg limits), fines content, and shear strength, where appropriate. Results of the laboratory tests are included on the boring logs and in Appendix B.

Because corrosive soil can adversely affect underground utilities and foundation elements, laboratory testing was performed to evaluate the corrosivity of the near surface soil. The results of the corrosivity analyses are presented in Appendix C.

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 Site Conditions

The site is roughly triangular in shape with plan dimensions of approximately 150 feet by 170 feet by 230 feet. The site slopes downward in the southeast direction. Existing grades for the site range from about Elevation 340 to 352 feet. Currently, the site is occupied by Building 17, a concrete pathway, and landscaping.

From our field observations and the site survey, we understand the following structures border the site:

- Building 21 located west of the Vivarium site; the building is a 1-story metal structure with adjacent site grades near Elevation 343 feet.
- Building 6 located northeast of the Vivarium site; Building 6 is a 3-story concrete structure with adjacent site grades near Elevation 342.5 feet.
- Building 26 is located northeast of the proposed Vivarium footprint. Building 26 is a 1-story metal container with adjacent site grades near Elevation 340 feet.
- Building 12 located south of the proposed building footprint; the building is a 2-story concrete structure with one basement level. The finished floor elevation of the basement is about 330.5 feet. A retaining wall is located north of Building 12, between Building 12 and the Vivarium site, and the top of wall elevation is about 343 feet.
- Building 205 located southwest of the Vivarium site; Building 205 is a 1-story structure with one basement level. The finished basement floor elevation is about 351.5 feet.
- Building 28 is adjacent to the portion of the west side of the Vivarium building pad. Building 28 is one-story with a slab-on-grade floor. The top of slab is at about Elevation 353 feet.

- The water tower is west of the proposed project footprint. The ground surface elevations near the base of the water tower are approximately 352 feet.
- The water tower was constructed about 1972 and originally was supported on spread footings bearing in bedrock. Subsequent to the Loma Prieta Earthquake in 1989, the footings (four) were supplemented by eight (two at each footing) 3-foot diameter drilled piers about 13 feet long.

The proximity of these structures to the site is shown on Figure 2.

4.2 Subsurface Conditions

Subsurface information from our investigation indicates that the site is underlain by 2 to 5 feet of fill. The fill consists of silty sand and clayey sand and very stiff sandy clay. Native soil, consisting of very stiff clay with bedrock fragments, underlies some of the fill beneath the southwest portion of the site. The native soil, where encountered, is about 2 feet thick. Atterberg limits tests performed on the fill indicate it is non-expansive; however the native soil was found to be moderately to highly expansive. Bedrock of the Franciscan Complex was encountered below the fill and/or native soil. Bedrock consists of sandstone and shale. The top of bedrock contours slope downwards to the southeast in the same direction as the ground surface elevation changes. The approximate top of bedrock range from Elevation 338 to 345 feet.

Groundwater was not encountered during this investigation.

5.0 REGIONAL SEISMICITY AND FAULTING

The major active faults in the region are the San Andreas, Hayward, and Calaveras Faults. These and other faults of the region are shown on Figure 3. For each of the active faults within 50 kilometers, the distance from the site and estimated maximum Moment magnitude[2], M_w, [Working Group on California Earthquake Probabilities (WGCEP) (2003) and Cao et al. (2003)] are summarized in Table 1.

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2 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Maximum Moment Magnitude
San Andreas – 1906 Rupture (SAS+SAP+SAN+SAO)	4.7	West	7.9
San Andreas – Peninsula (SAP)	4.7	West	7.2
San Andreas – SAP+SAN+SAO	4.7	West	7.8
San Andreas – SAS+SAP	4.7	West	7.4
San Andreas – SAS+SAP+SAN	4.7	West	7.8
San Andreas – SAN	5.9	West	7.5
San Andreas – SAN+SAO	5.9	West	7.7
San Gregorio – SGN	10	West	7.2
San Gregorio – SGS+SGN	10	West	7.4
Hayward-Rodgers Creek – NH	24	East	6.5
Hayward-Rodgers Creek – NH+RC	24	East	7.1
Hayward-Rodgers Creek – SH+NH	24	East	6.9
Hayward-Rodgers Creek – SH+NH+RC	24	East	7.3
Hayward-Rodgers Creek – SH	26	East	6.7
Point Reyes	34	West	6.8
Hayward-Rodgers Creek – RC	35	North	7.0
Mt Diablo – MTD	42	East	6.7
Calaveras – CC+CN	44	East	6.9
Calaveras – CN	44	East	6.8
Calaveras – CS+CC+CN	44	East	6.9
Monte Vista-Shannon	45	Southeast	6.8
Concord/GV – CON	47	East	6.3
Concord/GV – CON+GVS	47	East	6.6
Concord/GV – CON+GVS+GVN	47	East	6.7
Concord/GV – GVS	47	Northeast	6.2
Concord/GV – GVS+GVN	47	Northeast	6.2
West Napa	48	Northeast	6.5

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through January 1996. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated M_w for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 100 km from the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (M_w = 6.2).

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
WGCEP (2007) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	31
N. San Andreas	21
Calaveras	7
San Gregorio	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

6.0 GEOLOGIC HAZARDS

During a major earthquake, strong to violent ground shaking is expected to occur at the project site. Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction,[3] lateral spreading,[4] cyclic densification,[5] landsliding, or can cause a tsunami. Each of these phenomenon has been evaluated based on our literature review, field investigation and analysis, and is discussed in this section.

6.1 Liquefaction and Associated Hazards

When a saturated, cohesionless soil liquefies during a major earthquake, it experiences a temporary loss of shear strength caused by a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction. The California Division of Mines and Geology (CDMG) has prepared a map titled State of California Seismic Hazard Zones, City and County of San Francisco, released 17 November 2000. This map was prepared in accordance with the Seismic Hazards Mapping Act of 1990. Based on this map, the site is not within a liquefaction hazard zone.

Groundwater was not encountered during our investigation and all borings were terminated in bedrock; therefore, we judge the risk of liquefaction and lateral spreading at the Vivarium site is nil.

6.2 Cyclic Densification

Seismically-induced cyclic soil densification of non-saturated sandy soil may result in settlement of the ground surface. The soil overlying bedrock is generally clayey; therefore, we estimate little to no settlement will occur at the site due to cyclic densification.

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3 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

4 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

5 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground-surface settlement.

6.3 Tsunami

According to published data (URS/Blume, 1974) the maximum run up (tsunami wave) at the Presidio occurred after the 1964 Alaskan earthquake. The wave measured 7.5 feet at the Golden Gate; no damage was reported along the San Francisco shoreline. The United States Geologic Survey (USGS) estimates the maximum probable tsunami wave run up at the Golden Gate will be 20 feet (Ritter and Dupre, 1972). The site is over 300 feet above sea level and therefore is not within an area of potential tsunami inundation.

6.4 Surface Faulting

We evaluated the risk of surface faulting at the site associated with active or potentially active fault traces. Historically, ground surface displacements closely follow the trace of geologically young faults. Based on our study, we conclude the site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. Therefore, we judge the risk of surface faulting at the site is very low. However, in a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed.

