GEOTECHNICAL INVESTIGATION
350 MISSION STREET
SAN FRANCISCO, CALIFORNIA

GLL Properties
1981 North Broadway, Suite 330
Walnut Creek, California 94956

28 June 2012
Project 730466502

Treadwell&Rollo
A LANGAN COMPANY

Treadwell&Rollo
A LANGAN COMPANY

ADDENDUM #
DEC 2 0 2012
SITE PERMIT ADDENDUM FOR CONSTRUCTION OF:

28 June 2012
Project 730466502

Mr. David Wall
GLL Properties
1981 North Broadway, Suite 330
Walnut Creek, California 94956

Subject: Geotechnical Investigation
350 Mission Street
San Francisco, California

Dear Mr. Wall:

Treadwell & Rollo is pleased to present this geotechnical investigation report for the proposed 350 Mission Street project in San Francisco. This report presents our geotechnical findings, conclusions and recommendations for the proposed development. Treadwell & Rollo, Inc. previously issued a draft report dated 21 August 2008; the recommendations presented herein supersede those and the draft report should be discarded. Additional copies have been distributed as indicated at the end of this report. This letter omits detailed findings, conclusions and recommendations; therefore, anyone relying on the report should read it in its entirety.

Subsurface conditions at the site consist of heterogeneous fill over Marine Deposits underlain by dense to very dense sand (Colma Sand), stiff clay (Old Bay Clay) and Franciscan Complex bedrock. Bedrock has previously been encountered in the vicinity at a depth of about 240 feet below the existing ground surface. The proposed development consists of a 30-story residential tower over four levels of below grade parking with the lowest parking level finished floor at Elevation -42 feet (San Francisco City Datum). The proposed building will encompass the entire parcel and will measure about 137 by 137 feet in plan dimension. We recommend the tower structure be supported on a mat foundation bearing on very dense Colma sand.

The recommendations contained in this report are based on a limited subsurface exploration program. Consequently, variations between expected and actual soil conditions may be found in localized areas during construction. We should be retained to observe site excavation and shoring, compaction of backfill, and installation of mat during which time we may make any changes to our recommendations, if necessary.

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

Sincerely yours,
TREADWELL & ROLLO, A LANGAN COMPANY

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TABLE OF CONTENTS

TABLE OF CONTENTS (CONT.)

LIST OF FIGURES

APPENDIX A

APPENDIX B

APPENDIX C

APPENDIX D

GEOTECHNICAL INVESTIGATION
350 MISSION STREET
San Francisco, California

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed development at 350 Mission Street in San Francisco, California. The project site is at the northeast[1] corner of Mission and Fremont Streets as shown on the Site Location Map, Figure 1. The site occupies Lot 19 of Assessor's Block No. 3710 and is bound by Mission Street to the south, Fremont Street to the west, a 34-story building to the north (45 Fremont Street), and a 24-story building to the east (50 Beale Street). The site is currently occupied by a four-story, reinforced concrete building with a single basement that extends beneath the street sidewalks. The existing building was constructed in the early 1920s and seismically upgraded in 1998.

Current plans are to demolish and remove the existing building and construct a new high-rise structure with underground parking. We understand the proposed structure will likely be constructed of reinforced concrete. Plans prepared by Skidmore, Owings & Merrill LLP (SOM), the project structural engineer and architect, dated 7 September 2011 (50% construction documents) show the proposed development consists of a 30-story residential tower over four levels of below grade parking. The proposed building will encompass the entire parcel and will measure about 137 by 137 feet in plan dimension. The proposed below grade parking levels will not extend beyond the property line. The plans show the top of slab elevation at the lowest parking level will be at Elevation -42 feet[2].

2.0 BACKGROUND

Treadwell & Rollo, Inc. performed a geotechnical investigation for seismic strengthening of the existing building and presented the results in a report dated 3 July 1997. The seismic upgrade was completed in 1998 and consisted of constructing shear walls supported by steel pipe piles. The result of this investigation and

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1 Assumed project north is along Fremont Street, toward Market Street.

2 Elevations reference San Francisco City datum (SFCD).

previous investigations in the vicinity were used in our geotechnical engineering studies and analyses to develop our conclusions and recommendations for the proposed development. A list of references used during our study is presented at the end of this report.

3.0 SCOPE OF SERVICES

Our original scope of services, which was outlined in our proposal dated 14 March 2008, consisted of reviewing existing subsurface information and performing engineering analysis to develop conclusions and recommendations regarding following:

- soil, bedrock and groundwater conditions
- the most appropriate foundation type for the proposed structure
- design criteria for the recommended foundation type
- estimates of foundation settlement
- site seismicity and seismic hazards, including potential for liquefaction, lateral spreading and seismically-induced ground settlement
- 2007 San Francisco Building Code (SFBC)
- soil profile type and seismic coefficients
- shoring design
- construction considerations

Our 2008 scope of services also included performing a site-specific seismic study for the site. Treadwell & Rollo, Inc. issued a draft report, dated 21 August 2008, based on this scope of services. This report finalizes the draft geotechnical report to address comments and design revisions/updated from the design team, as well as to address and to be in accordance with the current building code (2010 SFBC). This report also updates and revises the results of our site-specific seismic ground motion studies. The revisions to the draft report were made in accordance to the scope of services outlined in our 13 September 2011 budget amendment proposal.

4.0 PREVIOUS GEOTECHNICAL INVESTIGATIONS

Prior to performing geotechnical engineering analysis for the proposed development, we reviewed the results of previous investigations performed at the site and vicinity. A summary of these investigations is presented in this section of the report.

4.1 Geotechnical Borings Performed by Dames & Moore

Dames & Moore performed geotechnical investigations in the vicinity of the project site in 1966 and 1980. The 1966 investigation was performed for the existing structure at 50 Beale Street. The geotechnical investigation report for this building was not available for our review, however, we were able to review a summary of the soil boring logs presented on Sheet S7 of the drawings titled *"Bechtel Corporation San Francisco Office Building"* prepared by SOM and dated 17 January 1967. Based on our review of this drawing, we understand Dames & Moore drilled three borings in the vicinity of 350 Mission Street to depths ranging from about 120 to about 260 feet below the ground surface. These borings are labeled Boring Nos. 2, 3, and 5 and their approximate locations are shown on Figure 2. Copies of these logs are presented in Appendix A, on Figure A-1.

The 1980 investigation was performed for the office building at 50 Fremont Street. The results of this investigation were presented in a report titled "*Geotechnical Investigation, Proposed Five Fremont Center Project*" and dated 13 March 1981. During this investigation, Dames & Moore drilled five soil borings to depths ranging from about 150 to 200 feet and installed two piezometers. Boring 1 was drilled near the corner of Fremont and Mission Streets, in the vicinity of the 350 Mission Street project, as shown on Figure 2. We understand Dames & Moore drilled the borings using truck-mounted, 6-inch-diameter, rotary wash drilling equipment and drilling fluid consisting of bentonite mud; where piezometers were installed the drilling fluid consisted of a water-soluble drilling gel. During drilling, their engineer logged the soil encountered and recovered samples of various soil strata. Log of Boring 1 is presented in Appendix A as Figure A-2; the soil encountered in this log was classified according to the Soil Classification Chart and Key to Test Data presented in Figure A-3. Soil Samples during these investigations were obtained using the following sampler types:

- Dames & Moore Type U sampler; this sampler was driven with 340-pound slip-jars falling 18 inches.

- Dames & Moore Piston sampler; this sampler was advanced using hydraulic pressure.
- Standard Penetration Test (SPT) sampler with a 2.0-inch-outside diameter and a 1-3/8-inch-inside diameter; this sampler was driven with a 140-pound hammer falling 30 inches in accordance with ASTM D-1586-67.

4.2 Cone Penetration Tests by Treadwell & Rollo, Inc.

Treadwell & Rollo, Inc. performed a geotechnical investigation for the seismic strengthening of the existing building at 350 Mission Street in 1997. During this investigation, two cone penetration tests (CPTs) were pushed to supplement the Dames & Moore borings drilled in the vicinity. The CPTs were advanced to depths of 43 feet below the existing basement slab, corresponding to approximate Elevation - 49 feet (SFCD). The CPT logs, showing tip resistance and friction ratio with depth, interpreted soil types, and interpreted SPT blow counts are presented in Appendix B, as Figures B-1 and B-2. The soil types are classified using the chart presented on Figure B-3.

The CPTs were performed by hydraulically pushing a 1.4-inch diameter, cone-tipped probe into the ground. The cone on the end of the probe measured tip resistance, and a sleeve behind the cone tip measured frictional resistance. These parameters were continuously measured by electrical gauges within the cone during the entire depth advanced. The data were transferred to a computer while conducting each test. Accumulated data were processed by computer to provide engineering information, such as the type and approximate strength characteristics of the soil encountered.

4.3 Geotechnical Borings Performed by Treadwell & Rollo, Inc.

Treadwell & Rollo, Inc. performed geotechnical and environmental investigations for the building recently constructed at 301 Mission Street. The studies were conducted between 2001 and 2004. This site is south of 350 Mission Street, across Mission Street. The structure is a 60-story tower over one basement level adjacent to a 3-story atrium connected to a 9-story structure, over five levels of basement.

Subsurface conditions beneath the 301 Mission Street site were evaluated by performing two separate investigations. In June of 2001, five exploratory borings (designated as B-1 through B-5) were drilled. In May of 2004, two additional borings (designated as B-6 and B-7) were drilled. The borings were drilled to depths ranging from 60.5 to 220 feet below the existing ground surface, corresponding to approximate

Elevations -57 and -216 feet, respectively. Drilling was performed by Pitcher Drilling Company of Palo Alto, California, using truck-mounted rotary wash drilling equipment, under the direction of Treadwell & Rollo, Inc.'s field engineer.

During drilling, Treadwell & Rollo, Inc.'s engineer logged the borings and obtained representative samples of the material encountered for visual classification and laboratory testing. Logs of the borings closest to the proposed development at 350 Mission Street (B-1, B-4, and B-7) are presented in Appendix C, as Figures C-1 through C-3. Their approximate locations are shown on Figure 2. The material encountered was classified according to the soil classification system described on the Classification Chart presented in Appendix C, as Figures C-5. Soil Samples during these investigations were obtained using the following sampler types:

- Standard Penetration Test (SPT) sampler with a 2.0-inch-outside diameter and a 1.5-inch-inside diameter, without liners
- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch-outside diameter, 2.5-inch-inside diameter, lined with brass tubes with an inside diameter of 2.43 inches
- Osterberg (O) piston sampler using 3.0-inch outside diameter, thin-walled Shelby tubes
- Thin-walled Shelby Tubes (ST) with 3.0-inch-outside diameter

The SPT and S&H samplers were driven with a 140-pound, above-ground, safety hammer falling 30 inches. The blow counts required to drive the S&H sampler the final 12 inches of an 18-inch drive (N-values) were converted to approximate SPT N-values using a conversion factor of 0.6 and are shown on the boring logs. Where the SPT sampler was used, the actual blow counts are shown on the boring logs. The Osterberg sampler and Shelby Tubes were advanced into the soil using hydraulic pressure. The hydraulic pressure required to advance the Osterberg sampler and Shelby Tubes is shown on the boring logs.

4.4 Environmental Borings Performed by Treadwell & Rollo, Inc.

On 5 July 2001, Treadwell & Rollo, Inc. observed the drilling of six shallow borings at 301 Mission Street as part of an environmental investigation. The borings, designated as TR-1 through TR-6, were hand-augered inside previously existing buildings to depths ranging from 3.5 to 8 feet below the previous basement or ground floor slabs at the site. The log of boring TR-1 is presented in Appendix C, as Figure C-4. The boring location is shown on Figure 2.

5.0 SITE CONDITIONS

5.1 Site History

The area in which the 350 Mission Street Building is located was once part of San Francisco Bay (Bay) known as Yerba Buena Cove. The cove was progressively filled between 1840 and 1900. Historic maps indicate the 1849 Bay shoreline passed immediately west of the building. Wharves were originally constructed into the Bay along the alignments of the primary east-west streets, with crosswalks connecting the wharves in the north-south direction. The general practice was to fill between the wharves and crosswalks to provide usable land. The fill was usually placed by dumping. Fill material was primarily generated from sand dunes to the west of the cove.

5.2 Existing Structure

The existing 350 Mission Street Building is a four-story, reinforced concrete structure with a single basement. It was constructed in the early 1920s and measures about 135 by 135 feet in plan, as shown on the Site Plan (Figure 2). The basement extends beneath the street sidewalks and measures about 155 by 155 in plan. The top of basement slab is at about Elevation -6 feet; the top of sidewalk elevations adjacent to the site vary from about Elevation 2 to 3 feet.

According to a plan titled "*Foundation Plan, Structural Design, Fremont and Mission Building, San Francisco*", dated December 1922, the building walls and columns are supported on about 700 timber piles. Based on the results our previous investigation and our experience with similar projects, we judge the timber piles were likely driven into a dense sand deposit at a depth of about 40 feet below the basement floor slab (approximate Elevation -46 feet). The timber pile caps vary in thickness from 3 to 3.5 feet and the pile cutoffs extend to between 3.6 and 6.6 feet below the top of the basement slab. The finished basement slab is six inches thick and reinforced with #6 wires in a 6- by 6-inch mesh. The approximate locations of the existing pile caps and timber piles are shown on Figure 3.

