INFORMATION HANDOUT

For Contract No. 04-014104
At 04-Ala-5785

Identified by
Project ID 0413000133

MATERIALS INFORMATION

Foundation Recommendation dated October 17, 2013

State of California
DEPARTMENT OF TRANSPORTATION

California State Transportation Agency

M e m o r a n d u m

Flex your power!
Be energy efficient!

To: MR. SEAN SAMUEL
Branch Chief
Structural Design - Section 2
Office of Transportation Architecture
Design South – Special Project
Division of Engineering Services

Date: October 17, 2013

Attention: Justin Uyehara

File: 04-ALA-80- PM 2.2/2.5
Efis # 0413000133
SFOBB Maintenance Complex
Warehouse
Bridge No. 58M5709

From: CAROLINE CHEN
Transportation Engineer
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

MAHMOOD MOMENZADEH
Chief, Branch C
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

Subject: **FOUNDATION RECOMMENDATIONS**

This memorandum provides foundation recommendations for the construction of two maintenance warehouse buildings at the south side of the San Francisco-Oakland Bay Bridge (SFOBB) toll plaza, as Phase II of the SFOBB Maintenance Complex project. Phase I consists of Maintenance Buildings, a wash rack, a fuel island, and material storage bins. Figure 1 shows the approximate location of the project. Currently, Phase I is under construction. The site of the proposed warehouses is intact and covered by asphalt concrete (AC) pavement. The foundation will consist of slab on grade with grade beams supported on piles.

SITE GEOLOGY

A geology report was prepared by Rifaat Nashed, Caltrans Geotechnical Design West, dated May 14, 2013. The following paragraphs are part of the referenced geology report:

"The project area is entirely covered by artificial fill. This fill is underlain by water-saturated, soft Young Bay Mud which becomes stiffer with depth. The Bay Mud is in turn underlain by Merrit Sand, which is described as late Pleistocene dune and beach sands. Underlying the Merritt Sand layer is Old Bay Mud consisting of stiff to hard layers of marine clay. In some areas, the Merrit Sand layer has been eroded away and is absent, and Young Bay Mud directly overlies the Old Bay Mud. In some areas the upper part of the Young Bay Mud includes thin layers of fine to medium -grained sand alternating with silty clay.

According to the USGS map, the project area is entirely covered by Artificial fill over marine and marsh deposits of Quaternary age (Qmf). This geologic unit contains mud, including much organic material, silty mud, silt, and sand; very soft to soft where wet and overlain by artificial fill." Geological Map provided in the geology report is presented on Figure 2.

SUBSURFACE CONDITIONS

Field investigation was conducted by the Office of Geotechnical Design West in May and August 2011 for the proposed new maintenance complex which includes several buildings and facilities. A list of the Cone Penetration Tests (CPTs) and rotary wash borings (SPTs) that are applicable to the proposed warehouse buildings is summarized and presented in Table 1.

Based on these borings, in the upper 10 to 15 ft, the site encountered fill consisting of medium dense to dense clayey sand and clayey gravel with sand. Beneath the fill very soft to firm clay (Young Bay Mud) was encountered to elevations ranging from -65 to -80 ft. The Young Bay Mud was underlain by stiff to very stiff clay (Old Bay Mud) to the maximum exploration depth.

Groundwater was encountered during the investigation at approximately 5.5 ft to 7.0 ft depths below existing ground surface in the elevation range of 0.1 ft to 1.7 ft. The groundwater elevations observed from the CPTs in the most recent Caltrans field investigation may not be accurate due to the CPT hole caving after the CPT probe was removed. Some of the shallow boreholes drilled by Environmental Division of Caltrans District 4 indicated groundwater depth as shallow as 4 ft below the existing ground surface. The site is adjacent to the San Francisco Bay. The groundwater is expected to be influenced by marine environment and tidal impacts.

During the early stages of construction of the SFOBB approach, the existing ripraps were subsequently buried and were not identified on the contract plans. The buried ripraps were found within footprints of the new Tow Services Building site, which is about 2000 ft northwest to this site. The thickness of the riprap varies from 4 ft to 18 ft. Historical aerial photos taken in 1931, 1939, and 2005 were superimposed in the effort to find out the possibility of buried riprap in this site, as presented in Figure 3. It appears the site is located within the footprint of the land that exists prior to 1931. The land is overlain by fill with varying thickness described in the above section. The chance of ripraps presence is considered low.

Numerous AC and PCC Concrete overlays that vary from 5 ft to 10 ft in depth were also encountered during the construction of the new Tow Service Building in year 2010. During the construction of the Phase I including Maintenance Building, Wash Rack and Fuel Island initiated in the summer of 2013, up to 4 ft thick 3" minus or ¾" minus ballast rocks were encountered over the northern foundation excavation area of Maintenance building and almost over entire area of two other facilities. A concrete pad also encountered within the excavation area of Maintenance Building. Locations of 7 piles at Maintenance Buildings and 4 at Washrack were

adjusted due to encountering existing concrete and/or wood piles. The depth of these obstacles was about 6 to 8 ft below existing ground surface, except for one at about 10 ft. It is very likely the same site conditions will be encountered during the construction of these proposed structures.