6.5 Landslides, Erosion, and Seepages

The site is not within an area designated as a landslide hazard zone as shown on the 2000 CDMG seismic hazard map. Active or potentially active landslides are present to the north of the site. However, during our site visit we did not observe evidence of landsliding at the Vivarium site nor any potential impact from any landslide mapped on the campus. No excessive erosion nor any springs or seepages were observed on the site.

7.0 DISCUSSION AND CONCLUSIONS

On the basis of our investigation and experience with similar sites, we conclude the project is feasible from a geotechnical standpoint. Geotechnical issues of concern include:

- adequate foundation support
- lateral earth pressures on subsurface walls
- potentially expansive native soil
- construction considerations, including shoring or sloping the sides of excavations and bedrock excavation
- temporary vertical and lateral support of adjacent structures during excavation.

These issues and their impact on the proposed grading, shoring, foundation design, and construction are discussed in the following sections.

7.1 Foundation Support and Settlement

The site is underlain by 2 to 5 feet of fill. Native soil, consisting of very stiff expansive clay with bedrock fragments, underlies some of the fill beneath the southwest portion of the site. Bedrock was encountered below the fill and/or native soil at depths ranging from 2 to 6 feet bgs. The fill, which is undocumented and variable, is not suitable for the support of the proposed building. The proposed structure should be supported on foundations bearing on bedrock. Because of the planned excavation and the shallowness of the bedrock, we judge spread footings can be used. Current plans include a finished floor elevation that will daylight in the northeast portion of the site, at approximate Elevation 340 feet. Therefore, a cut of approximately 12 feet, plus the depth of foundations, will be needed on the west side of the site. Temporary slopes or shoring will be required to provide stable sides for the cut, as discussed in Section 7.5.

Spread footings should be supported on bedrock. Where bedrock is relatively deep, the footing excavation should extend to bedrock and be backfilled with lean concrete to the foundation subgrade. Where foundation excavations extend deeper than five feet, shoring will be required before men can enter the excavation. We estimate total settlement of spread footings that gain support on bedrock will be on the order of ¼ to ½ inch, and differential settlement between column locations will be about ¼ inch.

7.2 Floor Slabs

Because of the planned excavation the fill and native soil should be removed throughout most of the building footprint. Native soil may be exposed in the eastern part of the building pad where the excavation depth will be shallow. Because of its expansion potential, where exposed, the native soil should be kept moist. If allowed to dry, it could expand and heave when it becomes wet again and cause distress to the floor slab. Floor slabs should be underlain by a moisture barrier supported on subgrade prepared in accordance with Section 8.1. Once the moisture barrier and floor slab are in place, the potential for the subgrade soil to dry out should be eliminated.

7.3 Corrosion Potential

We performed three corrosivity tests on soil collected from borings B-1, B-2, and B-3 at depths ranging from 0 to 7 feet bgs. The soil samples were tested in accordance with Caltrans and ASTM protocols. The test results are attached in Appendix C.

7.4 Groundwater

Groundwater was not encountered during our investigation to the maximum depth explored of 18.3 feet bgs. However, any water infiltrating the surface soil is likely to become perched on the bedrock and migrate along that interface. Therefore, depending upon the time of year excavations are made, water may be encountered. Further, because of the highly fractured nature of the bedrock, water could be encountered in seams and fractures in the rock and be present in a virtually any depth. If encountered during construction water will need to be dealt with through drains and other means to minimize impacts on the building floor and walls.

7.5 Construction Considerations

7.5.1 Shoring

We understand the finished floor will be at approximately Elevation 340 feet. Excavations ranging in depth between approximately 3 to 14 feet are anticipated, including a few feet for the foundations of the new building. It will be necessary to construct a permanent basement wall along the western and northern sides of the building and provide temporary shoring or slopes for the excavation sides. The primary considerations related to the selection of shoring system versus sloping are space and protection of surrounding improvements. Surcharges from the adjacent structures, including the water tower, should be evaluated in the design of a shoring system. Additionally, surcharges imposed on existing building basement walls and the retaining wall to the south by construction of the new building should be evaluated.

We conclude the excavation sides can be temporarily retained using a soldier-pile-and-lagging shoring system, with or without tie-backs. A soldier-pile-and-lagging system consists of concrete encased steel H-beams placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds. Lagging boards should be placed with every two feet of excavation. Voids that result from the excavation behind wood lagging should be grouted before proceeding to the next section of lagging. Tie-backs may be included to provide lateral support as deemed appropriate by the shoring designer.

As an alternative to a soldier pile and lagging system, soil/rock nails may be feasible. The effectiveness of a nailed system should be evaluated and, if appropriate, designed by an engineer experienced with such systems.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. A structural engineer knowledgeable in this type of construction should design the shoring.

7.5.2 Temporary Slopes

If space permits, and excavation outside the building limits is permissible, the excavation sides can be sloped to avoid shoring. If temporary slopes are to be near the water tower, the effects on the tower foundation should be evaluated.

7.5.3 Excavation

Grading for the proposed lower floor of the building will require excavating soil and bedrock. We anticipate most of the excavation in rock can be made using a large excavator or bulldozer with rippers. However, some hard rock should be anticipated and the use of a jack hammer or hoe ram may be required. The soil overlying the rock consists of soil fill and very stiff clay that can be excavated with earth-moving equipment. Other than utilities, we are not aware of obstructions to excavations that could be present at the site.

8.0 RECOMMENDATIONS

8.1 Site Preparation and Fill Placement

Site preparation should include removal of all existing structures, foundations, slabs, pavements, and underground utilities within the footprint of the proposed building. All areas to receive improvements should be stripped of vegetation and organic topsoil. Stripped materials should be removed from the site or stockpiled for later use in landscaped areas, if approved by the landscape architect. Existing foundations should be removed to expose native soil. Any other subsurface structures and debris should also be removed, and any fill uncovered should be overexcavated and recompacted. Underground utilities should be removed to the service connections and properly capped or plugged with concrete.

Where existing utility lines will not interfere with the proposed construction, they may be abandoned in-place, provided the lines are filled with lean concrete or cement grout to the limits of the project. Voids resulting from demolition activities should be properly backfilled with engineered fill as described in this section.

From a geotechnical standpoint, concrete and asphalt generated by demolition may be crushed and reused as fill provided it is free of organic material and rocks or lumps greater than three inches in greatest dimension. The acceptability of using crushed asphalt at the site should be verified by the VA and architect. Where crushed concrete or asphalt is used, particles between 1-1/2 and 3 inches in greatest dimension should comprise no more than 30 percent of the fill by weight.

Based on a finished floor at about Elevation 340 feet, most of the site will require excavation. We anticipate the excavation will expose bedrock except in the eastern portion of the building pad where only shallow excavation is planned. However, footing excavations are anticipated to extend to rock throughout.

In areas to receive new fill or site improvements, we recommend the exposed soil subgrade be scarified to a depth of at least 6 inches, moisture-conditioned to above the optimum moisture content, and compacted to at least between 90 percent relative compaction. Bedrock does not need to be scarified and recompacted. The soil subgrade should be kept moist until it is covered by fill or improvements. The upper six inches of soil subgrade beneath areas that will receive vehicular traffic should be compacted to at least 95 percent relative compaction and be non-yielding.