The original structure was seismically strengthened in the late 1990s for compliance with the requirements of the San Francisco Building Code. The new strengthening elements consisted of two concrete shear walls along column lines C and 4 and other elements that are supported on a combination of existing timber and new steel pipe piles. The approximate locations of the new shear walls and underlying piles are shown on Figure 4. The shear walls are about 18 inches wide; the new pipe pile caps are about three feet wide, four feet deep, and they are supported on driven steel pipe piles.

Twenty 12.75-inch-outside-diameter steel pipe piles were installed for support of two new shear walls. The piles were driven using a low-headroom pile driver operating at an energy level between 30,000 to 35,000 foot-pounds. The piles were driven to refusal in the dense sand layer encountered at depth from about 40 to 45 feet below the basement slab, tipped at approximate Elevations -46 to -51 feet. The steel pipe piles achieved uplift capacities ranging from about 40 to 50 kips.

On the basis of our review of the drawings titled, "*Bechtel Corporation San Francisco Office Building*", Sheet S2 (Shoring and Bracing Details) prepared by Skidmore Owings & Merrill Architects and dated January 17, 1966, we understand the existing building at 350 Mission Street was underpinned and braced laterally during construction of the 50 Beale Street Building. A concrete underpinning system was constructed along the east pile cap of the 350 Mission Street Building. The size and depth of the underpinning elements are not shown on the existing drawings; however, section "F" on sheet S2 indicates the underpinning elements extend a few feet below the bottom of the 50 Beale Street pile cap, as shown on Figure 5. The lateral struts consisted of steel H12x53 ("12 BP 53") piles and they were preloaded to about 50 kips.

5.3 Adjacent Structures

The 350 Mission Street building is bound by existing high-rise structures to the east (50 Beale Street) and north (45 Fremont Street). The 50 Beale Street building has 24 stories of office space over a single basement and is supported on driven piles. The drawings, titled "*Bechtel Corporation San Francisco Office Building*", indicate the top of basement slab is at about Elevation -12 feet and the perimeter pile cap adjacent to the 350 Mission Street building is bottomed at about Elevation -18.5 feet. The outer edge of this pile cap is about two feet east of the property line. The piles installed at this perimeter cap have a design capacity of 100 tons and their design tip elevation is -63 feet.

The existing structure at 45 Fremont Street has 34 stories of office space over a basement. Based on our review of the drawings titled "*Bechtel II Office Building*" prepared by SOM and dated 1975, the basement finished floor is at about Elevation -12 feet. We understand the building is supported on 12-inch-square, pre-cast, concrete piles driven to about tip elevation -60 feet. These piles have a design capacity of about 100 tons. The perimeter pile caps at the south end of the building are within inches of the south property line and they are bottomed at about Elevation -16 feet.

6.0 SUBSURFACE CONDITIONS

On the basis of our interpretation of the results of previous CPTs performed at the site and borings drilled in the vicinity, we conclude the site is underlain by fill, recent bay deposits (Marine Deposits), dense sand (Colma Sand), Old Bay Clay, and bedrock. Figures 6 and 7 present idealized subsurface profiles across the project site. These profiles are based on the Dames & Moore boring logs and their approximate locations are shown on Figure 2.

Fill

The 350 Mission Street Building is currently underlain by about 13 to 17 of undocumented fill. The fill generally consists of loose to medium dense sand with varying silt, clay, and gravel content. Where previously encountered in the site vicinity, the fill extends to between approximate Elevations -17 and -23 feet.

Heavy drilling fluid circulation loss occurred while drilling through fill in Boring 3 (Dames & Moore) at about Elevation -20 feet. Boring 5 (Dames & Moore) encountered bricks, decomposed sandstone and shale fragments, and rubble in the fill. Construction rubble such as bricks, wood, and concrete fragments were also encountered in the Treadwell & Rollo, Inc. borings drilled at 301 Mission Street. We expect the fill beneath the 350 Mission Street site to vary in relative density, strength, and composition. The fill may contain construction debris, voids, and perhaps old shoring and underpinning elements.

Marine Deposits

The fill is underlain by relatively compressible Marine Deposits extending to depths ranging from about 48 to 53 feet below the ground surface, corresponding to elevations ranging from -46 to -50 feet. The Marine Deposits consist of weak, highly compressible clay, known locally as Bay Mud, and marine sand. Where encountered in the vicinity, the Bay Mud extends to approximate Elevations -30 and -40 feet. The Bay Mud is gray to dark gray, soft to medium stiff, and contains shell fragments and decomposing organic matter. Marine sand deposits were encountered within and below the Bay Mud.

The results of the Dames & Moore investigation performed for the 50 Beale Street building, indicates a four-foot-thick layer of gray silty sand is interbedded in the Bay Mud Deposit at about Elevation -30 feet in Boring 5. Furthermore, about seven to ten feet of clayey sand with lenses of silty clay and sandy clay were encountered below the Bay Mud in Borings 2, 3, and 5. The marine sand layers were described as loose to compact.

At Borings B-1 and B-4, drilled for the 301 Mission Street development, medium dense clayey sand and silty sand were encountered between approximate Elevations -45 and -55 feet. Standard penetration test results performed in this stratum indicate blow counts of about 13 and 12 blows per foot; the results of laboratory plasticity tests performed indicate plasticity indices (PIs) of about 9 and 4 at Borings B-1 and B-4, respectively.

Colma Sand

In general, the Marine Deposits are underlain by about 30 to 40 feet of dense to very dense sand with varying amounts of clay and silt of the Colma formation (Colma Sand). The Colma Sand is strong, has low compressibility and is capable of accommodating relatively large compression loads (bearing layer). The bottom of this deposit extends to 80 to 95 feet below the ground surface, corresponding to approximate Elevations -77 and -92 feet.

The Dames & Moore Borings 2 and 3 describe the upper five to ten feet of the Colma Sand as light brown, slightly clayey, fine sand (compact). The remaining Dames & Moore borings drilled in the vicinity (Borings 1 and 5), indicate the upper Colma Sand is dense to very dense sand.

Old Bay Clay

Old Bay Clay, which underlies the Colma and Marine Deposits, is slightly to moderately over-consolidated, stiff to hard clay. Occasionally, dense sand layers of variable thickness are present within the Old Bay Clay. The Old Bay Clay thickness varies from about 110 to 120 feet and extends to about Elevation -170 feet in Dames & Moore Boring 2, -200 feet in Dames & Moore Boring 5, and -196 feet in T&R Boring B-7.

The Old Bay Clay consists of very stiff to hard clay and sandy clay, and very dense sand and silty sand to the maximum explored depth.

Bedrock

The map entitled "*Bedrock surface Map of the San Francisco North Quadrangle,*" prepared by Julius Schlocker and dated 1961 indicates the top of bedrock in the vicinity is at about Elevation -250 feet (Mean Sea Level), which corresponds to about Elevation -240 feet San Francisco City Datum. The bedrock consists of highly deformed and fractured Franciscan Formation. Generally, the rock previously encountered in the vicinity consists of deeply weathered sandstone, shale, and occasionally serpentine and chert.

During the Dames & Moore investigation for the 50 Beale Street building, red and green chert fragments in a sandy clay matrix were encountered from approximate Elevations -220 and -248 feet at Borings 2 and 5, respectively, to the maximum explored depths.

Groundwater

The available information from previous investigation at the site and vicinity, including the 301 Mission Street and Five Fremont Center investigations, indicate groundwater levels are within the fill and were encountered between approximate Elevation -6.5 and -10 feet.

Groundwater levels were measured in the Colma Sand deposit during the geotechnical investigation for the Five Fremont Center Project. We understand the initial water level reading in the Colma Sand was observed at about Elevation -17 feet; however, it was later measured, after stabilizing, at about Elevation -7 and -8 feet.

We anticipate the groundwater level in the fill will vary seasonally a few feet depending on rainfall infiltration and time of year. The groundwater level may also vary due to dewatering activities in the vicinity, utility leaks, and perhaps tidal influences. On the basis of the available groundwater information and to account for seasonal fluctuations and the effects of temporary dewatering in the vicinity, we judge the high groundwater level within the project site may be as high as Elevation -3 feet; the low groundwater level is likely near Elevation -12 feet.

7.0 SEISMIC CONSIDERATIONS

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground rupture, liquefaction and differential compaction. Our evaluation of seismic considerations for the project site is presented in the following sections.

7.1 Regional Seismicity

The major active faults in the area are the San Andreas, Hayward, San Gregorio, and Calaveras Faults. These and other faults of the region are shown on Figure 8. For each of the active faults, the distance from the site and estimated mean characteristic Moment magnitude[3] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
N. San Andreas – Peninsula	13	West	7.23
N. San Andreas (1906 event)	13	West	8.05
N. San Andreas – North Coast	16	West	7.51
Total Hayward	16	East	7.00
Total Hayward-Rodgers Creek	16	East	7.33
San Gregorio Connected	19	West	7.50
Mount Diablo Thrust	33	East	6.70
Rodgers Creek	33	North	7.07
Total Calaveras	34	East	7.03
Green Valley Connected	38	East	6.80
Monte Vista-Shannon	41	Southeast	6.50
Point Reyes	42	West	6.90
West Napa	44	Northeast	6.70
Greenville Connected	51	East	7.00
Great Valley 5, Pittsburg Kirby Hills	55	East	6.70
Great Valley 4b, Gordon Valley	68	Northeast	6.80
Hunting Creek-Berryessa	76	North	7.10
N. San Andreas - Santa Cruz	77	Southeast	7.12
Great Valley 7	77	East	6.90
Zayante-Vergeles	87	Southeast	7.00
Great Valley 4a, Trout Creek	90	Northeast	6.60
Maacama-Garberville	91	North	7.40
Monterey Bay-Tularcitos	100	Southeast	7.30

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3 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.

Figure 8 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through December 2000. 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 9) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, 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 (km) in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 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 95 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) predicts 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 (2008) 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

7.2 Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[4], differential compaction[5] and lateral spreading[6]. We used the results of the test borings to evaluate the potential of liquefaction and differential compaction at the project site.

7.2.1 Historic Failures

Based on our review of the report titled, "*Historic Ground Failures in Northern California Triggered by Earthquakes*," (Youd and Hoose 1978), we understand ground settlement, ground surface cracks, and sand boils were observed in the vicinity of the site following major earthquakes. Table 3 presents a summary of the historic ground failures described in the report.

4 Liquefaction is a phenomenon in which saturated, cohesionless soil experiences a temporary loss of strength due to the buildup of excess pore water pressure, especially during cyclic loading such as that induced by earthquakes.

5 Differential compaction is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing differential settlement.

6 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.

TABLE 3
Specific Descriptions of Historic Ground Failures

202		A	1868	Lawson and others, 1908, p. 437.	At Fremont and Mission Streets the ground opened in many places. The general course of damage in the city was along the irregular line of the "made land," or low alluvial soil, where it met the hard or rocky base beneath it. Along the line of the old shore of Yerba Buena Cove, we found the damage to brick buildings much the largest.
		A	1906	Lawson and others, 1908, p. 437, plate 146.	The floor of the Pacific foundry was raised about 2 feet in places. The center of Mission Street (opposite Fremont Street) exposed an opening from 8 to 10 inches wide; and openings of the ground were also plainly to be seen on Fremont Street, in the same vicinity. Outside of the immediate district described above, damage to the rest of the city was very meager. * * * the region of greatest agitation was confined to the low portions of the city, or the vicinity of some old creek bed or swamp.
		B	1868	Huber, 1930, p. 267.	The shop of Mr. Garratt, brass founder, near the corner of Mission and Fremont Streets, has been raised to its original position. It was found to have been lowered about eight inches by the recent earthquake.
		A	1906	Himmelwright, 1906, p. 86.	SCOTT BUILDING. S. W. Cor. Fremont and Mission Streets. * * * The S. W. corner of the building is badly racked and cracked by the earthquake.
		B	1865	Huber, 1930, p. 264.	Stoddard's warehouse on Beale Street is said to have been thrown out of place several inches as though it had been lifted up and set down again, while the south side of the building appears to have settled considerably. After the shock, the water rushed into the cellar, or basement, but whether from a disarrangement of the water pipes, or from any fissure in the earth which might have opened, was not known.
		A	1838	Louderback, 1947, p. 52.	Mr. Spear informed me that during the earthquake of June, '38, before mentioned, a large sand-hill standing in the vicinity of what is now Fremont street, between Howard and Folsom, and between which and the bay at high tide there was a space of about twenty feet, permitting a free passage along the shore to Rincon Point (the coves of which were then much resorted to for picnics and mussel parties), was moved bodily close to the water, so as to obstruct the passage along the shore. After that no one could pass there at high tide, and we were compelled to go around back of the sand-hill, and wade through loose sand to reach that point, a much more laborious walk.