To identify the buried object and possible ripraps for the new maintenance complex, geophysics survey including performing Electrical Resistivity and Magnetometers testing over the site area were conducted by Caltrans Geophysics Branch. Report prepared by Momoh Mallah, Caltrans Geophysics Branch, dated July 14, 2011, summarizes the results and interpretation of a nondestructive electromagnetic (EM) conductivity survey. This report is attached to this foundation report as Appendix A. The report identifies an anomalous area and is interpreted as an area of sand and gravel. The obstacles encountered as mentioned above were not clearly shown on the geophysical survey maps.

Ground Penetrating Radar (GPR) survey was performed to designate the location of underground utilities and possible buried objects in year 2011 by Caltrans District 4 and the contracted consultants. A report, if available, may be obtained from District 4 Surveys Coordination Branch. An additional GPR was conducted in 2013 to locate and show the gas line on the project plan.

Table 1 Summary of CPTs and SPTs conducted by Caltrans in May and August 2011

Boring No.	**CPT/Boring Depth (ft)**	**Top of CPT/Boring Elevation (ft)**	**Bottom Elevation (ft)**	**Groundwater (1) Depth (ft)**	**Groundwater Elevation (ft)**
CPT-11-009	101.5	7.2	-94.3	5.7	1.4
CPT-11-010	101.2	7.2	-94.0	5.5	1.7
CPT-11-011	101.1	7.1	-94.0	5.6	1.5
CPT-11-012	104.6	7.1	-97.5	5.8	1.3
RW-11-008	121.5	7.3	-114.2	6.0	1.3
RW-11-019	121.5	7.1	-114.4	7.0	0.1

Note: 1. These depths were recorded at time of drilling and actual depth could be higher or lower.

SITE CONTAMINATION ASSESSMENT

Certain area of the site is contaminated and was investigated by Caltrans Hazard Group. A shallow investigation was conducted by Caltrans Environmental Division in April 2011, contracting with Geocon Consultants, Inc.. The investigation and findings are included in the Site Investigation Report prepared by Geocon Consultants, Inc., dated September, 2012.

SCOUR

Scour is not anticipated to be an issue for this site, since there is no watercourse running through the site.

CORROSIVITY

Corrosion test results are presented on Table 2. Test results from Boring B-21 are obtained from Geotechnical Engineering Investigation report prepared by Parikah Consultants, Inc., dated June 2007. Boring B-21 is approximately 500 ft west to the proposed Warehouse Building 1. The results indicate the site is considered corrosive based on the current Caltrans guidelines.

Table 2 Corrosivity Test Results

Boring No.	Depth (ft)	Soil Type	pH	Minimum Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)
RW-11-008	3-6	Clayey sand	6.89	2766	-	-
RW-11-019	3-5	Sand and silt	7.95	3480	-	-
B-21	20	Young Bay Mud	7.78	70	4759	613

SEISMICITY/LIQUEFACTION POTENTIAL

A Seismic Design Report (SDR) dated July 18, 2011 was prepared by Hossain Salimi from Office of Geotechnical Design West to provide seismic recommendations for the Phase I structures. The SDR is attached to this report as Appendix B. The SDR concluded that based on the 2007 investigation, the layers of loose to medium dense poorly graded sand/silty sand encountered blow the water table poses a moderate to high liquefaction potential during a strong seismic event.

FOUNDATION RECOMMENDATIONS

a) Mat Foundation

Mat foundation is not considered feasible option for the buildings due to the potential differential settlement caused by the following factors:

1) The ongoing settlement rate of the general site area is unknown;
2) The fill crust overlying the site has variable thickness, variable density, and unknown buried objects, which could produce uneven ground settlement under building loads and seismic loadings;
3) Softening of the very soft and thick Young Bay Mud at the site due to strong ground shaking under seismic events will cause additional settlement and ground deformation that is hard to predict accurately.

b) Driven Piles

Use of driven piles is a feasible foundation support option based on the site conditions and experience from the previous and ongoing Phase 1 Construction. Precast pre-stressed concrete driven piles Class 200 Alt "X" are proposed to support the proposed buildings. Pile recommendations are presented in Table 3.

Undersized drilling prior to pile installation to Elevation -5.0 ft through dense sand layer is required to avoid hard driving and minimize the vibration impact on the adjacent facilities. Pre-drilling through this layer will also minimize the downdrag force caused by liquefaction and thus the potential downdrag is ignored accordingly. Jetting shall not be used. Any gap remaining between the sides of the piles and pre-drilled hole shall be back filled with loose sand.

Table 3 Pile Data Table

Pile Type	Design Loading (kips)	Nominal Resistance (kips)		Design Tip Elevation (ft)	Specified Tip Elevation (ft)
		Compression	Tension		
Class 200 Alt "X"	200	400	160	-110.0(1) -95.0 (2) -33.0 (3)	-110.0

Notes: 1. Pile tip elevations are controlled by the following demands: (1) Compression; (2) Tension, (3) Lateral service load of 5 kips.
2. The cut-off elevation is assumed to be at Elevation 7.8 ft.
3. Undersized drilling to Elevation -5.0 ft through dense sand layer is required prior to pile installation.

We recommend that the depth of the grade beams shall be sized to stay above Elevation +3.0 ft to minimize the need for dewatering during their construction.