Onsite fill, soil, and rock are suitable for reuse as backfill provided they meet the requirements given below for general fill. All materials to be used as fill, including onsite soil and rock, should be free of organic material, contain no rocks or lumps larger than three inches in greatest dimension, and have a low expansion potential (defined by a liquid limit of less than 35 and a plasticity index lower than 12). Fill should be placed in lifts not exceeding eight inches in loose thickness and compacted to at least 90 percent relative compaction[6]. Fill thicker than five feet or containing less than 10 percent fines should be compacted to at least 95 percent relative compaction. During construction, we should check that the onsite and any proposed import material are suitable for use as fill.

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[6] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-07 laboratory compaction procedure.

Wall backfill should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy equipment is used, the wall should be appropriately designed to withstand loads exerted by the equipment and/or temporarily braced.

The Geotechnical Engineer should approve all sources of imported fill at least three days before use at the site. The grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed for analytical testing of the proposed import fill.

8.2 Utilities and Utility Trench Backfill

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least four inches on both sides. Where necessary, trench excavations should be shored and braced to prevent cave-ins and/or in accordance with safety regulations. Where trenches extend below groundwater; if encountered, it will be necessary to temporarily dewater them to allow for placement of the pipe and/or conduits and backfill.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped. Backfill should be placed in lifts of eight inches or less, moisture-conditioned to near the optimum moisture content, and compacted to at least 90 percent relative compaction. Beneath streets or sidewalks, the upper three feet of fill should be compacted to at least 95 percent relative compaction. If fill with less than 10 percent fines is used, the entire depth of the fill should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements resulting in damage to the pavement section.

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. Excavations that will be deeper than five feet and will have to be entered by workers should be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA standards (29 CFR Part 1926)). The contractor should be responsible for the design, construction, and safety of temporary shoring.

8.3 Foundations

The building can be supported on spread footings gaining support in bedrock. Where uplift resistance greater than can be provided by footings is required, tiedown anchors can be used. Recommendations for spread footings and anchors are presented in the following section.

8.3.1 Spread Footings

Spread footings should gain support in bedrock and may be designed for a maximum allowable bearing pressure of 8,000 pounds per square foot (psf) for dead plus live loads with a one-third increase for total loads including wind and seismic forces. Continuous and isolated footings should be at least 18 and 24 inches wide, respectively. All footings should be embedded at least 12 inches into rock. Where the design depth of the footing is insufficient to achieve this embedment, the excavation should be extended the required depth into rock and lean concrete placed back to the design bottom of footing. Lean concrete placed to support footing should have an unconfined compression strength of at least 1,000 pounds per square inch.

Lateral forces can be resisted by a combination of passive resistance acting against the vertical faces of the footings and friction along the bases of the footings. To calculate the passive resistance against the vertical face of footings directly supported on rock, we recommend using a uniform pressure of 2,600 psf. The upper one foot of passive resistance should be ignored where not confined by a slab. Frictional resistance should be computed using a base friction coefficient of 0.4. These values include a factor of safety of at least 1.5.

Footing excavations should be free of loose soil, debris, and water prior to placement of concrete. We should check footing excavations for bearing prior to placing reinforcing steel or lean concrete. Footing excavations should be maintained in a moist condition until concrete is placed.

8.3.2 Tiedown Anchors

Tiedown anchors may be used to provide uplift resistance to supplement that of footings. Tiedown anchors typically consist of relatively small-diameter, drilled, concrete or grout-filled shafts with steel bars or tendons embedded in the concrete or grout. The anchors develop their uplift resistance from friction between the sides of the shaft and the surrounding rock.

Tiedown anchors should be spaced at least three shaft diameters apart or four feet center-to-center, whichever is greater. The ultimate bond strength between the anchor and rock will depend on the type of anchor and installation procedure. For planning purposes, however, we recommend using an allowable skin friction of 2,000 psf. This value assumes the anchors will be post-grouted. The actual bond strength should be determined by the shoring designer. Higher values may be obtained depending upon the techniques employed by the contractor and the results of pullout tests.

Because the tiedowns will be permanent, we recommend they be double corrosion protected. If water is present in the shaft, concrete should be placed using a tremie system. High strength bars or strands may be used as tensile reinforcement in the anchors; however, if strands are used, a lock-off load should be used to limit deflections of the anchor during actual loading. The lock-off load should be accounted for in the footing design. A minimum stressing length (free length) of 10 and 15 feet should be provided for bar and strand tendons, respectively.

The first two production tiedowns and two percent of the remaining tiedowns should be performance-tested to 1.5 times the design load. All other tiedowns should be proof-tested to 1.5 times the design load. The anchors should be tested as recommend in Section 8.7. After testing, all anchors should be loaded and locked off to a portion of their design load as determined by the structural engineer and indicated on the structural drawings and/or specifications.

8.4 Slab-on-Grade Floors

The floor slab may be supported on grade, provided the subgrade is prepared in accordance with Sections 7.2 and 8.1. If the subgrade is disturbed during excavation for footings and utilities, it should be re-rolled. Loose, disturbed materials should be excavated, removed, and replaced with engineered fill during final subgrade preparation. Native soil subgrade should be kept moist until the floor slab is placed.

Moisture is likely to condense on the underside of the slabs, even though they will be above the design groundwater table. Consequently, a moisture barrier should be installed beneath the slabs if movement of water vapor through the slabs would be detrimental to its intended use. A moisture barrier is generally not required beneath parking garage slabs, except for areas beneath mechanical, electrical, and storage rooms. A typical moisture barrier consists of a capillary moisture break and a water vapor retarder.

The capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 3.

TABLE 3
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

The sand overlying the membrane should be dry at the time concrete is cast. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

8.5 Below-Grade Walls

Below-grade walls should be designed to resist lateral pressures imposed by the adjacent soil, rock, and any surcharge loads. Because the site is in a seismically active area, the design should also be checked for seismic condition. Under seismic loading conditions, there will be added seismic increment that should be added to active earth pressures (Lew et al. 2010). We used the procedures outlined in Lew et al. (2010) to compute the seismic active pressure. Table 4 presents the active, at-rest and total pressure (active plus seismic pressure increment) for both soil and rock for level backfill. All pressures are presented as equivalent fluid weights (triangular distribution).

TABLE 4
Retaining Wall Design Earth Pressures
(Drained Conditions)

Retained Material	Static Conditions		Seismic Conditions**
	Unrestrained Walls - Active	Restrained Walls – At-rest	Total Pressure – Active Plus Seismic Pressure Increment
Soil	40 pcf	60 pcf	55 pcf
Rock	30 pcf	50 pcf	35 pcf

** The more critical condition of either at-rest pressure for static conditions or active pressure plus a seismic pressure increment for seismic conditions should be checked.

If surcharge loads occur above an imaginary 45-degree line (from the horizontal) projected up from the bottom of a retaining wall, a surcharge pressure should be included in the wall design. If this condition exists, we should be consulted to estimate the added pressure on a case-by-case basis. Potential surcharges from Building 28 and the water tower as well as any other structure meeting these conditions, should be evaluated. Where truck traffic will pass within 10 feet of retaining walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 pounds per square foot applied in the upper 10 feet of the walls.