7.2.2 Ground Shaking

The seismicity of the site is predominantly governed by the activity of the San Andreas and Hayward Faults. However, ground shaking from future earthquakes on any of the nearby faults will be felt at the site. The intensity of earthquake ground motion at the site will depend upon the characteristics of the generating fault, distance to the earthquake epicenter, magnitude and duration of the earthquake, and specific subsurface conditions. On the basis of our knowledge of subsurface conditions, we conclude ground shaking at the site during a large earthquake on one of the faults discussed in Section 7.1 will be very strong.

7.2.3 Liquefaction

The site is in an area of San Francisco that is designated as a seismic hazard zone by the California Division of Mines and Geology (CDMG 2000). The primary purpose of this designation is to identify areas of potential soil liquefaction. Soil most susceptible to liquefaction is loose, clean, saturated, uniformly graded, fine-grained sand and silt of low plasticity that is relatively free of clay. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction.

The results of our studies indicate that sandy portions of the fill and Marine Deposits below the ground water table are susceptible to liquefaction. Using the procedure outlined in the proceedings of the NCEER workshop (1997) and Tokimatsu and Seed (1984), we estimate the ground surface may settle between approximately 1-1/2 to 3 inches after a major earthquake as a result of post-liquefaction reconsolidation. This range agrees with the 0 to 6 inches of liquefaction-induced settlement estimate presented in the report titled "*Final Report, Liquefaction Study, North Beach, Embarcadero Waterfront, South Beach, and Upper Mission Creek Area*"[7]. However, because the liquefiable layers encountered in the fill and Marine Deposits should be excavated as part of foundation construction, liquefaction-induced settlement should be negligible below foundation level.

Outside of the excavation, we judge that significant subsidence of streets and sidewalks could occur during an earthquake. This settlement is expected to be random and erratic, and will most likely disrupt utilities and damage sidewalks and streets. The design of utilities and slabs at the building perimeter should take this settlement into account.

7.2.4 Differential Compaction

Differential compaction can occur during strong ground shaking in loose, clean, granular deposits above the water table, resulting in ground surface settlement. The soil deposits encountered above the water table are generally susceptible to differential compaction.

Using the simplified procedure developed by Tokimatsu and Seed (1987) to evaluate settlements in dry sands, we estimate up to approximately one inch of differential compaction settlement could occur in the existing fill during a major earthquake. The differential compaction settlement is expected to cause damage to sidewalks, streets, and underground utilities; however, because the proposed lowest finished floor will be at about Elevation -42 feet, differential compaction settlement should not occur beneath the proposed foundation.

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7 Report prepared by prepared by Harding & Lawson Associates (HLA), Dames & Moore and others for the City and County of San Francisco Department of Public Works, dated January 1992.

7.2.5 Lateral Spreading

Ground failure associated with liquefaction includes lateral spreading, a phenomenon in which surficial soil displaces along a shear zone that has formed in an underlying liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a channel, by earthquake and gravitational forces.

Studies performed for the City and County of San Francisco predict that shaking from a large earthquake would result in liquefaction of the fill. Lateral displacements associated with such an event have been estimated to range from 0 to 3 inches in the vicinity of the site. However, because the proposed building will be founded below the liquefiable sand, and the adjacent buildings are pile-supported, lateral spreading is unlikely to affect the proposed structure.

7.2.6 Ground Rupture

Historically, ground surface ruptures closely follow the trace of geologically young faults. 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. We therefore conclude the risk of fault offset at the site from a known active fault is low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is low.

8.0 DISCUSSION AND CONCLUSIONS

We conclude from a geotechnical engineering standpoint, the site can be developed as currently planned provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction.

The project will involve demolishing and removing the existing structure, installing temporary shoring, dewatering the site, excavating the basement to a depth of about 55 feet (including foundation thickness) below the existing ground surface, and constructing the new building supported on a mat foundation. The primary geotechnical issues for this site are:

- support of adjacent structures, streets, utilities, and other improvement during excavation for the proposed basement

- foundation support for the proposed structure and the anticipated static settlement
- the potential for seismically-induced ground settlement resulting from liquefaction and differential compaction.
- the presence of groundwater at a level higher than the proposed excavation depths

These geotechnical issues and their impact on the proposed grading, foundation design, and construction are discussed herein.

8.1 Foundations and Settlement

According to the current project plans prepared by the SOM, the proposed finished floor elevation at the lowest parking level is about 44 to 45 feet below existing sidewalk grades, at Elevation -42 feet. According to correspondence with SOM, the bottom of the proposed mat foundation will be at about Elevation -52 feet.

Based on the available subsurface data, we estimate the top of the bearing sand layer (Colma Sand) varies from about Elevation -46 to -50 feet, and it is deeper toward the east end of the structure. The bottom of the bearing layer ranges from Elevation -77 to -92 feet; therefore it appears to vary in thickness from about 30 to 40 feet. This dense to very dense sand layer can support moderate to heavy foundation loads without excessive settlement. Core building loads and column loads should be effectively distributed. The proposed reinforced mat foundation bottomed on undisturbed dense sand should be adequate to effectively distribute these loads, reduce differential settlement, and resist hydrostatic uplift forces. We anticipate dense to very dense sand will be present at the bottom of mat elevation; however, if material other than that is encountered at the foundation subgrade, it should be removed to expose dense to very dense sand and replaced with engineered fill or control density fill (CDF).

The foundation subgrade will be susceptible to disturbance from construction activities and it may become unstable. To help protect the soil subgrade a working pad should be constructed in these areas. Recommendations for construction of a working pad are presented in Section 9.3 of this report.

As the Colma Sand and Old Bay Clay are compressed due to new foundation pressures[8], we estimate to about 3 to 4 inches of settlement (immediate elastic settlement of the Colma Sand and primary consolidation and secondary compression of the Old Bay Clay) will occur in these layers. Differential settlement will depend on the rigidity of the mat. We estimate differential settlement could be on the order of about ¾ inch between column locations. These settlements should be accounted for in the design of the mat and the structure, particularly at building entrances and other areas where there are structural or aesthetic connections between the surrounding areas outside of the building that should not experience static ground settlement and the building, which is expected to settle up to 4 inches.

As discussed in Sections 7.2.3 and 7.2.4, seismically-induced settlement should be negligible beneath the proposed foundation if it bears on undisturbed Colma Sand. However, the adjacent sidewalks, streets, and underground utilities will likely experience seismically-induced settlements of up to four inches and they may become damaged following a strong earthquake.

8.2 Construction Considerations

The main construction considerations are removal of existing piles, shoring requirements, dewatering for the basement excavations, differential settlement at building/sidewalk interface. Additional concerns are the presence of concrete rubble, debris, abandoned shoring and underpinning elements in the near-surface fill. These issues are discussed in the remainder of this section.

8.2.1 Differential Settlement at Building/Sidewalk Interface

Utilities, exterior slabs and utility connections entering the buildings should be designed to accommodate potential differential settlement of up to about four inches. To mitigate the anticipated differential ground settlement, flexible connections should be used where utilities enter the buildings. Additionally, exterior slabs and ramps attached to the building should be designed to accommodate differential settlement between the buildings and exterior ground at all entrances, and sidewalks; other on-grade improvements should be designed to accommodate the expected movement where they cross into the building. Maintenance of utilities, sidewalks and entry slabs should be expected throughout the life of the project. This may include periodically replacing some of the improvements at the building/outside area interface.

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8 Proposed building bearing pressures provided by SOM on a bearing pressure plot titled "350 Mission", dated 15 December 2011, Page 8 of 9. Applied bearing pressures range from 8,300 psf to 10,100 psf for dead plus live loads.

8.2.2 Pile Removal

Current plans are to demolish and remove the existing improvements at 350 Mission Street to accommodate the excavation for the proposed below-grade parking levels. During demolition, we expect 20, 12.75-inch diameter steel pipe piles and roughly 700 timber piles will be encountered. Piles should be cut off as the excavation proceeds. The removal of piles prior to excavation could result in some adverse effects to the proposed construction, including:

- Voids – Pile removal will result in voids in the existing foundation subgrade. The voids correspond to loss of volume of the pile and soil that becomes attached to it as the pile is pulled. These voids will likely be filled in by squeezing of the existing fill and Marine Deposits and may result in settlement of the ground surface, and reduced lateral support for existing piles that will remain in place, as well as any shoring that relies on passive resistance for support.
- Vibration – Pile removal equipment may create vibrations that are noticeable at the adjacent structures and may result in localized liquefaction of fill and Marine Deposits, further contributing to surface settlement
- Disturbance – The upper portion of the Colma Sand may become disturbed and weakened by the pile removal operation. However, the proposed excavation will extend to approximate Elevation -52 feet, which is anticipated to be deeper than the existing pile tip elevations; therefore, the disturbed soil should be removed during construction and should not affect the proposed foundation subgrade.
- Seepage – Groundwater seepage channels may be created by the removal of existing piles; however, because we anticipate the proposed excavation will generally extend to below the tips of existing piles, we judge the potential for this to occur is low. Furthermore, dewatering wells should be installed to prevent ground water seepage from occurring at the base of the excavation.

Considering the potential for adverse effects resulting from removal of the existing piles in one piece, the piles should be cut off as the excavation proceeds. This should reduce the potential for seepage channels, ground surface settlement, and disturbance to the proposed foundation subgrade to occur during construction.

8.2.3 Shoring

We understand the existing top of basement floor is at about Elevation -6 feet. The finished floor for the proposed basement will be about at about Elevation -42 feet. Construction of the basement levels and foundation for the proposed structure will require excavations extending to about Elevation -52 feet. Because the proposed excavation will be adjacent to existing buildings and existing street improvements, shoring will be required. There are several key considerations in selecting a suitable shoring system. Those we consider of primary concern are:

- protection of surrounding improvements, including roadways, utilities, and nearby structures
- the ability of the shoring system to minimize the inflow of groundwater and required dewatering
- the ability of the shoring system to reduce potential for ground movement
- presence of groundwater between approximate Elevations -3 to -12 feet
- cost

Several methods of shoring are available; we have qualitatively evaluated the following systems:

- soil nailing
- sheet piles
- conventional soldier-pile-and-lagging
- soldier pile tremie concrete (SPTC) or mixed-in-place soil/cement walls
- concrete-diaphragm walls
- secant pile wall

Soil nailing is a method of shoring using grouted reinforcing bars (nails), which are typically spaced between 4 and 6 feet, horizontally and vertically. Soil nails can be used where the soil has sufficient binder (cohesion) to allow a shallow vertical cut without the potential for a wedge failure. Facing, usually consisting of a four-inch-thick layer of shotcrete reinforced with wire mesh, stabilizes the excavation surface between nails. Considering the presence of loose to medium dense sand, weak Bay Mud, and shallow groundwater at the site, we conclude soil nailing would not be appropriate. Furthermore, the existing pile foundations at the adjacent structures and previously installed tiebacks for temporary shoring at 301 Mission Street would interfere with the installation of soil nails at this site.

We considered sheet piles, however, in our opinion this system is not appropriate because of: 1) the difficulty of driving them through the existing fill and into dense Colma Sand; 2) sheet pile installation generates significant vibrations that could densify and even liquefy the loose to medium dense sand outside the project vicinity and cause ground settlement; and 3) sheet piles are not sufficiently rigid to limit vertical and lateral movements of the adjacent improvements.

Soldier-pile-and-lagging systems are not suitable for this site because: 1) the sand layers encountered in the fill and Marine Deposits are highly permeable and difficult to drain; 2) there is also a risk that dewatering associated with a soldier-pile- and-lagging system may lower the groundwater level outside the excavation for an extended period of time and may cause areal subsidence; and, 3) soldier-pile-and-lagging systems are not sufficiently rigid to reduce the potential for lateral and vertical movements of the adjacent improvements.

We conclude that soldier pile tremie concrete (SPTC) or mixed-in-place soil/cement walls, diaphragm walls, and secant pile walls are appropriate for the proposed construction. These systems would likely be somewhat watertight and thus require the least dewatering. These shoring walls would also be relatively rigid, which should reduce lateral deflections and ground subsidence related to the excavation. The disadvantages of these systems are cost and space requirements. Installation for these systems will require a width of about three feet around the perimeter of the site. We anticipate the proposed shoring system will be constructed inside the existing basement area.

Soil-cement-mixed walls are installed by advancing hollow-stem augers and pumping cement slurry through the tips of the augers during auger penetration, or using jet grouting equipment. In one type of soil-cement-mixed wall system, the walls are constructed by excavating grooves with a moving chain-saw cutter. The soil is mixed with the cement slurry in situ, forming continuous overlapping soil-cement columns or continuous walls. Steel beams are placed in the soil-cement columns or walls at pre-determined spacing to provide rigidity. Soil-cement walls are considered temporary; permanent walls are usually built in front of the walls.

Concrete-diaphragm walls are reinforced concrete walls constructed by slurry trench method. The walls are constructed in sections, called panels. During excavation of a panel, bentonite slurry is pumped into the trench to prevent the soil from caving. After the excavation reaches the design depth and the reinforcement cage is placed, the slurry is displaced by concrete that is poured through a tremie pipe. Diaphragm walls can be used as temporary shoring and permanent walls.