The site under the proposed buildings will be excavated to Elevation 5.8 ft and backfilled with imported borrow to the bottom of the gravel cushion according to the Plans for the contaminant removal. The imported borrow shall have Expansion Index (EI) 20 or less, and Plasticity Index (PI) less than 15. The maximum size of the backfill should be less than 1.5 inch. The backfill soil shall be compacted to minimum 90% with the exception of upper 2.0 ft which shall be compacted to 95% relative compaction. The fill shall be placed in maximal 8 inch thick lifts. The above recommendations are intended to minimize the potential expansion pressure on slabs and beams caused by the backfill material and provide a better drainage path.

The trench for the beams can be excavated in the compact fills with vertical sides no deeper than 4 ft and the concrete poured against the trench sides provided that there is no sloughing of the excavated trench side walls, the trench bottom is cleaned from the loose soil, debris/ slough material and concrete is poured as soon as possible after the trench is excavated and is ready for pour. Otherwise, the trench side shall be sloped no steeper than 1H:1V and the trench be wide enough to enable use of the manual tamper to compact the beams backfill. The beam backfill shall have Expansion Index (EI) 20 or less, and Plasticity Index (PI) less than 15. The fill shall be compacted in lift of 8 inch manually to 95% relative compaction.

A 12 inch 240 psi gas line is currently located at south to the warehouse buildings, with the closest distance about 10 ft from the southeast corner of the building. Based on the vibration attenuation study conducted during Phase I construction, we expect the peak particle velocity (PPV) of about 0.45 in/sec at the gas line section closest to the proposed warehouse, and about 0.30 in/sec where distance between gas line and the closest pile is 40 ft, during pile driving and earth work activities for the proposed Warehouse foundations. The contractor is responsible to maintain the summed PPV at all time during the construction below 0.25 in/sec by using an appropriate pile driving rated energy and suitable mitigation measures to protect the pipe from ground vibration impact to keep the vibration below 0.25 in/sec.

GROUND SETTLEMENT AND MITIGATION

It is anticipated that a substantial differential settlement will occur in the long term and under seismic events between the proposed building area which are supported on piles and the adjacent ground. The differential settlement is anticipated to be small for the ground area immediately around the perimeter of the building, and increase to the maximal over the distance 15 to 30 ft away from the building. The utility lines serving the buildings must have flexible joints that can accommodate up to 2 ft differential settlement. Site grades shall not be raised more than 6 inches from current level except the associated total and differential settlement can be accommodated by the buried utilities.

We recommend that all pipelines which rely on the gravity flow be backfilled with sand up to 4 inches above the pipeline crown, or groundwater level if dewatering is not used, whichever is higher, followed by the lightweight cellular concrete up to the bottom of the pavement structure. The cellular concrete shall have unit weight of no more than 45 pcf. If cellular concrete becomes concern for maintenance, lightweight granular material can be used as backfill.

INDICATOR PILE PROGRAM

It is recommended to have an indicator pile program to verify the length of the piles and capacities, to establish the driving criteria, and to conduct vibration monitoring along the existing EBMUD outfall pipeline along Burma Road and the 12 inch 240 psi gas line mentioned above. The 108-inches concrete outfall pipe belonging to EBMUD is about 100 ft from the proposed building. One indicator piles at the design length are proposed at each of the two warehouse buildings. The exact location will be determined by the Engineer and approved by the Office of Geotechnical Design West.

VIBRATION MONITORING

Vibration impact on the EBMUD pipeline and the gas line during pile driving and earthwork activities along Burma Road is of concern. Vibration monitoring is required during driving of both indicator piles and production piles as well as during earthwork including excavation of existing fill and AC, replacement, and compaction of backfills. It is recommended to monitor the pipeline crown and the adjacent ground for vibration and movements frequently during the pile driving and the earthwork. Vibration cut-off element may be installed along the pipe if needed depending the vibration level and impact monitored during the indicator pile program. Based on the vibration monitoring for the current construction, a vibration limit (warning level) of 0.3 in/sec PPV shall be used for EBMUD pipe and 0.25 in/sec PPV for the gas line. Any construction activity that cause the vibration level reaches this level shall be stopped and be subject to review and modification.

CONSTRUCTION CONSIDERATIONS

- Piles may be unable to achieve the required capacity when driven to the specified tip elevation. If this happens during pile installation, piles should be allowed to set up for minimal 24 hours and then re-struck. We recommend that pile driving criteria be established for the proposed pile by using the pile indicator program as mentioned above similar to that used for the current construction.

- Stockpiling of the imported fill or any other material more than 2 ft in height shall be avoided within 50 ft of the propose buildings, existing facilities, or the locations with existence of underground utilities, due to potential consolidation settlement of Young Bay Mud.

- The hammer energy to be used for the production pile driving shall be established during the indicator pile program for the same driving equipment to achieve the capacity and not to induce compression and tensional stress in excess of the allowable values shown on the plans.

- As mentioned above one indicator piles at the design length are proposed at each of the two warehouse buildings. Pile Dynamic Analysis (PDA) shall be conducted for the initial drive and restrike near the specified pile tip elevations. PDA is required for both indicator piles as well as two production piles one at each of the proposed warehouses. The exact location will be determined by the Engineer and approved by the Office of Geotechnical Design West.

- Since significant length of piles is in soft clay, the vertical alignment of piles shall be well maintained during pile driving. Easy driving and tension in piles caused by easy driving should be anticipated and minimized by using the appropriate driving system.

- Pile splicing is not allowed.

- Groundwater is shallow and may be encountered during excavation for grade beam and slab construction. Dewatering and shoring may be needed.