The recommended lateral earth pressures assume drainage systems will be installed behind walls and below the slab. Although groundwater is not expected to be a factor, water from other sources such as rainfall and irrigation can accumulate behind the walls. Below-grade walls should therefore be backdrained using prefabricated, drainage panels. The panels should extend from a depth of two feet below the existing ground surface to the bottom of the excavation. The drainage panels should extend to six-inch diameter perforated PVC collector pipes at the base of the wall. The pipes should be surrounded on all sides by at least six inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) and wrapped in filter fabric. At the bottom of the excavation, the geotextile face of the drainage panel should be wrapped around the perforated perimeter drain pipes.

Below-grade walls should be waterproofed and provided with water stops at all construction joints. The waterproofing should be placed directly against the backside of the walls (between the drainage panels and the walls).

8.6 Shoring

8.6.1 Soldier-Pile-and-Lagging

A soldier-pile-and-lagging system is an acceptable method to retain the sides of the planned excavation. The deeper sections of the shoring may need to be tied-back or internally braced. For design of a cantilevered shoring system, we recommend using an active earth pressure equivalent to a fluid weight of 40 pcf, assuming the ground behind the shoring is level. Shoring should be designed for surcharge loads where there will be existing foundation, construction equipment and/or stockpiled material above an imaginary 45 degree line (from the horizontal) projected upward from the bottom of the shoring. We can provide recommendations for surcharge pressures once surcharge loads are known. The potential for Building 28 and the water tower to surcharge the shoring should be evaluated. If traffic occurs within 10 feet of the shoring, a uniform surcharge load of 100 psf should be added to the design. The anticipated deflections of the shoring system should be estimated to check if they are acceptable. Lateral resistance can be gained by passive pressure acting on the face of the toe of the soldier piles. We recommend computing passive resistance using a uniform pressure of 2,600 psf. This value includes a factor of safety of at least 1.5. Passive pressure can be assumed to act over an area of three soldier pile widths assuming the toe of the soldier pile is filled with structural concrete. The upper foot of soil should be ignored when computing passive resistance.

The lateral pressures recommended for designing tied-back or braced shoring system are presented on Figure 5. The pressures shown assume no groundwater is encountered or the site is dewatered; hydrostatic pressures will not develop behind the wall. Surcharge loads should be consider the same as for a cantilevered wall.

The shoring designer should evaluate the required penetration depth of the soldier piles. The soldier piles should have sufficient axial capacity to support the vertical load component of the tiebacks and the vertical load acting on the piles, if any. To compute the axial capacity of the piles, we recommend using an allowable friction of 1,000 psf on the perimeter of the piles below the excavation level; this value assumes the piles will gain support in bedrock.

Tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation and sloping upwards at 60 degrees from the horizontal, where H is the wall height in feet. Tiebacks should have a minimum unbonded length of 15 feet. All tiebacks should have a minimum bonded length of 15 feet and spaced at least four feet on center. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

Tieback allowable capacity will depend upon the drilling method, hole diameter, grout pressure, and workmanship. For estimating purposes, we recommend using the skin friction values presented on Figure 5. These values assume the tiebacks will gain resistance in soil and include a factor of safety of about 1.5. Higher skin friction values may be used if confirmed with pre-production performance tests.

The contractor should be responsible for determining the actual length of tiebacks required to resist the lateral earth and water pressures imposed on the temporary retaining systems. Determination of the tieback length should be based on the contractor's familiarity with his installation method. The computed bond length should be confirmed by a performance- and proof-testing program (Section 8.7) under the observation of an engineer experienced in this type of work. Replacement tiebacks should be installed for tiebacks that fail the load test.

The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to at least 1.25 times the design load. All other temporary tiebacks should be proof-tested to at least 1.25 times the design load. Recommendations for tieback testing are presented in Section 8.7. The performance tests will be used to determine the load carrying capacity of the tiebacks and the residual movement. The performance-tested tiebacks should be checked 24 hours after initial lock off to confirm stress relaxation has not occurred. The geotechnical engineer should evaluate the results of the performance tests and determine if creep testing is required and select the tiebacks that should be creep tested. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer.

The shoring system should be designed by a licensed structural engineer experienced in the design of retaining systems, and installed by an experienced shoring specialty contractor. The safety of workers and equipment in or near the excavation is the responsibility of the contractor. The shoring engineer should be responsible for the design of temporary shoring in accordance with applicable regulatory requirements. Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. We should review the shoring plans and a representative from our office should observe the installation of the shoring. During construction, we should have the opportunity to observe the installation of the shoring system and check the condition of the soil encountered during excavation.

Based on our investigation, we expect that the soil to be retained by the shoring has insufficient cohesion to stand vertically for large cuts. During excavation, lagging boards should be installed after every one-foot of cut to minimize caving. If voids are created behind lagging boards, they should be filled with cement slurry or hand-packed soil prior to proceeding with excavation. Casing or drilling slurry may need to be used while drilling the soldier piles.

8.6.2 Soil Nails

On the basis of the soil and bedrock conditions at the site, we conclude that an alternative shoring system to soldier-pile-and-lagging is a soil-nail wall. Soil-nail shoring system consists of reinforcing bars, which are grouted in predrilled holes through the face of the excavation, and shotcrete facing. Several computer programs, such as SNAILZ (California Department of Transportation, 1999) and GoldNail (Golder Associates, 1996), are available for designing a soil-nail wall. For input parameters, we recommend the values presented in Table 5.

TABLE 5
Recommended Input Parameters for Design of A Soil-Nail Wall

Depth Below Ground Surface (feet)	Soil Type	Total Density (pcf)	Ultimate Soil-Nail Friction (psf)	Shear Strength Parameters	
				c^1 (psf)	ϕ^2 (deg)
0 to 7	Soil	130	1,000	200	30
7 to 14	Weathered Shale	140	3,000	200	35

Notes:
1. Cohesion intercept or undrained shear strength, without a safety factor.
2. Angle of internal friction, without a safety factor.

The soil-nail wall should be backdrained using prefabricated drainage panels between the nails. These panels should be at least 2 feet wide and conduct the water to either weap holes or an approved collection system at the base of the wall. Surcharge loads should be included in the design. The soil-nail wall should be designed with adequate factor of safety as discussed in Section 8.6.3.

8.6.3 Design Factor of Safety

In accordance with the FHWA manual on soil nail walls (2003), we recommend designing the soil-nail walls using the minimum safety factors listed in Table 6, below:

TABLE 6
Recommended Safety Factors for Design of Soil-Nail Walls

Failure Mode	Resisting Component	Minimum Safety Factor Static Condition Temporary Structure
External Global Stability	Final Condition	1.35
	Interim Condition	1.25
Internal Stability	Grout-Soil Bond Strength	2.0
	Bar Tensile Strength	1.8
Shotcrete Facing	Punching Shear	1.35

Notes:

Interim condition corresponds to the case where temporary excavation lifts are unsupported for up to 24 hours before nails are installed.