Secant piles are drilled shafts that interlock to form a continuous wall. The wall is constructed by drilling alternate shafts and then "back stepping" to drill the intervening shafts in order to interlock the two adjacent shafts. Every second shaft is reinforced usually with a wide flanged steel section or reinforcing steel cage. The reinforced shafts are called "primaries." The alternate shafts, which are not reinforced, are called "intermediates" or "secondaries." The concrete used for the secondary piles is usually lean concrete that remains weak enough for the drilling and interlocking of the primary shafts. The primaries are usually poured with structural concrete. Because the subgrade soil includes loose sand and medium stiff clay, measures should be taken to reduce the potential for ground loss associated with drilled pier construction. Based on our experience, procedures such as using drilling slurry, tremie placement of concrete, and the use of steel casing should reduce the potential for ground loss.

Lateral resistance against movement may be mobilized by extending the shoring below the bottom of the excavation and using internal braces. Internal bracing can be either cross-lot or inclined rakers. Tiebacks should not be used at the site because of interference from pile foundations of adjacent structures and the tiebacks installed as part of the shoring of the structures across the streets.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. However, the shoring should be designed by a shoring engineer knowledgeable in this type of construction. The contractor should anticipate potential underground obstructions such as existing piles, concrete debris, rubble, and abandoned shoring and underpinning elements during construction of the shoring wall.

8.2.4 Dewatering

Current plans for the basements require excavations which will be well below the groundwater level. The design groundwater level should be taken as Elevation -3 feet, although the actual groundwater level at the time of construction may be somewhat lower. We anticipate the proposed excavation will extend to approximate Elevation -52 feet (about 49 feet below design groundwater).

Our study of the available information indicates the piezometric water level (head) in the Colma Sand has been measured between approximate Elevation -7 and -8 feet. Piping of the proposed excavation could occur if the piezometric pressure in the Colma Sand is not reduced. Piping would result in loss of passive pressure on the toe of the proposed shoring wall and would disturb the foundation subgrade making it unsuitable for support. A relatively impervious shoring extending through the Colma Sand into the Old Bay Clay layer would reduce the potential for piping to occur.

Within the limits of the proposed "cut off" shoring wall, the groundwater level should be maintained at least three feet below the planned maximum excavations until sufficient weight and/or uplift capacity is available to resist the hydrostatic uplift forces on the bottom of the structure. The project structural engineer should evaluate when the dewatering can be stopped.

The efficiency of the dewatering system will depend to some extent on the type of shoring system used. For example, a soil-cement wall or concrete-diaphragm wall would likely be the most water-tight and thus require the least dewatering. A secant wall shoring system should also be watertight provided the piers are installed straight and continuously overlap. The depth of the shoring will also affect the dewatering as the shoring can be extended downward to cut-off water-bearing layers. Relatively impervious shoring extending through the Colma Sand into the Old Bay Clay would reduce dewatering needs. The regional groundwater (groundwater outside the proposed shoring) should not be lowered. This will reduce the risk of damage to structures, utilities and roadways in the vicinity of the site.

The working pad discussed in Section 8.1, may also be used as a temporary subdrain system or drainage blanket. Perforated pipes may be placed in the gravel to collect water and conduct it to a sump. The sump and collector pipes should be decommissioned once they are no longer needed. Recommendations for construction of a working pad are presented in Section 9.3.

The selection and design of the dewatering system should be the responsibility of the contractor. The contractor will need to obtain a dewatering and discharge permit from the City and County of San Francisco for discharging water into the local municipal storm drain system. Currently, there is a fee for disposing of construction generated water into the City's wastewater collection system. Selection of the shoring and dewatering systems should be coordinated to minimize overall costs.

Variables that significantly influence the performance of the dewatering system and the quantity of water produced include the number, depth, and positioning of the wells, the interval over which each well is screened, and the rate at which each well is pumped. Different combinations of these variables can be used to dewater the site. The site dewatering should be designed and implemented by an experienced dewatering contractor. However, we should check the dewatering system proposed by the contractor prior to installation.

Excessive site dewatering could result in subsidence of the immediate area due to increases in effective stress in the soil. Therefore, adjacent improvements should be monitored for vertical movement, and groundwater levels outside the excavation should be monitored through wells while dewatering is in progress. Should excessive settlement or groundwater drawdown be measured, the contractor should be prepared to recharge the groundwater outside the excavation through recharge wells.

8.2.5 Excavation Monitoring

During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle and move laterally. The magnitude of shoring movements and resulting ground deformations are difficult to estimate because they depend on many factors, including the type of shoring system used and the contractor's skill in the shoring installation. We believe ground surface settlement associated with properly designed and constructed soil-cement wall, concrete-diaphragm walls, or secant pier walls should be less than about ½ inch; lateral movement of ground surface at the top of the wall should be about one inch. The amount of lateral movement will also depend on how much soil is excavated below the specified location of supports.

A monitoring program should be established to evaluate the effects of the construction on the adjacent improvements. The contractor should install surveying points to monitor the movement of shoring and settlement of adjacent structures during excavation. This monitoring system should provide timely data which can be used to modify the shoring system during construction if needed. In addition, geotechnical instrumentation including inclinometers and piezometers should be installed to monitor movement of the shoring system and the groundwater level during excavation and construction.

8.3 Environmental Considerations

The site is within the Article 22A (Maher Ordinance) zone of San Francisco because it is bayward of the historic 1852 San Francisco high tide line. Construction projects located within the Maher zone that will disturb more than 50 cubic yards of soil are required, by the ordinance, to have their site history and soil quality assessed. In addition to assessing the site history and soil quality, analytical tests on groundwater samples will be required for the dewatering permit.

The dewatering permit will require chemical testing for characterizing the water to be discharged into the storm drain system.

9.0 RECOMMENDATIONS

Our recommendations regarding site preparation and grading, temporary shoring, mat foundation design, lateral earth pressures for basement walls, and seismic design are presented in this section of the report. The results of our site-specific seismic study are presented in Appendix D.

9.1 Site Preparation and Grading

We anticipate excavation for this project can be made using conventional earth moving equipment. Old slabs, foundations (including pile caps, timber and steel pipe piles), abandoned shoring and underpinning systems, other obstructions may be encountered during shoring installation and excavation within the fill and Marine Deposits. Where encountered, these obstructions should be removed.

We anticipate engineered fill will be required to backfill the existing basement beneath street sidewalks and utility trenches. On-site sandy fill is suitable for reuse as backfill provided it is non-corrosive, acceptable from an environmental standpoint, and meets the requirements given below for general fill. Soil below the groundwater will require drying by aeration prior to its reuse as compacted fill. All materials to be used as fill, including on-site soil, should be non-corrosive, non-hazardous, 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 40 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[9]. Fill deeper than five feet or containing less than 10 percent fines (percent passing the No. 200 sieve) should be compacted to at least 95 percent relative compaction. During construction, we should check that the on-site and any proposed import material is suitable for use as fill.

9.2 Mat Foundation Preparation

Because the bottom of the excavation will be below the present groundwater level, the soil at subgrade will be near saturation even after dewatering. To protect the mat subgrade, we recommend

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9 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 laboratory compaction procedure.

overexcavating the site and constructing a working slab on which to construct the mat. We anticipate an excavation of about 12 inches will suffice if used in conjunction with a woven reinforcing fabric (geotextile) such as Mirafi HP370 (or equivalent). After placing the reinforcing fabric on the exposed subgrade, the excavation should be backfilled with clean one-inch minus crushed rock. Alternatively, a 3-inch-thick mud slab may be used as a working pad.

To provide a smooth construction surface for the mat foundation and water-proofing membrane, the crushed rock may be covered with a thin concrete mud-slab. As discussed in Section 8.2.4, depending on the amount of water at the subgrade elevation, it may be desirable to use the crushed rock as a temporary subdrain system. To drain the crushed rock, four-inch diameter perforated PVC pipe should be placed near the bottom of the rock, spaced every 30 feet, to direct water trapped in the rock to a sump. The sump should be properly abandoned before the completion of construction.

Prior to placing the reinforcing fabric and crushed rock, we should evaluate the soil subgrade at base of the excavation to check if the excavation is in dense sand. We anticipate the top of the dense sand is between approximate Elevations -46 and -50 feet; consequently, we anticipate it will be exposed at the planned bottom of mat elevation. The areas where the subgrade has been over-excavated should be replaced with Class II aggregate base rock or lean concrete. Other subgrade areas where disturbed or soft soil is encountered should also be removed and replaced with crushed rock or lean concrete.

9.3 Waterproofing the Base of Mat

Because the proposed foundation will extend below the groundwater level, waterproofing the base of the mat is recommended. The mat should be designed to resist hydrostatic pressure based on a design groundwater elevation of -3 feet. The waterproofing should be designed by a waterproofing consultant and placed in accordance with the manufacturers specifications; however, waterproofing is typically placed directly on the soil subgrade and covered by a mud slab (thin layer of lean concrete). The mud slab will reduce the potential for subgrade disturbance and protect the waterproofing from damage during mat construction. The mud slab should also provide a firm, smooth working surface for placement of reinforcing steel.

If it is essential to prevent moisture accumulation on the garage floor, we recommend a back-up moisture barrier be included between the structural mat and a topping slab as an additional precaution. A typical moisture barrier includes a capillary moisture break consisting of at least a six-inch-thick layer of clean, free-draining crushed rock (½- to ¾-inch gradation) overlain by a moisture-proof membrane of at least 10 mil thickness. The membrane should be covered with two inches of sand to protect it during construction and to aid in curing the concrete floor slab. Perforated pipes may be installed in the capillary break to collect any water that accumulates and direct it to a sump or other suitable outlet. Water should not be allowed to accumulate in the drain rock or sand prior to casting the topping slab.

9.4 Mat Foundation

We recommend that the proposed structure be founded on a mat bearing on dense to very dense Colma Sand. In the event soft areas are encountered at final mat subgrade elevations, they should be overexcavated to competent material and backfilled with lean concrete.

SOM performed mat analyses using a modulus of vertical subgrade reaction value that we previously provided to them. We reviewed the results of their mat analyses and, in turn, performed settlement analyses, which resulted in adjusted vertical moduli values, and the process was repeated. After several iterations with the structural engineer regarding applied bearing pressures and resulting settlements, using recommended vertical subgrade moduli values of 25 to 30 kips per cubic foot (kcf), the bearing pressures for dead plus live loads on the mat foundation was calculated to be on the order of 8,000 to 10,000 pounds per square foot (psf). These values are within the allowable bearing pressure range for Colma Sand.

We judge that the guidelines set forth in Section 4.4.2.1.2 of FEMA 356 for foundation spring dynamic analysis are an appropriate approach to consider dynamic foundation soil/structure effects. For this analysis, we recommend the following parameters be used:

- Effective Shear Modulus, $G = 5.05 \times 10^5$ psf
- Poisson's Ratio, $\nu = 0.3$
- Stiffness increase and decrease of 20 percent.

Resistance to lateral loads can be mobilized by a combination of passive pressure acting against the vertical faces of the mat and friction along the base of the mat. Passive resistance may be calculated using lateral pressures corresponding to an equivalent fluid weight of 200 pounds per cubic foot (pcf) (submerged). In the event the passive resistance is used to resist lateral loads, the basement walls should be designed for the approximate passive earth pressure. Frictional resistance should be computed using a base friction coefficient of 0.2. These values include a factor of safety of about 1.5 and may be used in combination without reduction.

We should observe mat subgrade prior to placement of reinforcing steel. The excavation for the mat should be free of standing water, debris, and disturbed materials prior to placing concrete.

9.5 Basement Walls

Basement walls should be waterproofed. We recommend all below-grade and retaining walls be designed to resist lateral pressures imposed by the adjacent soil and vehicles. Lateral earth pressures on basement walls will depend partially on the restraint at the top of the walls. Accordingly, walls should be designed for the equivalent fluid weights presented below, where H is the height of the wall in feet.

TABLE 3
Lateral Earth Pressures Restrained Wall Condition

	Static	Seismic
Above the water table[10]	60 pcf	40 pcf + 30 pcf
Below the water table	90 pcf	80 pcf + 30 pcf

Surcharge loads from traffic and the pile foundations of adjacent structures should be included in the wall design. Where traffic is expected within a distance equal to the height of the walls, the wall should be designed for a additional uniform lateral pressure of 100 psf to be applied over the entire height of the wall or 10 feet, whichever is less.

The surcharge load acting on the north basement wall (adjacent to the 45 Fremont Street building) should be computed using an equivalent fluid weight of about 12 pcf. The surcharge loads acting on the east basement wall (adjacent to the 50 Beale Street building) should be computed using an equivalent

[10] Design groundwater level is Elevation -3 feet.

fluid weight of 20 pcf. The surcharge load from these adjacent building piles should be applied below the elevation of their existing pile caps, (below approximate Elevation -18 feet). Table 4 presents a summary of the surcharge pressures recommended for design of the basement wall.