- Underground obstruction such as paved Asphalt, PCC Concrete overlays, abandon utility boxes, and even local area of ripraps are very likely to encounter during pile driving and excavation. Use of sonic drilling for drilling prior to pile installation is an option the Contractor should consider where reinforced PCC slab and ripraps are encountered. Based on the current construction observation relocation of the piles at the location of obstructions may be required and shall be coordinated with the Structural Designer.

If you have any questions or need additional information, please contact Caroline Chen at 916-227-5386 or Mahmood Momenzadeh at 510-286-5732.

Attachments:

c: TPokrywka -Email
MMomenzadeh -Email
MMacaranas -Email
APang – Email
FThomas - Email
Geodog

CChen/MMomenzadeh/mm

REGISTERED PROFESSIONAL ENGINEER
CAROLINE JINXIA CHEN
No. C 62438
Exp. 9/30/2015
CIVIL
STATE OF CALIFORNIA

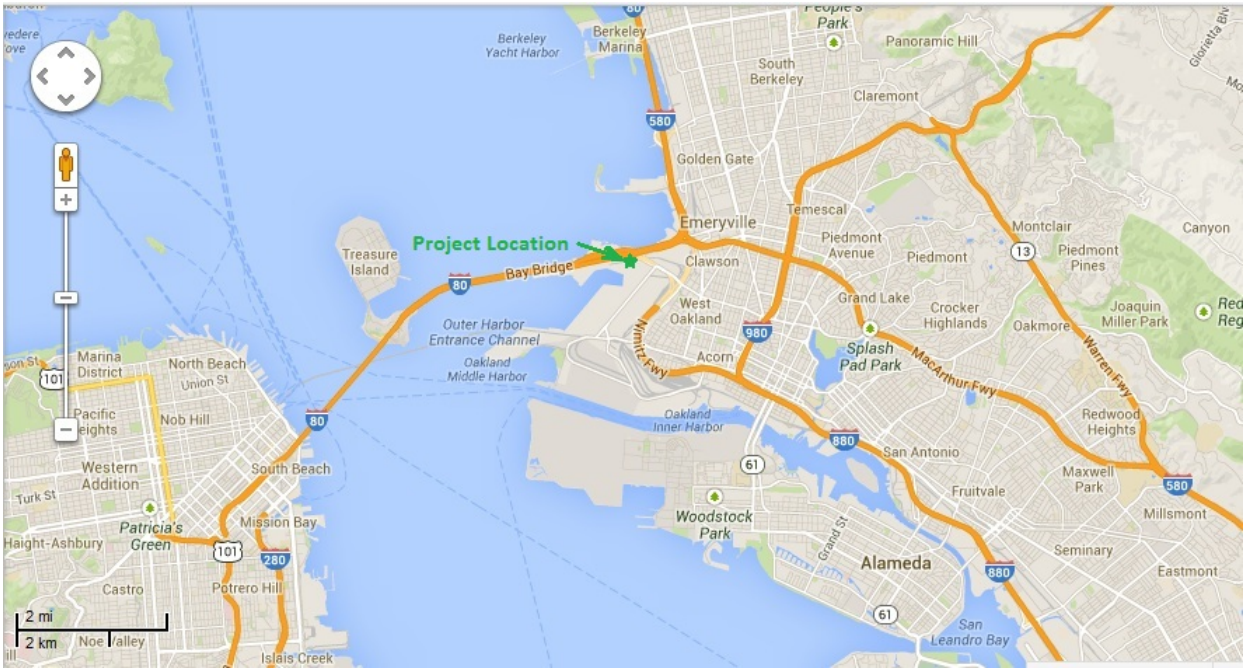


Figure 1 Location Map

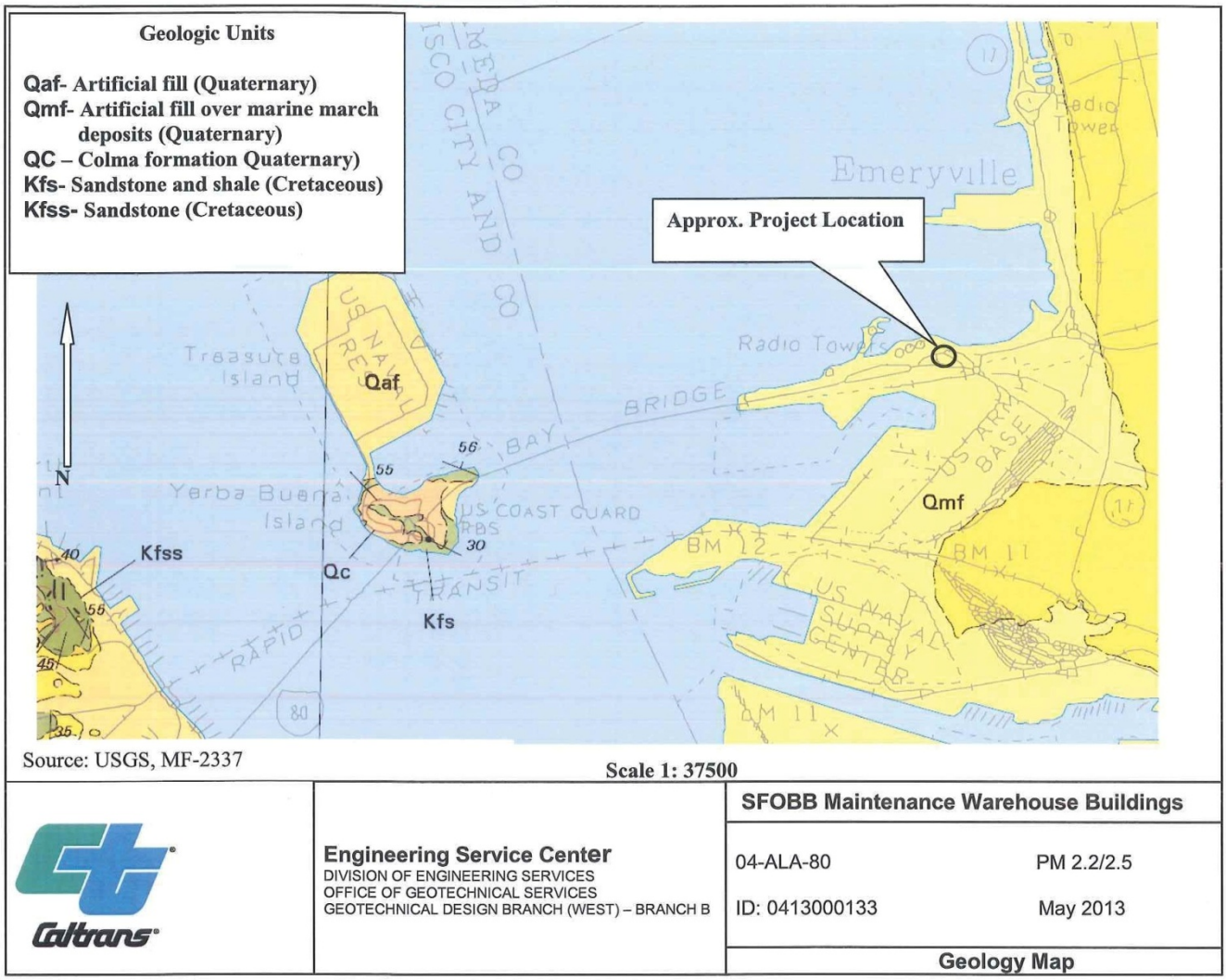


Figure 2 Geology Map

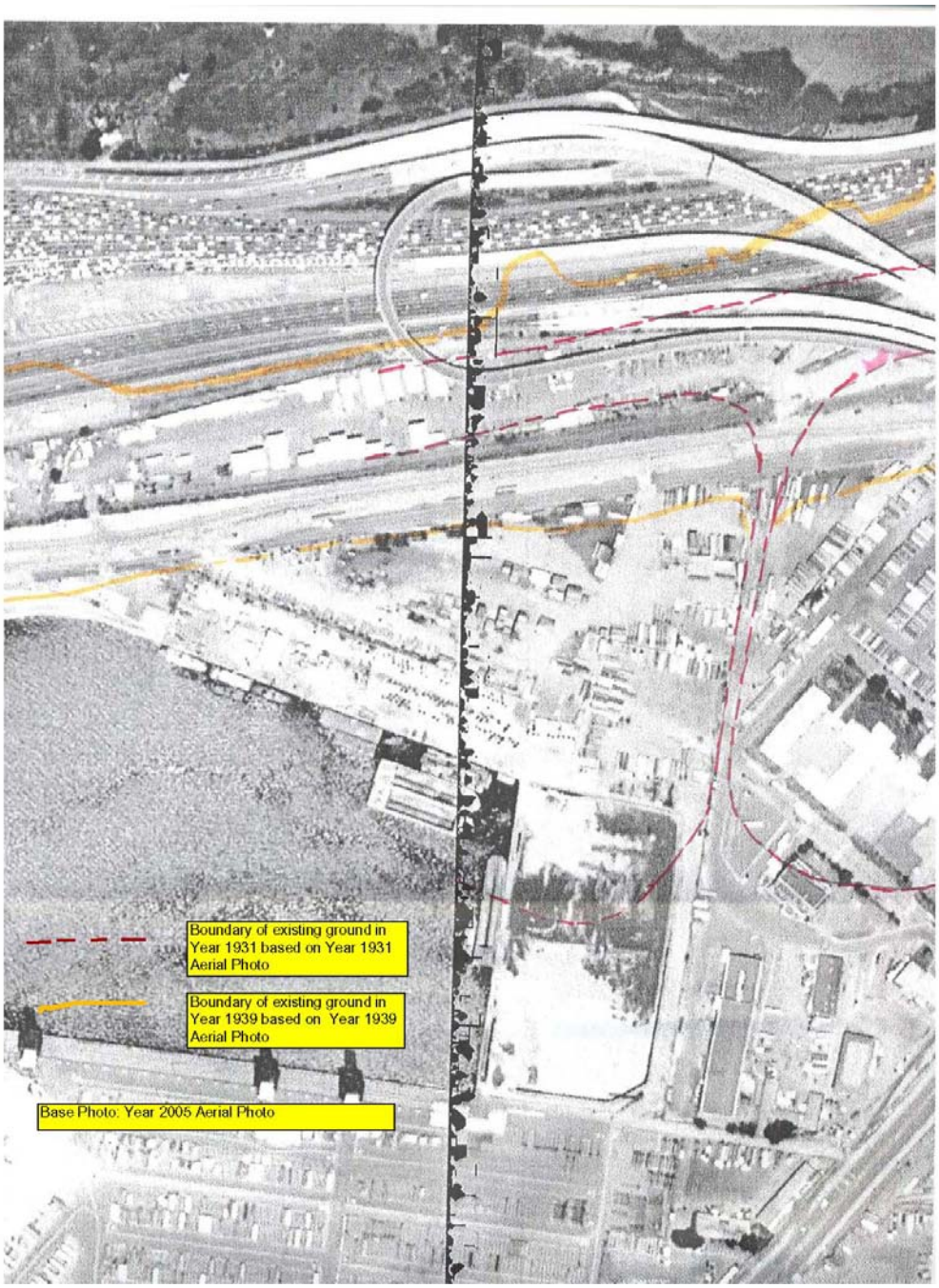


Figure 3 Superimposed Historical Aerial Photos

Appendix A

M e m o r a n d u m

Flex your power!
Be energy efficient!