8.6.4 Soil Nail Testing

Test nails should be installed using the same equipment, method, and hole diameter as planned for the production nails. Verification and proof tests should be performed. Verification tests are performed prior to production nail installation to verify the pullout resistance (bond strength) value used in design. Two verification tests should be performed for each soil type assumed in design. Proof tests are performed during construction to verify that the contractor's procedure remains the same or that the nails are not installed in a soil type not tested during the verification stage testing. At least five percent of the production nails should be proof tested.

Tests should be performed on production or sacrificial nails to a test load corresponding to the 100 and 75 percent of the ultimate pullout resistance value used in the design for verification and proof test, respectively. Test nails should have at least one foot of unbonded length and 10 feet of bond length. The nail bar grade and size should be designed such that the bar stress does not exceed 80 percent of its ultimate strength during testing.

In the verification and proof tests, the load should be applied to the nails in 8 and 6 increments, respectively. The maximum test load should be held for a minimum of 10 minutes; the movements of the nails should be recorded at 0, 1, 2, 3, 4, 5, 6, and 10 minutes. If the difference in movement between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the test results and determine whether the test nail performance is acceptable. Generally, a test with a ten-minute hold is acceptable if the nail carries the maximum test load with less than 0.04 inch movement between one and 10 minutes. A test with a 60-minute hold is acceptable if the nail carries the maximum test load with less than 0.08 inch movement between six and 60 minutes.

8.7 Tieback and Tiedown Anchor Testing

Each tieback/tiedown should be tested. The maximum test load should not exceed 80 percent of the yield strength of the tendons or bars. The movement of each tieback/tiedown should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing.

The movement of each tieback or tiedown should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing. The performance test is used to verify the capacity and the load-deformation behavior of the tiebacks/tiedown. It is also used to separate and identify the causes of movement, and to check that the designed unbonded length has been established. In the performance test, the load is applied to the tieback in several cycles of incremental loading and unloading. During the test, the tieback/tiedown load and movement are measured. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

A proof test is a simple test used to measure the total movement of the tieback/tiedown during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the test results and determine whether the tiebacks/tiedown are acceptable. A performance- or proof-tested tieback or tiedown with a ten-minute hold is acceptable if it carries the maximum test load with less than 0.04 inch movement between one and 10 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with less than 0.08 inch movement between six and 60 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. Tiebacks that failed to meet the first criterion will be assigned a reduced capacity.

If the total movement of the tiebacks at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the unbonded length, the contractor should replace the tiebacks.

8.8 Temporary Slopes

Inclinations of temporary slopes should not exceed those specified in local, state or federal safety regulations. Specifically, the requirements of the current OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926 should be followed. We recommend the contractor design temporary construction slopes to conform to the OSHA's "Guidelines for Excavations and Temporary Slopes." Temporary slope inclinations should be determined by the Contractor or responsible subcontractor based on the subsurface conditions exposed at the time of construction.

If temporary slopes are open for extended periods of time, exposure to weathering and rain could result in sloughing and erosion. Vehicles and other surcharge loads should be kept at least 10 feet away from the tops of temporary slopes and the slopes be protected from excessive drying and/or saturation during construction.

Using OSHA's classifications, the soil at the site can be considered as Type C. The maximum allowable slope inclination for Type C soils is 1½:1 (horizontal to vertical). Our preliminary classification for onsite fill is based on the materials encountered in widely spaced borings and our observations during installation of the existing shoring system. The contractor should confirm similar conditions exist throughout the proposed excavation area. If during construction different subsurface conditions are encountered, we recommend that we be contacted immediately to evaluate these conditions. Using OSHA's classifications, the rock can be considered a Type A; the maximum allowable slope inclination for Type A is ¾:1.

8.9 Construction Monitoring

The contractor should establish survey points on the shoring and on adjacent improvements within 50 feet of the excavation perimeter prior to the start of excavation. These survey points should be used to monitor the vertical and horizontal movements of the shoring and surrounding improvements during construction. The horizontal displacements for the soil/rock nailing shoring system should be monitored; monitoring points should be installed every 40 feet. In addition, a thorough crack survey of the buildings within 50 feet of the top of the soil/rock nailed wall should be performed by the project surveyor prior to the start of construction and immediately after its completion.

8.10 Seismic Design

Based on the soil and rock conditions at the site, for seismic design in accordance with the provisions of 2007 California Building Code (CBC) we recommend the following:

- Maximum Considered Earthquake (MCE) S_s and S_1 of 1.84g and 0.95g, respectively.
- Site Class C
- Site Coefficients FA and FV of 1.0 and 1.3
- Maximum Considered Earthquake (MCE) spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.84g and 1.24g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.23g and 0.83g, respectively.

9.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, Treadwell & Rollo should review the project plans and specifications to check their conformance with the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing services for the following:

- foundation subgrade to check for proper bearing and cleanout
- floor slab preparation
- tiedown installation (if any)
- shoring installation, including soldier piles and tiebacks or soil nails,
- site preparation and fill placement

These observations will allow us to compare actual with anticipated soil/rock conditions and to check the contractor's work conforms with the geotechnical aspects of the plans and specifications. Furthermore, we should test compaction of fill and utility backfill.

10.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited engineering studies based on our interpretation of the geotechnical conditions existing at the time of the investigation. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Treadwell & Rollo, Inc. should be notified to make supplemental recommendations, if necessary.

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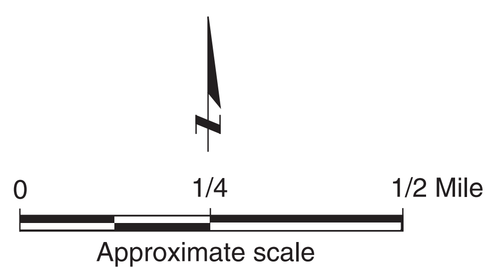
FIGURES

Base map: Thomas Guide
San Francisco County
1999



VIVARIUM REPLACEMENT AND EXPANSION
Veterans Affairs San Francisco, California

Treadwell&Rollo

SITE LOCATION MAP

Date 08/17/10	Project No. 5106.01	Figure 1

EXPLANATION

B-1 — Approximate location of boring by Treadwell & Rollo, Inc., 23 July 2010

17 — Building Number

Approximate location of proposed Vivarium

0 — 40 Feet

Approximate Scale

Reference: Base map from a drawing titled "Topo.dwg", by Sandis e-mailed to Treadwell & Rollo on 5 August 2010.