TABLE 4
Surcharge Lateral Pressures

	Pressures (psf)
North Basement Wall (adjacent to 45 Fremont Street)	12H (H in feet, triangular distribution)
East Basement Wall (adjacent to 50 Beale Street)	20H (H in feet, triangular distribution)
Traffic Surcharge	100 (rectangular distribution)

The recommended design pressures assume the walls will be properly backdrained above Elevation -3 feet. One acceptable method for backdraining a basement wall is to place a prefabricated drainage panel against the backside of the newly cast wall. The drainage panel should extend down to Elevation -3 feet. The drainage panel will reduce the risk of hydrostatic pressure against the upper portion of the basement wall by allowing water to drain to the groundwater level, about Elevation -3 feet. We should review the manufacturer's specifications regarding the proposed prefabricated drainage panel material to check it is appropriate for the intended use.

To protect against moisture migration, basement walls should be waterproofed and water stops should be placed at all construction joints.

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.

9.6 Seismic Design Criteria

We developed site-specific response spectra and time histories consistent with the guidelines in the 2010 San Francisco Building Code (SFBC) and a response spectrum for the Serviceability Level Earthquake (SLE) consistent with the guidelines of Peer Tall Building (2010).

9.6.1 2010 SFBC Mapped Values

On the basis of the results of our subsurface investigation, the site is classified as stiff soil with an average shear wave velocity in top 30 meters (100 feet), V_{s30} of 243 m/s (797 ft/sec). This is consistent with site class S_D. The site coefficients, F_a and F_v are 1.0 and 1.5, respectively. For an S_D site and a Maximum Considered Earthquake (MCE), the mapped values S_{MS} and S_{M1} are 1.5 and 0.91, respectively. The Design Earthquake (DE) mapped values S_{DS} and S_{D1} are, 1.0 and 0.61, respectively.

9.6.2 Site-Specific Spectra

We performed probabilistic seismic hazard analysis and deterministic seismic hazard analysis to develop site-specific response spectra consistent with the provisions of 2010 SFBC and Chapter 21 of ASCE 7-05.

The details of our analysis are presented in Appendix D. We used appropriate Next Generation Attenuation (NGA) relationships to develop the site-specific spectra. Table 5 presents the recommended MCE and DE spectra in accordance with the 2010 SFBC guidelines. Table 6 presents the recommended spectra for the SLE for 2.5 and 5 percent damping.

TABLE 5
Recommended Acceleration Response Spectra (g) per 2010 SFBC
Damping Ratio 5 percent

Period (seconds)	MCE	DE
0.01	0.778	0.519
0.10	1.295	0.863
0.20	1.500	1.000
0.30	1.500	1.000
0.40	1.466	0.978
0.50	1.248	0.832
0.75	0.971	0.647
1.00	0.728	0.485
2.00	0.374	0.249
3.00	0.266	0.177
4.00	0.200	0.133
5.00	0.161	0.108
6.00	0.129	0.086

TABLE 6
Recommended Serviceability Level Earthquake (SLE) Spectra
Spectral Acceleration (g's)

Period (seconds)	Damping Ratio 2.5 percent	Damping Ratio 5.0 percent
0.01	0.186	0.186
0.10	0.390	0.319
0.20	0.500	0.409
0.30	0.467	0.382
0.40	0.396	0.324
0.50	0.312	0.255
0.75	0.190	0.162
1.00	0.132	0.112
2.00	0.050	0.043
3.00	0.029	0.025
4.00	0.020	0.017
5.00	0.014	0.012
6.00	0.010	0.008

9.6.3 Site-Specific Site Coefficients

Because site-specific procedure was used to determine the recommended MCE and DE response spectra, the corresponding values of S_{MS}, S_{M1}, S_{DS} and S_{D1} per Section 21.4 of ASCE 7-05 should be used as shown in Table 7.

TABLE 7
Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g's)
S_{MS}	1.500
S_{M1}	0.748*
S_{DS}	1.000
S_{D1}	0.498*

* 2.0 second spectral values govern

9.6.4 Design Ground Motion Time Histories

We developed scaled time histories in accordance with provisions of 2010 SFBC and ASCE 7-05. The time histories may be scaled by 2/3, if they are used for the DE level of shaking. The details of our analysis are presented in Appendix D. Digitized values of the scaled time histories for the MCE level of shaking are included on the attached CD-Rom.

9.7 Utilities and Utility Trenches

The design of the underground utilities should consider earthquake-induced settlement of up to about four inches may occur in the fill surrounding the site. Flexible utility connections that can accommodate differential movement between the ground and the proposed structure should be used.

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 sheet piling is used as shoring for trenches and is to be removed after backfilling, it should be placed a minimum of two feet away from the pipes or conduits to prevent disturbance to them as the sheet piles are extracted. Where trenches extend below the groundwater level, 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. Backfill deeper 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 on-site soil and any proposed import material is suitable for use as fill.

Utility backfilling should comply with Section 703 of the City of San Francisco Standard Specifications, except jetting of trench backfill is not permitted. Special care should be taken in controlling utility backfilling in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to exterior improvements.

9.8 Shoring

The proposed excavation will need to be shored. The shoring should be designed to limit ground deformations to less than an inch.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. The contractor or his designer should be responsible for determining the type and size of bracing and struts required to resist the given pressures.

We recommend that a rigid, relatively impermeable shoring wall with internal bracing be used to support the sides of the proposed excavation. Based on our experience with similar project, three shoring systems may be used at this site. They are:

- mixed-in-place soil-cement walls
- concrete-diaphragm walls, and/or
- secant pier walls.

The proposed shoring system should be designed using the same earth pressures as recommended for the static condition for the permanent basement walls (see Section 9.6), including traffic and foundation surcharges. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled material is within a distance equal to the shoring depth. Construction equipment should not be allowed within ~~five feet from the edge of the excavation~~ unless the shoring is specifically designed for the surcharge. The increase in pressure should be determined after the surcharge loads are known. The anticipated deflections of the shoring system should be estimated to check if they are acceptable. The shoring system should be sufficiently rigid to prevent detrimental movement and possible damage to adjacent streets, utilities and structures. Passive pressure below the bottom of the excavation may be calculated using an equivalent fluid weight of 400 pcf and 200 pcf above and below the groundwater level, respectively.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. The contractor or his designer should be responsible for determining the type and size of bracing and struts required to resist the given pressures.

Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. Internal bracing should be installed as close to the time of excavation as possible. Excavation should not proceed below a level of bracing until the all bracing at that level has been installed. Jacking (preloading) of the bracing against the sides of the excavation can reduce movement of the shoring.

The shoring system should be designed by a licensed engineer, experienced in the design of shoring. The shoring engineer should be responsible for the design of temporary shoring in accordance with applicable regulatory requirements.

We recommend both Treadwell & Rollo and SOM Engineers review shoring plans. In addition, a representative from our office should observe the installation of the shoring system.

9.9 Dewatering

The groundwater should be drawn down so that the groundwater level in the soil layers below the base of the excavation is at least three feet below the bottom of the excavation. These levels should be maintained until sufficient building weight and/or uplift capacity is available to resist the hydrostatic uplift pressure of the groundwater based on a design groundwater elevation of -3 feet. The structural engineer should evaluate and provide recommendations when the dewatering system can be turned off. The number and depth of dewatering wells should be determined by a specialty dewatering contractor. The volume of water discharged should be monitored and a record of the amount should be submitted to the owner.

9.10 Construction Monitoring

To monitor ground movements, groundwater levels, and shoring movements, we recommend installing the instrumentation listed below:

> **Slope indicators (inclinometers)**: We recommend installing at least six slope indicators. Where possible, a slope indicator should be installed behind each of the exterior walls. The remaining two slope indicators should be embedded in the shoring walls along the north and east sides of the site.
>
> **Piezometers:** One piezometer should be installed behind each exterior shoring wall. The piezometers should each have two casings, one to measure groundwater level in the

fill and Marine Deposits and the other in the Colma Sand. The upper portions of the piezometers should be properly sealed with cement-bentonite mix to reduce surface water infiltration.

Survey points: Survey points should be installed on the adjacent buildings and streets that are within 50 feet of the site.

The instrumentation should be read regularly and the results should be reviewed in a timely manner. Initially, the instrumentation should be read weekly. The frequency of readings may, in the later stage of construction, be modified as appropriate. In addition, the conditions of existing buildings within 50 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction.

10.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Prior to construction, we should review the project plans and specifications as required by the City and County of San Francisco for building permit approval. This will allow us to check conformance with the intent of our recommendations.

During construction, an engineer from our office should observe installation of inclinometers, piezometers, and the shoring system, placement and compaction of any backfill, and the excavation for basement and mat foundation. These observations will allow us to compare actual with anticipated soil conditions and verify that the contractors work conforms to the geotechnical aspects of the plans and specifications.

11.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited subsurface 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 should be notified so that supplemental recommendations can be made.

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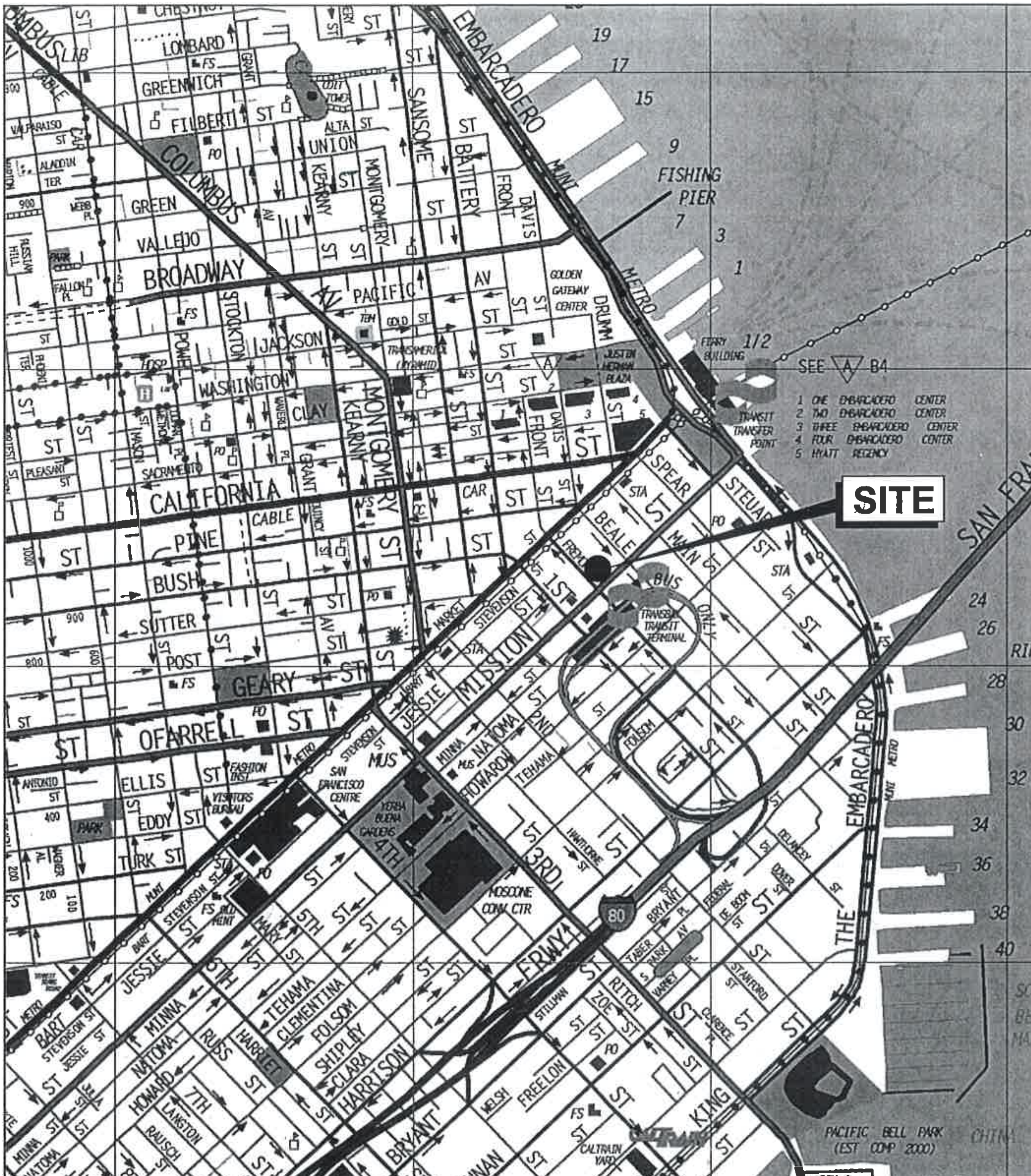
FIGURES