To: MAHMOOD MOMENZADEH
Geotechnical Design West

Attention: David Nesbitt

Date: July 14, 2011
File: 04_ALA_80_2.0

Project: 0400000038

From: MOMOH MALLAH
Division of Engineering Services
Geotechnical Services-MS#5

Subject: Electromagnetic (EM) conductivity survey at SFOBB Maintenance Complex along State Route 80

Introduction

This report summarizes the results and interpretation of a nondestructive electromagnetic (EM) conductivity survey at the proposed SFOBB Maintenance Complex along State Route 80 in Alameda County. The survey was requested to assist in identifying buried rock rubble, related to the original bridge construction, that might impact pile installation for the proposed buildings at the site.

Results and Conclusions

Two figures with interpretations are presented. Figure 1 is a location map of the area. The table shows the coordinates (in UTM units) of the corners (1 through 7) of the surveyed area. Figure 2 is a map of the apparent resistivity of the area surveyed. Diagrams 1 through 3 (in Figure 2) have effective depth ranges of 2.2 m (7.22 feet), 4.2 m (13.78 feet) and 6.7 m (~ 22 feet) respectively. Corners marked A, B, C, and D (in diagram 1) encompass a high resistivity anomalous area. This is interpreted as an area of sand and gravel. The homogeneous area surrounding this anomaly is likely clayey sediments and bay mud.

The anomalous area ABCD (see Figure 2) outlined in blue is the area of interest that should be investigated further. Additional test borings are recommended to confirm this feature.

Data Acquisition and Processing

An EM system (CMD-Explorer) manufactured by GF Instruments was used to conduct this survey. Three effective depth-ranges 2.2 m (7.22 feet), 4.2 m (13.78 feet) and 6.7 m (~ 22 feet) were used. This system was coupled with a GPS unit for measurement location.

Effective depth range (EDR) is defined as the depth where instrument sensitivity drops below approximately 25%. The configuration we used was intended for deeper targets. The peak sensitivity for diagram 1 above EDR is about 0.5 m (20 inches), for diagram 2, it is about 1 m (3.3 feet), and for diagram 3, it is about 1.5 m (5 feet). However objects and materials within the EDR contribute to the conductivity measurement, resulting in an "averaged" response over the interval at each measuring point.

The EM system consists of a transmitter and receiver coil. The transmitter coil at the surface generates a sinusoidally-varying primary field which induces electrical currents within the ground below. This induced current generates a secondary EM field which is measured by the receiver coil. The measured secondary field is proportional to the ground conductivity. For this report, the inverse of conductivity (resistivity) was used for ease of presentation.

Data were transferred from the CMD-Explorer to a personal computer. A contouring program, Surfer 10, manufactured by Golden Software was used to generate the contour plots.

Thank you for the opportunity to work on this project. If you have any questions or need additional assistance, please call me at (916) 227-4400 or Bill Owen at (916) 227-0227.

Report by:

Momoh Mallah
Engineering Geologist
Geophysics and Geology Branch

Reviewed By:

William Owen, CEG 1735
Chief, Geophysics and Geology Branch

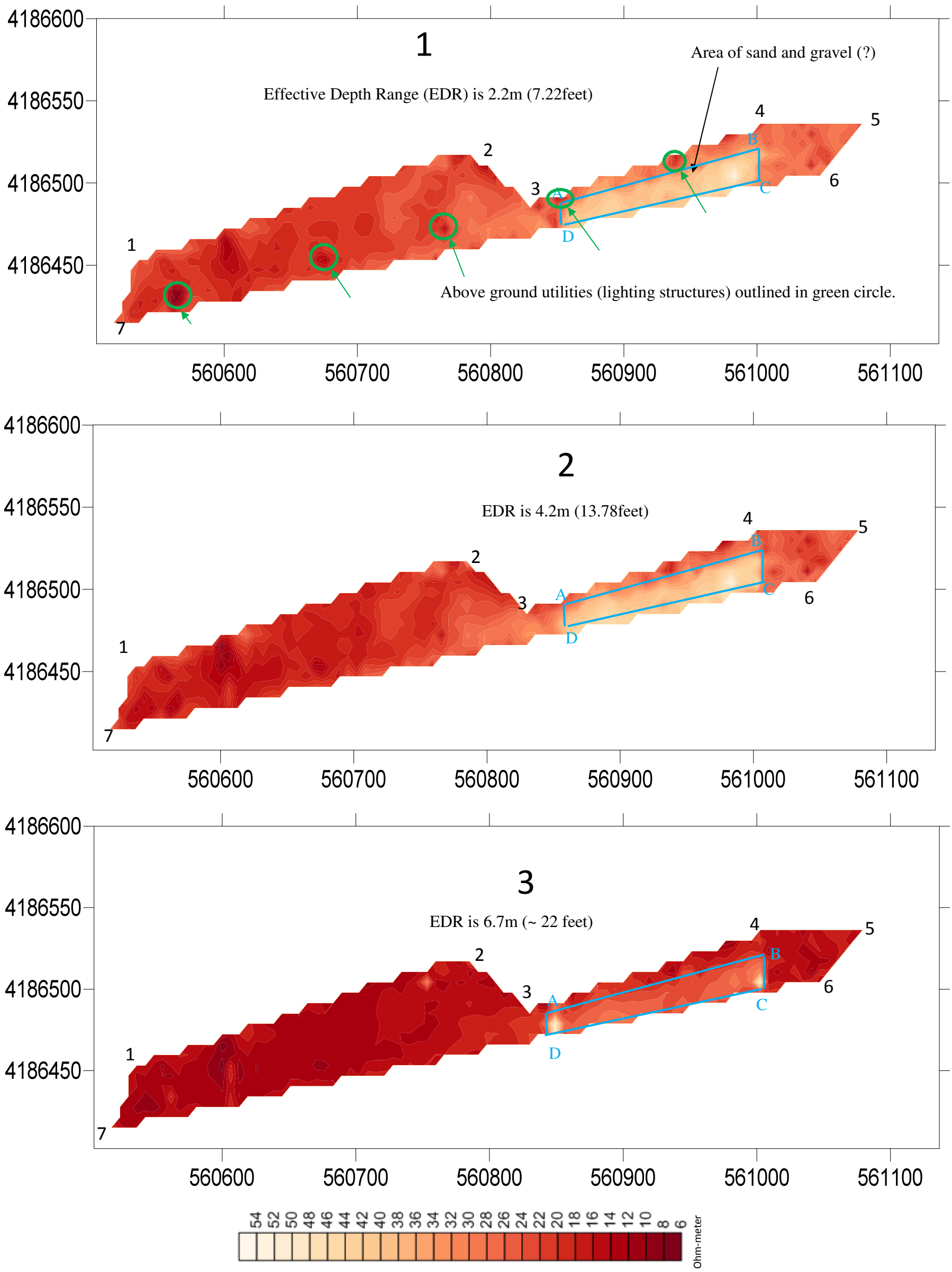
PROFESSIONAL GEOPHYSICIST
W.P. OWEN
No. 1031
Exp. 3/31/2012
STATE OF CALIFORNIA

Attachments: Figures 1, and 2

C: Project File: 04_ALA_80_2.0_2011_EM

Figure 1: Aerial map of surveyed area (outlined red) in relationship to I – 80
The UTM coordinates of the corners are shown in the table below.

CORNERS	X	Y
1	560531.5576	4186455.0779
2	560777.4905	4186523.4662
3	560830.5417	4186490.5715
4	560997.4288	4186536.1906
5	561085.4369	4186536.844
6	561050.4808	4186503.2973
7	560514.2824	4186410.5681



CORNERS	X	Y
A	560856.4	4186487.2
B	561002.7	4186523.3
C	560999.9	4186568.7
D	560854.6	4186342.8

Figure 2: A map of the apparent resistivity in ohm-meter (ohm-m) of the area surveyed. The corners marked 1, through 7 are the same as those in figure 1. The UTM coordinates of the anomalous area are shown in the table. The anomalous high resistivity area (outlined in blue) is interpreted as an area of sand and gravel. The area surrounding the anomaly is likely low-resistivity clays and estuarine bay mud.

Appendix B

Memorandum

Flex your power!
Be energy efficient!

To: MR. GORDON JEONG
Branch Chief
Design South-Special Projects

Date: July 18, 2011

File: 04-SF-ALA-80
0400000038
SFOBB Maintenance Complex

Attention: Mr. Eugene Wollak

From: HOSSAIN SALIMI
Senior Materials and Research Engineer
Division of Engineering Services
Geotechnical Services – MS-5
Office of Geotechnical Design-West

Subject: Seismic Design Recommendations

This memorandum is in response to your request dated February 17, 2011 and presents the Seismic Design Recommendations for the proposed SFOBB Maintenance Complex located just south of the Westbound Highway 80 Bay Bridge toll plaza in Alameda County.

Parikh Consultants, Inc had conducted a field investigation for this site, including various Cone Penetrometer Test (CPT) holes and wet rotary sample borings in 2007 for the Service Buildings proposed in this area. In addition, Caltrans personnel conducted a new field investigation including 15 CPT holes in the area of the proposed Maintenance Complex. According to these new CPT borings, the geology at this site mainly consists of soft to stiff clay with occasional layers of granular material. The water table based on the 2007 investigation is approximately 4 to 5 feet below ground. Based on this latest information, the following recommendations apply to the proposed complex.

According to the 2007 edition of the CBC, the Maximum Considered Earthquake Ground Motion for 0.2 Second and 1.0 second Spectral Response Acceleration, is 1.5 and 0.6, respectively (Figure 1613.5(3) and Figure 1613.5(4)), and the available geological data characterize the soil as type E (Table 1613.5.2). Based on the aforementioned data, the Site Coefficient for short and long period, S_{MS} and S_{M1} is 1.35 and 1.44, respectively. The Design Earthquake Spectral Acceleration for short period S_{DS} and long period S_{D1} is calculated to be 0.9 and 0.965, respectively, with T_s and T_o calculated to be 1.07 and 0.21

seconds, respectively. The Acceleration Response Spectrum for the site, based on these values and as specified in Figure 11.4-1 (page 115) is shown on the attached Figure 1.