R:\Trgraphics\5100's\5106.01\5106.01 Site Plan 2.dwg 9/21/10

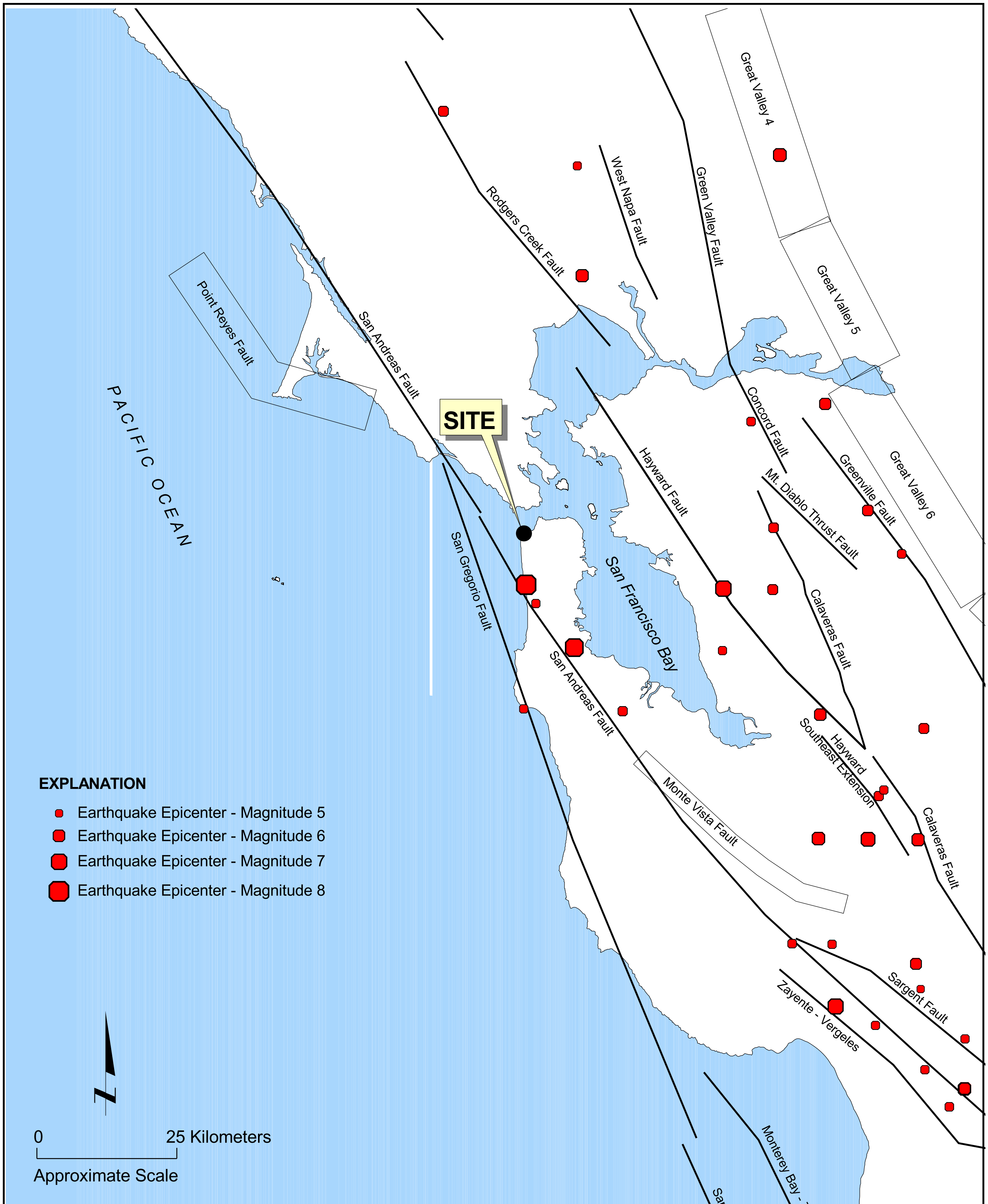
VIVARIUM REPLACEMENT AND EXPANSION

Veterans Affairs San Francisco, California

SITE PLAN

Date 09/21/10	Project No. 5106.01	Figure 2

Treadwell&Rollo



NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

VIVARIUM REPLACEMENT AND EXPANSION
Veterans Affairs San Francisco, California

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MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Date: 08/17/10	Project No. 5106.01	Figure 3

I **Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.**
Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II **Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.**
As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III **Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.**
Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV **Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.**
Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V **Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.**
Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI **Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.**
Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII **Frightens everyone. General alarm, and everyone runs outdoors.**
People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII **General fright, and alarm approaches panic.**
Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

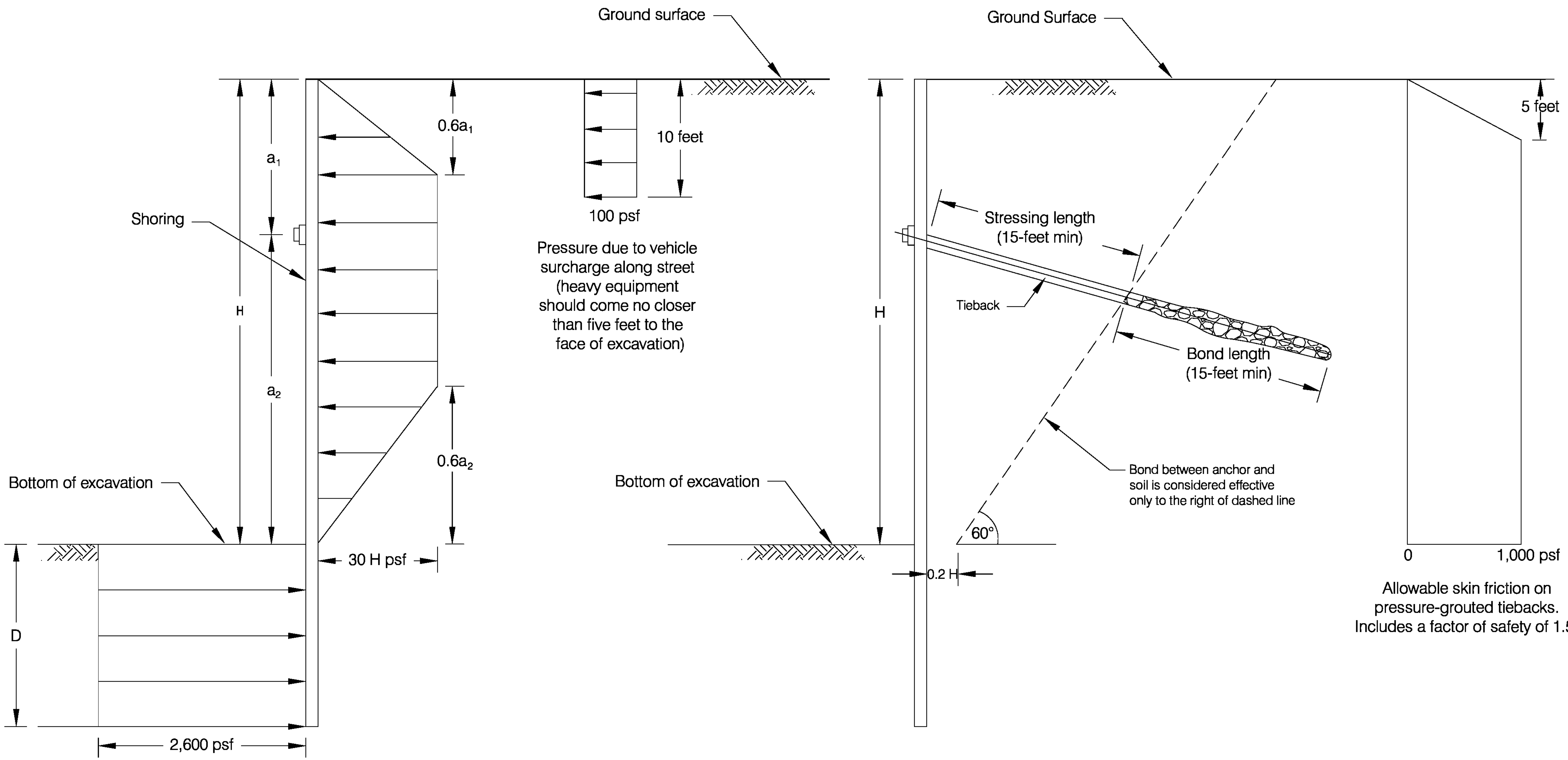
IX **Panic is general.**
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X **Panic is general.**
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI **Panic is general.**
Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII **Panic is general.**
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