Base map: The Thomas Guide
San Francisco County
1999

0 1/4 1/2 Mile

Approximate scale

N

350 MISSION STREET
San Francisco, California

Treadwell&Rollo
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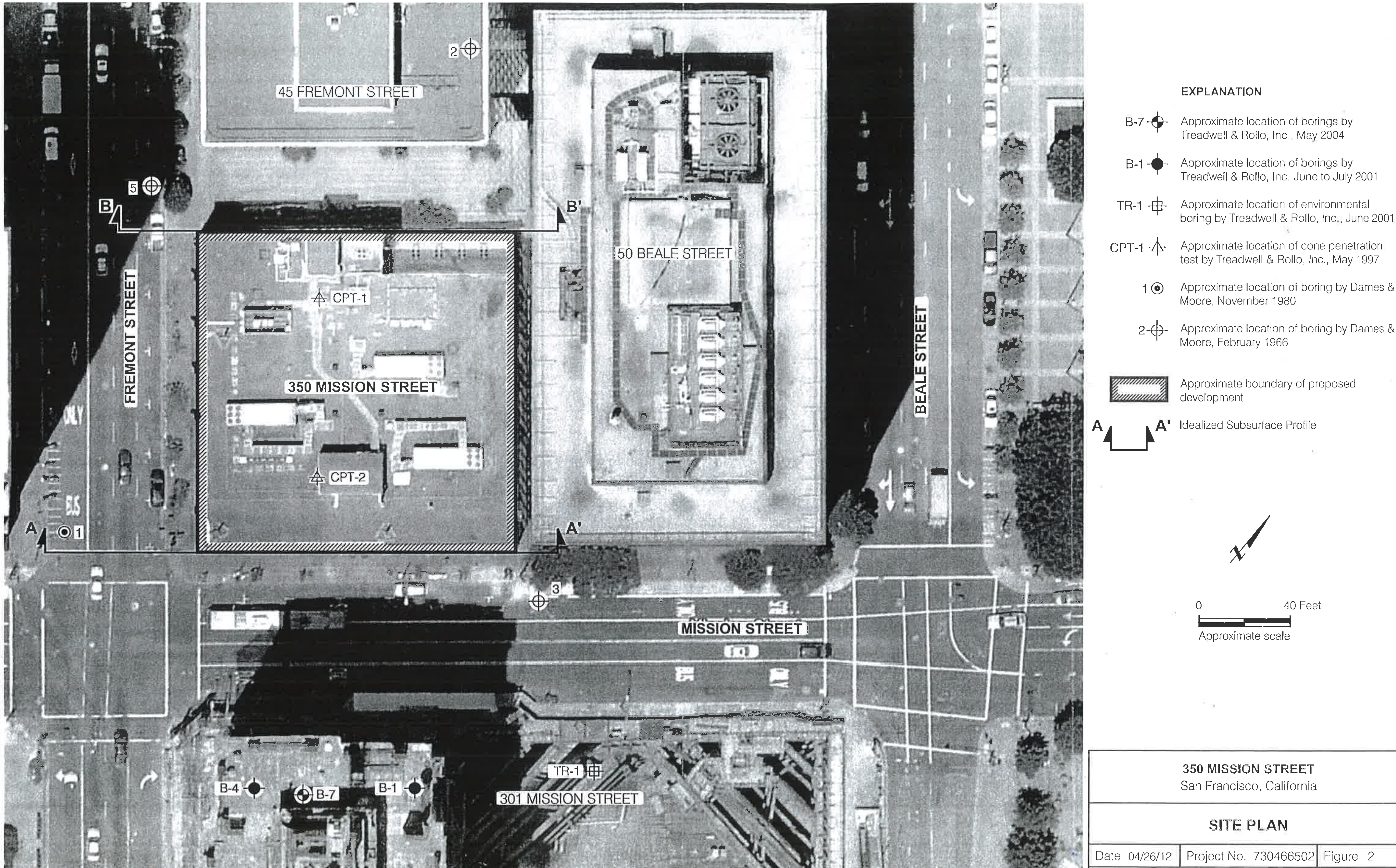
SITE LOCATION MAP

Date 04/26/12	Project No. 730466502	Figure 1



Base Map: Google Earth Pro, 2012.

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350 MISSION STREET
San Francisco, California

SITE PLAN

Date 04/26/12	Project No. 730466502	Figure 2

Treadwell&Rollo
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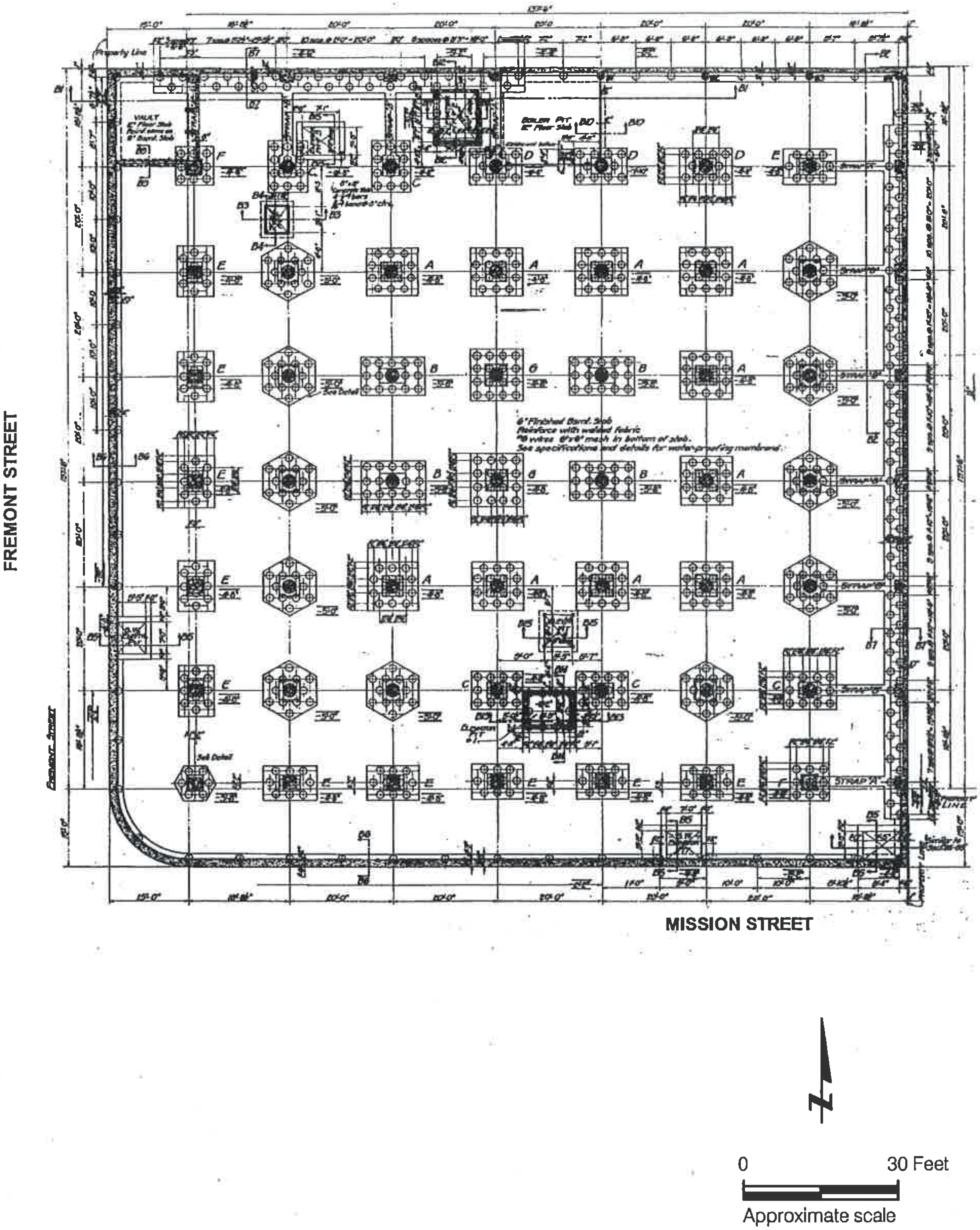
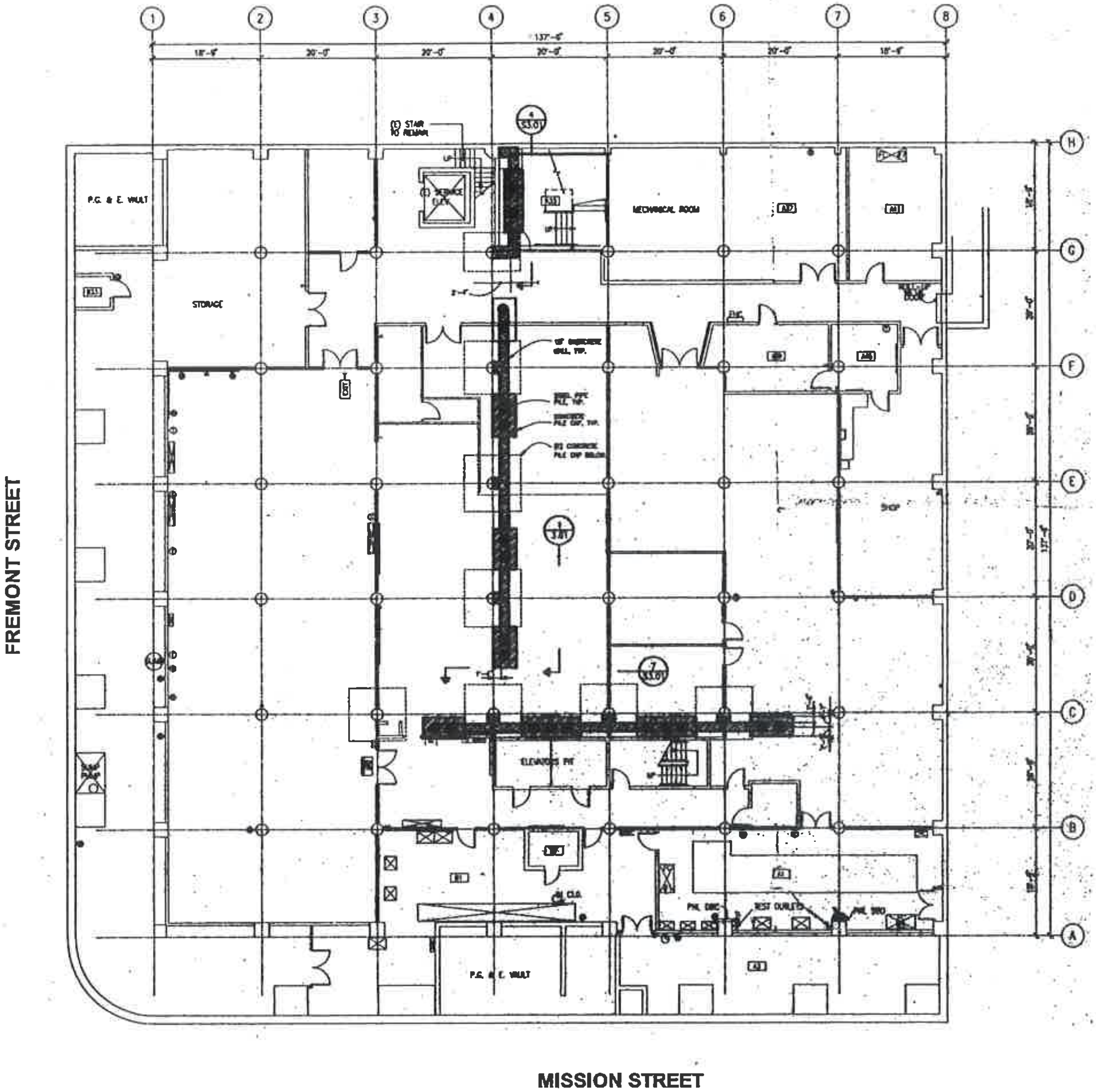


Reference: Base map from a drawing titled "Structural Design Fremont and Mission Building, Sheet S1", by H.J. Brunnier, dated Circa 1920.

350 MISSION STREET San Francisco, California	**APPROXIMATE LOCATIONS OF EXISTING WOODEN PILES**
Treadwell&Rollo A LANGAN COMPANY	Date 04/26/12 | Project No. 730466502 | Figure 3



EXPLANATION

New piles/shear walls

0 30 Feet

Approximate scale

Reference: Base map from a drawing titled "350 Mission Street Renovation, Sheet S2.00", by EQE International, dated 1997.