It should also be noted that the review of the LOTBs from the 2007 investigation revealed locations where layers of loose to medium dense poorly graded sand/silty sand was encountered below the water table, posing a moderate to high liquefaction potential during a strong seismic event. Caroline Chen of the Office of Geotechnical Design-West was informed and will account for any liquefaction potential with her geotechnical analysis.

If there are any questions, please contact Hossain Salimi at (916) 227-7147.

Attachment

c: TPokrywka (OGD-W)
MMacaranes (OGD-W)
CChen (OGD-W)
Project file

REGISTERED PROFESSIONAL ENGINEER
HOSSAIN SALIMI
No. 54567
Exp. 12/2011
CIVIL
STATE OF CALIFORNIA

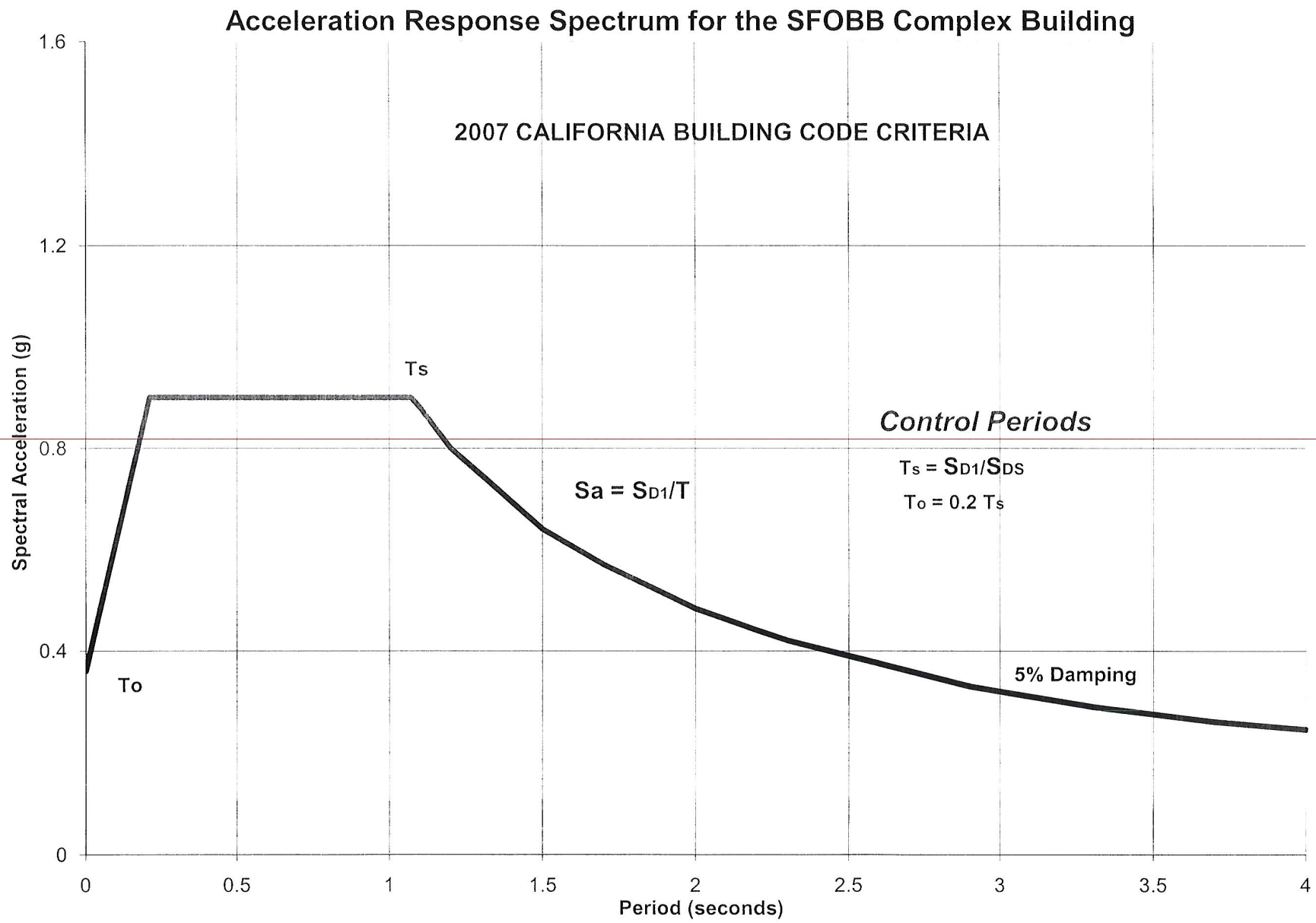

Acceleration Response Spectrum for the SFOBB Complex Building
2007 CALIFORNIA BUILDING CODE CRITERIA
Spectral Acceleration (g)
1.6
1.2
0.8
0.4
0
Ts
To
Control Periods
Ts = SD1/SDS
To = 0.2 Ts
Sa = SD1/T
5% Damping
0
0.5
1
1.5
2
2.5
3
3.5
4
Period (seconds)

Figure 1