VIVARIUM REPLACEMENT AND EXPANSION Veterans Affairs San Francisco, California	**MODIFIED MERCALLI INTENSITY SCALE**
Treadwell&Rollo	Date 08/17/10 | Project No. 5106.01 | Figure 4



NOT TO SCALE

Notes:
1. Passive pressures include a factor of safety of approximately 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.

VIVARIUM REPLACEMENT AND EXPANSION
Veterans Affairs San Francisco, California

DESIGN PARAMETERS FOR SOLDIER-PILE-AND-LAGGING TEMPORARY SHORING SYSTEM

Date 08/20/10	Project No. 5106.01	Figure 5

Treadwell&Rollo

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APPENDIX A

Logs of Test Borings

PROJECT: **VIVARIUM REPLACEMENT AND EXPANSION**
Veterans Affairs San Francisco, California

Log of Boring B-1

Boring location: See Site Plan, Figure 2
Date started: 7/23/10 | Date finished: 7/23/10
Drilling method: Minute Man
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Pulley
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: R. Severn

LABORATORY TEST DATA

Ground Surface Elevation: 351.1 feet[2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.5				SM	SILTY SAND (SM) brown, moist, with organics (FILL)						
0.5–4.3	S&H	20 15 17	19	CL	SANDY CLAY (CL) brown with yellow-brown mottling, very stiff, moist, with fine to medium gravel (FILL)						
4.3–4.7					2-3-inch Concrete (FILL)						
4.7–6	S&H	18 23 17	24	CL	CLAY (CL) olive-brown, very stiff, moist, with shale inclusions LL = 47, PI = 26		400	1,970		15.5	117
6–8	SPT	15 18 21	39		SANDSTONE/SHALE soft, plastic, deep weathering (BEDROCK)						
8–9.5	SPT	21 27 35	62		friable (BEDROCK)						

Boring terminated at a depth of 9.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on City and County of San Francisco datum.

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Project No.: 5106.01 | Figure: A-1

TEST GEOTECH LOG 510601.GPJ TR.GDT 8/26/10

PROJECT:	**VIVARIUM REPLACEMENT AND EXPANSION** Veterans Affairs San Francisco, California	**Log of Boring B-2** PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 7/23/10 | Date finished: 7/23/10

Drilling method: Minute Man

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Pulley

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: R. Severn

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 339.6 feet[2])		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					SM	SILTY SAND (SM) brown, moist, with organics	FILL						
1	DIST				CL	CLAY (CL) brown with trace yellow-brown, moist, with red gravel, asphalt fragments, serpentine, and trace organics	FILL						
2	S&H		35 50/3"	30/3"		LL = 25, PI = 9	BEDROCK					7.6	122
3	SPT		50/4"	50/4"		SANDSTONE low hardness, weak, deep weathering	BEDROCK						
4													
5													
6													
7													
8													
9													
10													
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													

LABORATORY TEST DATA (columns: Type of Strength Test through Dry Density)

Boring terminated at a depth of 3.1 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on City and County of San Francisco datum.

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Project No.: 5106.01 | Figure: A-2

PROJECT:	VIVARIUM REPLACEMENT AND EXPANSION Veterans Affairs San Francisco, California	Log of Boring B-3 PAGE 1 OF 1

Boring location:	See Site Plan, Figure 2
Date started:	7/23/10
Date finished:	7/23/10
Drilling method:	Minute Man
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Rope & Pulley
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	R. Severn

Ground Surface Elevation: 345.0 feet[2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0				SM	SILTY SAND (SM) brown, moist, with organics	FILL						
1				CL	SANDY CLAY with GRAVEL (CL) brown, gray and yellow-brown, moist, fine to coarse gravel	FILL						
3	S&H	3 4 5	5	SP	SAND (SP) brown, loose, moist, with silt	FILL					7.4	102
3.3					SHALE crushed, soft, plastic, deep weathering, sheared shale LL = 40, PI = 22	BEDROCK						
6	SPT	10 14 17	31			BEDROCK						
8	SPT	8 10 17	27			BEDROCK						
11	SPT	12 14 19	33		friable	BEDROCK						
15	SPT	10 11 20	31		low hardness	BEDROCK						
18	SPT	20 24 50/4"	74/ 10"			BEDROCK						

Boring terminated at a depth of 18.3 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on City and County of San Francisco datum.

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Project No.: 5106.01
Figure: A-3

PROJECT: **VIVARIUM REPLACEMENT AND EXPANSION**
Veterans Affairs San Francisco, California

Log of Boring B-4

Boring location: See Site Plan, Figure 2
Date started: 7/23/10 | Date finished: 7/23/10
Drilling method: Minute Man
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Pulley
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: R. Severn

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 341.7 feet[2])		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					GP	GRAVEL (GP) gray and light brown, moist	FILL						
1	DIST				CL/ SC	SANDY CLAY (CL)/ CLAYEY SAND (SC) dark brown, brown, gray and yellow brown, moist	FILL						
2	S&H SPT SPT		50/3" 50/1" 50/0"	30/3" 50/1" 50/0"		SANDSTONE moderately hard, strong, moderate weathering	BEDROCK						
3													
4													
5													
6													
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20													

Boring terminated at a depth of 2.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on City and County of San Francisco datum.

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Project No.: 5106.01 | Figure: A-4

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes: U.S. Standard Sieve Size	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample, hand auger

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

Sonic

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V CONSOLIDATION OF SEDIMENTARY ROCKS: usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