350 MISSION STREET
San Francisco, California

APPROXIMATE LOCATIONS OF EXISTING STEEL PIPE PILES

Treadwell&Rollo
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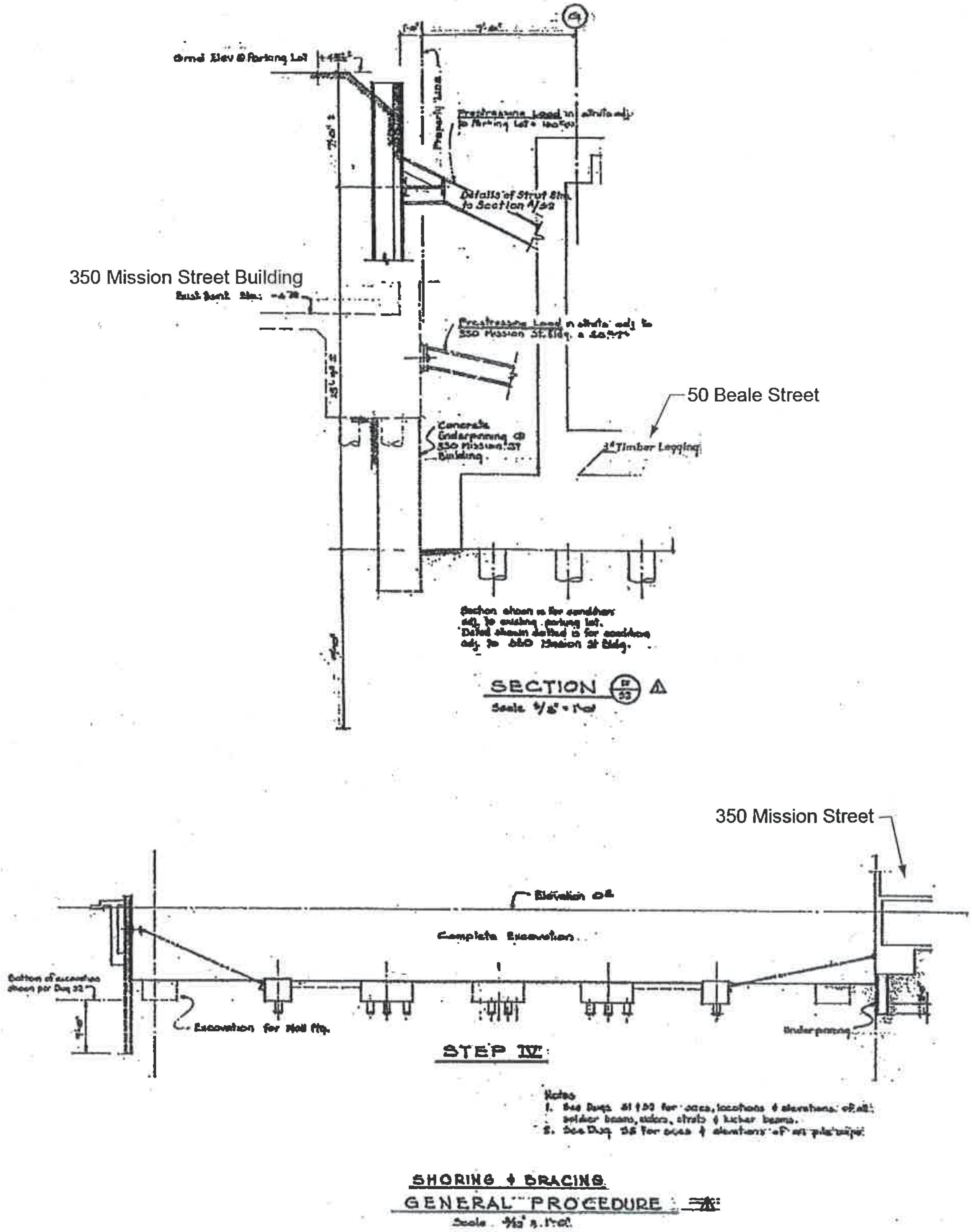
Date 04/26/12	Project No. 730466502	Figure 4



350 MISSION STREET
San Francisco, California

Treadwell&Rollo
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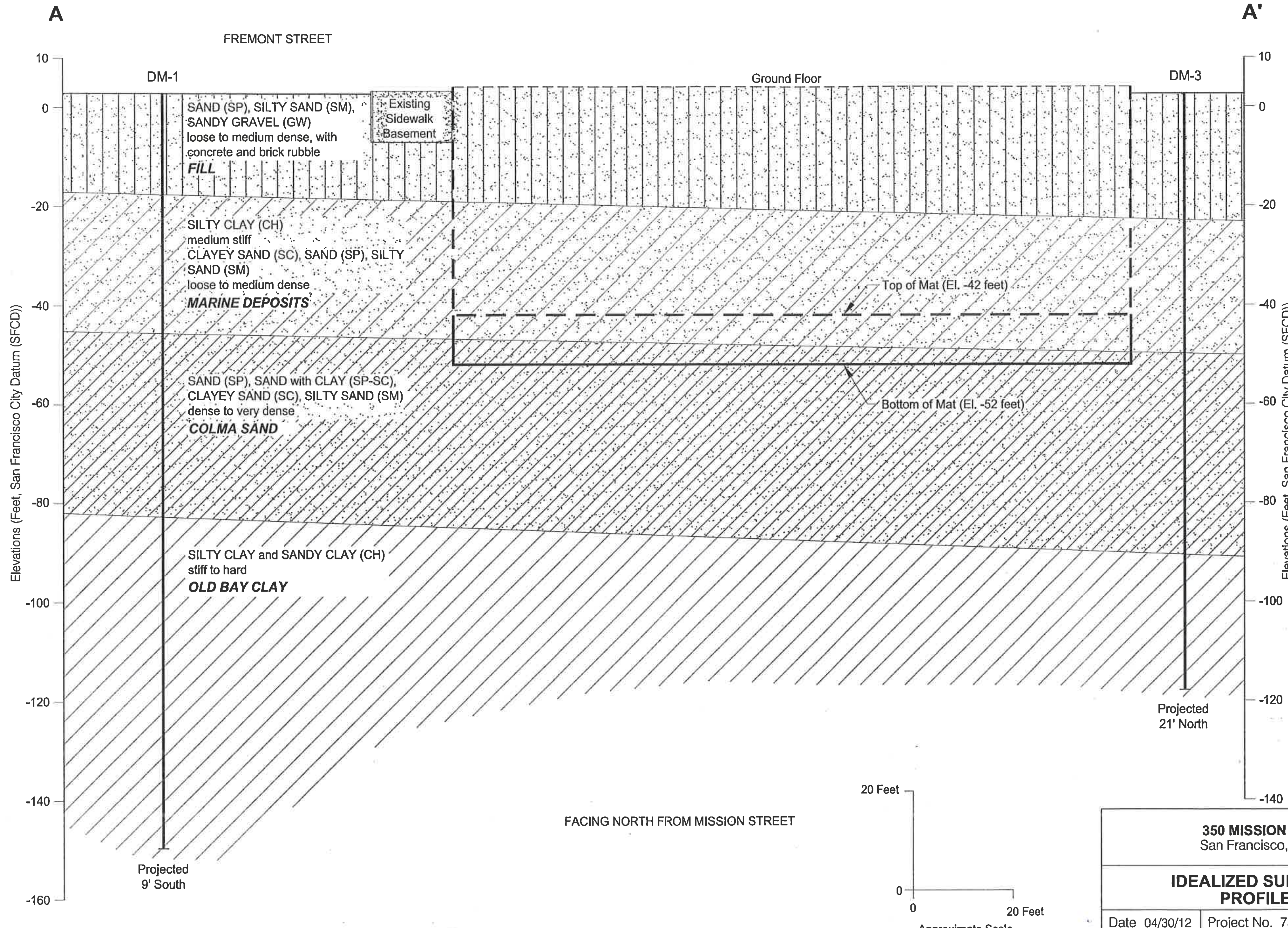
PREVIOUS SHORING AND UNDERPINNING SYSTEM INSTALLED CIRCA 1960

Date 04/26/12	Project No. 730466502	Figure 5



Notes:
The above profile represents a generalized soil cross section interpreted from widely spaced borings.
Soil deposits may vary in type, strength, and other important properties between points of exploration.

350 MISSION STREET
San Francisco, California

IDEALIZED SUBSURFACE PROFILE A-A'

Date 04/30/12	Project No. 730466502	Figure 6

Treadwell&Rollo
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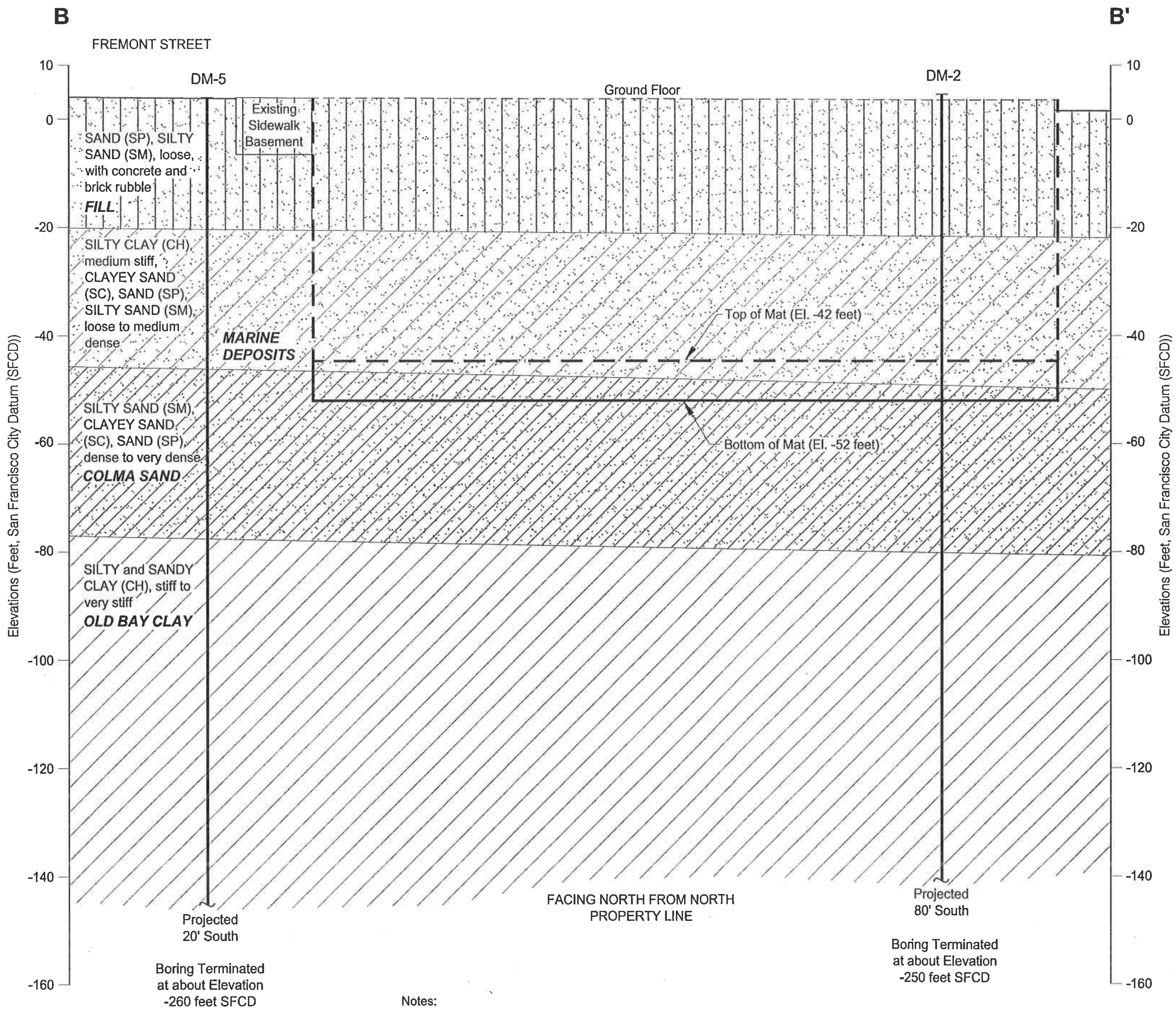

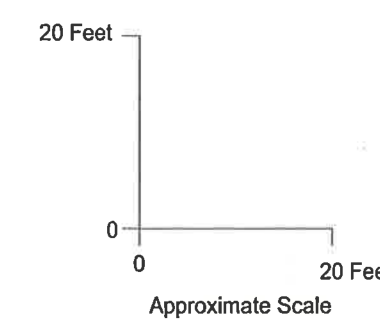

Notes:
The above profile represents a generalized soil cross section interpreted from widely spaced borings.
Soil deposits may vary in type, strength, and other important properties between points of exploration.