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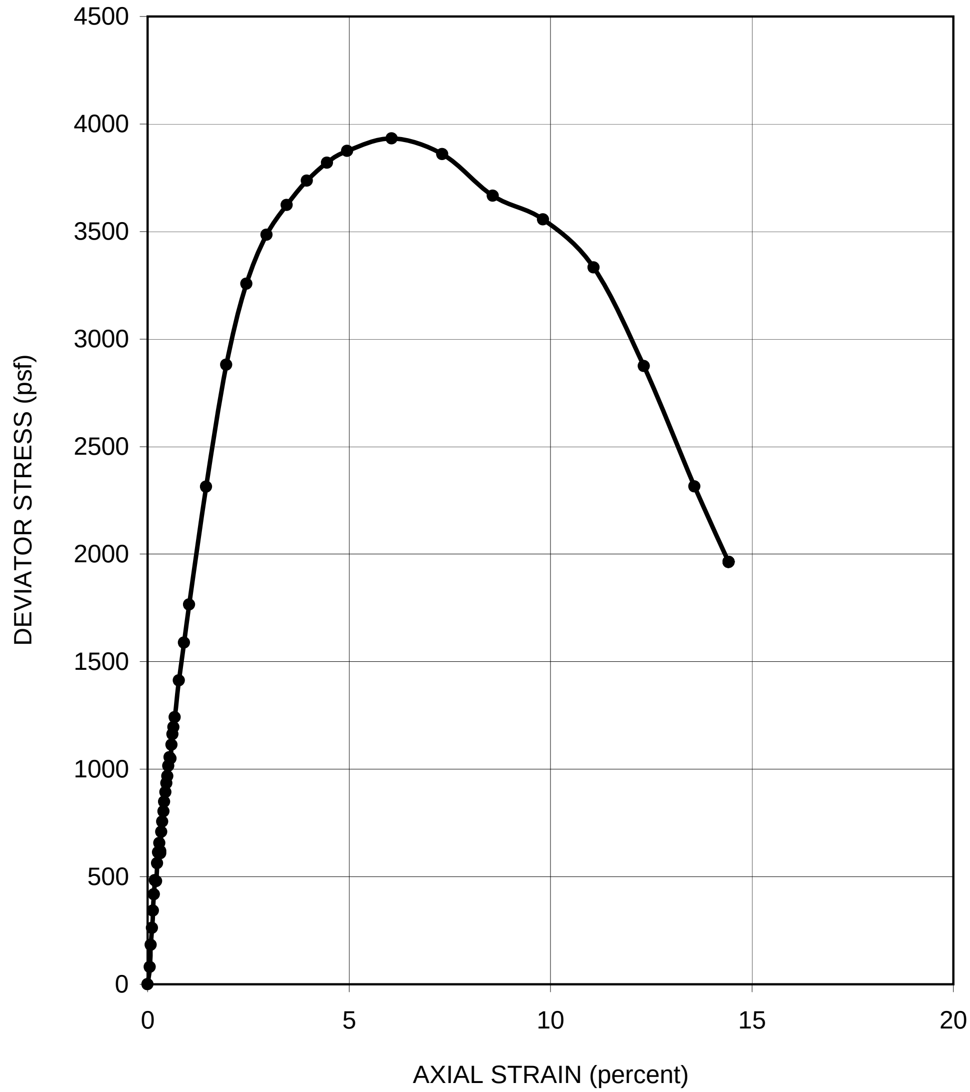
APPENDIX B

Laboratory Test Results



SAMPLER TYPE	Sprague & Henwood		SHEAR STRENGTH	1,970	psf
DIAMETER (in.) 2.4	HEIGHT (in.) 5.6		STRAIN AT FAILURE	6.1	%
MOISTURE CONTENT	15.5	%	CONFINING PRESSURE	400	psf
DRY DENSITY	117	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CL), olive-brown			SOURCE	B-1 @ 5.5 feet

VIVARIUM REPLACEMENT AND EXPANSION
Veterans Affairs San Francisco, California

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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 08/26/10 | Project No. 5106.01 | Figure B-2

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APPENDIX C

Corrosivity Test Results

ETS
Environmental Technical Services
975 Transport Way, Suite 2
Petaluma, CA 94954
(707) 778-9605/FAX 778-9612
-Soil, Water & Air Testing & Monitoring
-Analytical Labs
-Technical Support
Serving people and the environment so that both benefit.

COMPANY:	Treadwell & Rollo, 555 Montgomery Street, Suite 1300, San Freancisco, Ca 94111		ANALYST(S)	SUPERVISOR
ATTN:	Richard Rodgers	DATE of	D. Salinas	D. Jacobson
JOB SITE:	San Francisco VA, San Francisco, California	DATE RECEIVED / COMPLETION	S. Santos	LAB DIRECTOR
JOB #:	5106.01	8/4/2010 / 8/16/2010		G.S. Conrad PhD

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
04078-1	SFVA1/SF	B-1-2 @ 2.5'	**7.04**	**1,300**	**[770]**	**75**	**39**
04078-2	SFVA2/SF	B-2 2 2.0'	**6.83**	**1,520**	**[660]**	**6**	**27**
04078-3	SFVA3/SF	B-3 @ 7.0'	**7.73**	**1,000**	**[1000]**	**87**	**75**
Method	Detection	Limits --->	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
04078-1	SFVA1/SF	B-1-2 @ 2.5'				**+278.4**	
04078-2	SFVA2/SF	B-2 2 2.0'				**+304.6**	
04078-3	SFVA3/SF	B-3 @ 7.0'				**+261.1**	
Method	Detection	Limits --->	---	0.1	0.1	1	0.1

** COMMENTS **

Resistivities range from just 1,000 to >1,500 ohm-cm, i.e., very low to mediocre; soil reactions (i.e., pHs) range from very mildly acidic to mildly alkaline; sulfates and chlorides are low; soils are only mildly to very mildly reduced. The standard CalTrans times to perforation for these soils are as follows: for SFVA1 and 18 ga steel the time is ≈19.5 yrs, and for 12 ga it goes up to ≈43 yrs; for SFVA2 perf times are at 17.5 yrs for 18 ga & 38.5 yrs for 12 ga; and for SFVA3 the respective times are <25 yrs, & <55 yrs. For gray and mild steels the calculated average pitting rate for SFVA1 is @ ≈0.135 mm/yr, thus pitting to 2 mm depth is <15 yrs, and to a 4 mm depth is >30 yrs; for SFVA2 the rate is ≈0.11 mm/yr, thus the 2 mm depth time is ≈18 yrs, and the 4 mm depth time is >36 yrs; and for SFVA3 the rate is ≈0.12 mm/yr, thus the 2 mm depth time is ≈16.7 yrs, the 4 mm depth time is >33 yrs. Chloride level is low enough that it should not have any significant corrosion impact on concrete steel reinforcement; and sulfate is so low that there should be no significant adverse impact on concrete, cements, grouts and mortars. Soil redoxes do not appear to be an issue as they are mild to very mild. The SFVA1 and SFVA2 soils could benefit some from alkaline treatment in that raising their pH to the 7.5-8.5 range would increase the 18 ga times to perf for SFVA1 to <28 yrs, and for SFVA2 to >29 yrs; however, the pitting rates would only decline to ≈0.09 mm/yr for SFVA1 and to 0.1 mm/yr for SFVA2, thus putting the respective 2 mm depth times only up to ≈22 yrs, and ≈20 yrs. Otherwise, metals longevity in these soils could be improved by upgrading (e.g. increased gauge or more resistant steels, etc.); and/or using cathodic protection (which would require a nominal number and size of sacrificial anodes plus a moderate impressed current) along with coating or wrapping the steel; other alternatives include increased/specialized engineering fill, or use of plastic (esp. HDPE), fiberglass or concrete pipe, etc. Last, standard concrete mixes should be fine in these soils based on these results.

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

DISTRIBUTION

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QUALITY CONTROL REVIEWER:

Hadi J. Yap

Hadi J. Yap, PhD
Geotechnical Engineer