350 MISSION STREET San Francisco, California		
IDEALIZED SUBSURFACE PROFILE B-B'		
Date 04/30/12	Project No. 730466502	Figure 7

Treadwell&Rollo
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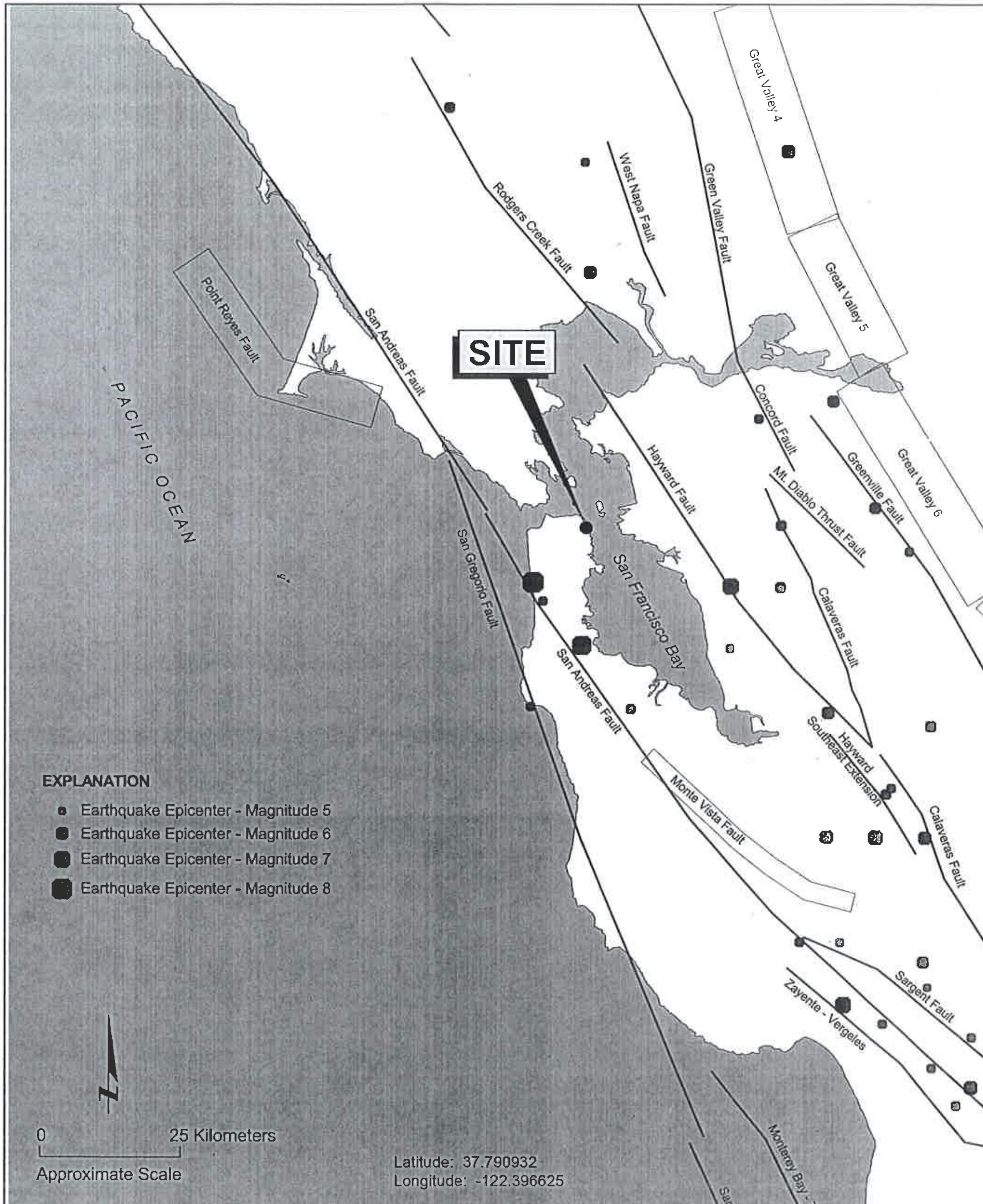


NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Geological Survey (formerly CDMG).
The historic earthquake catalog includes events from January 1800 to December 2000.

350 MISSION STREET San Francisco, California	MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA
Treadwell&Rollo A LANGAN COMPANY	Date 04/26/12 | Project No. 730466502 | Figure 8

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.

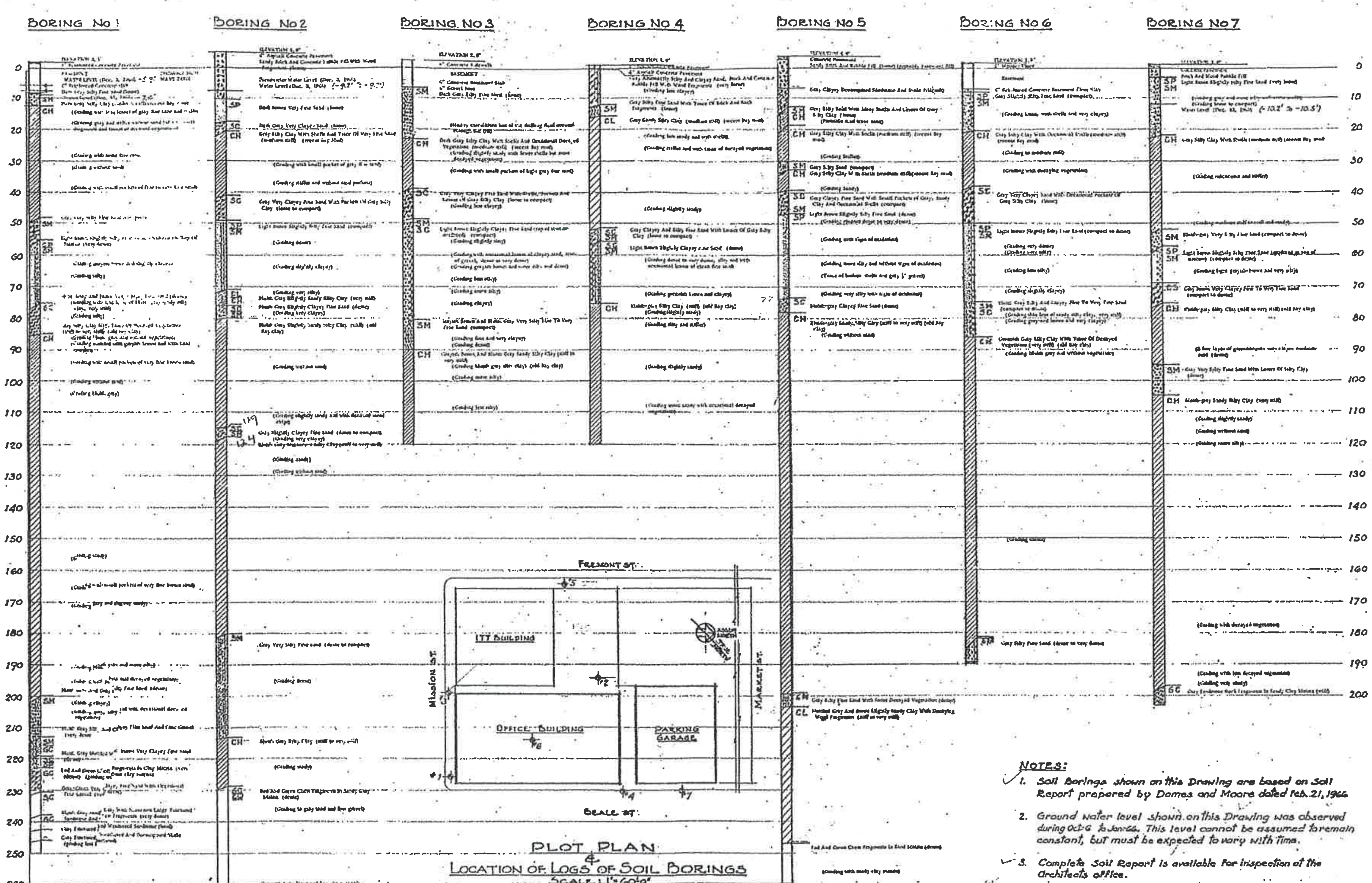
X **Panic is general.**
Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water 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.

APPENDIX A

Previous Boring Logs by Dames & Moore



NOTES:

1. Soil Borings shown on this Drawing are based on Soil Report prepared by Dames and Moore dated Feb. 21, 1966.
2. Ground water level shown on this Drawing was observed during Oct. 6 to Jan. 66. This level cannot be assumed to remain constant, but must be expected to vary with time.
3. Complete Soil Report is available for inspection at the Architects office.
4. Elevations are relative to San Francisco Datum.

Reference: Base map from a drawing titled "Bechtel Corporation San Francisco Office Building, Sheet S7" by SOM, dated 17 January 1967.

350 MISSION STREET
San Francisco, California

DAMES & MOORE BORINGS DRILLED FOR THE 50 BEALE STREET INVESTIGATION

Date 04/26/12 | Project No. 730466502 | Figure A-1

Treadwell&Rollo
A LANGAN COMPANY

BORING 1

DATE DRILLED 11/12-13/80
SURFACE ELEVATION +2.7'

LABORATORY TEST DATA / SAMPLING

DEPTH IN FEET	TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	STRENGTH TEST DATA: TYPE OF STRENGTH TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE, PSF	STRENGTH TEST DATA: SHEAR STRENGTH, PSF	MOISTURE CONTENT, %	DRY DENSITY, PCF	SAMPLING: TYPE OF SAMPLER	SAMPLING: SAMPLING RESISTANCE
0–10				DS/CD DS/CD	600 1200	600 1080	9	116	U	13
10–20				DS/CD DS/CD	1500 3000	1340 2130	20	110	U	20
20–30				TX/UU	1700	830	43 40	79 78	P	200 PSI
30–40							22	104	TW	39
50–60	-200 (7%)						19	110	U	100/6"
	-200 (15%)						20		SPT	78
60–70							20	113	U	80/6"
	-200 (19%)						18		SPT	97
70–80	-200 (16%)						22	111	U	90/6"
	-200 (13%)						21		SPT	119
80–90							18	116	U	68/6"
90–100	C	53	23	TX/UU	3500	2740	31 33 32	90 89 91	P	600 PSI
100–110				TX/UU	4000	2160	42 44	66 77	TW	32
120–130	C	64	28	TX/UU	4600	2800	44 45 44	76 76 77	P	400 PSI
140–150				TX/UU	5300	4330	42	78	P	400 PSI

SYMBOLS / DESCRIPTION

SYMBOLS	DESCRIPTION
GC	3" ASPHALTIC CONCRETE & 4" CONCRETE BROWN GRAVELLY SAND WITH SOME CLAY (MEDIUM DENSE)
SP	BROWN FINE TO MEDIUM SAND WITH SCATTERED GRAVEL (MEDIUM DENSE) (GRADING WITH SOME CRUSHED ROCK)
GW	DARK GRAY TO BLACK FINE TO COARSE GRAVEL & ROCK (MEDIUM DENSE)
SP	BLACK FINE SAND WITH TRACE OF SILT & SOME SHELLS (MEDIUM DENSE) (GRADING CLAYEY)
CH	BLACK TO GRAY SILTY CLAY WITH TRACE OF ORGANICS (MEDIUM STIFF) (GRADING WITH SOME SAND SEAMS)
SP/SC	GRAY FINE TO MEDIUM SAND WITH POCKETS OF GRAY SILTY CLAY & TRACE OF GRAVEL (DENSE)
CH	GRAY SILTY CLAY WITH TRACE OF SAND (GRADING SANDIER) (GRADING MORE CLAYEY)
SP	GRAY FINE SAND (VERY DENSE)
SP/SC	MOTTLED LIGHT GRAY & BROWN FINE SAND TO CLAYEY FINE SAND WITH SOME CEMENTATION (VERY DENSE)
SP	BROWN TO REDDISH-BROWN FINE SAND (VERY DENSE) (GRADING BROWN & FINE TO MEDIUM)
SP/SM	BROWN & REDDISH-BROWN FINE SAND TO SILTY FINE SAND WITH TRACE OF CLAY (CEMENTED) (VERY DENSE)
SP/SC	GRAYISH-BROWN FINE SAND TO CLAYEY FINE SAND (PARTIALLY CEMENTED) (VERY DENSE) (GRADING MORE CLAYEY)
SP/SM	DARK GRAY VERY FINE TO FINE SAND TO SILTY SAND (VERY DENSE)
CH	GREENISH-GRAY SILTY CLAY WITH SCATTERED LENSES OF SAND (VERY STIFF) (GRADING WITH LAYER OF FINE SAND) (GRADING WITH ORGANIC MATERIAL, WOOD & SHELL FRAGMENTS) (GRADING SANDY) (GRADING GRAY) (GRADING SANDY) (GRADING HARD) (GRADING DARK GRAY)

GENERAL NOTES:
1. ELEVATIONS REFER TO CITY OF SAN FRANCISCO DATUM
2. BLOW COUNTS SHOWN ARE FOR THE LAST 12 INCHES (OR PORTION THEREOF) OF A TOTAL 18 INCHES PENETRATION OF THE SAMPLER. THE U-SAMPLER WAS DRIVEN WITH SLIP-JARS WEIGHTING 340 POUNDS AND FALLING 18 INCHES. THE SPT-SAMPLER WAS DRIVEN WITH A 140 POUND HAMMER FALLING 30 INCHES.

350 MISSION STREET
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

DAMES & MOORE BORING DRILLED FOR THE 50 FREMONT STREET INVESTIGATION

Date 06/19/12	Project No. 730466502	Figure A-2

Symbol	Description	Major Divisions		
GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	CLEAN GRAVELS (little or no fines)	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	COARSE-GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE
GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES			
GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	GRAVELS WITH FINES (appreciable amount of fines)		
GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES			
SW	WELL-GRADED SAND OR GRAVELLY SANDS, LITTLE OR NO FINES	CLEAN SANDS (little or no fines)	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	
SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES			
SM	SILTY SANDS, SAND-SILT MIXTURES	SAND WITH FINES (appreciable amount of fines)		
SC	CLAYEY SANDS, SAND-CLAY MIXTURES			
ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		FINE-GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS			
OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY			
MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		
CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS			
PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	HIGHLY ORGANIC SOILS		

For visual classification, the 1/4" size may be used as equivalent to the No. 4 sieve size.

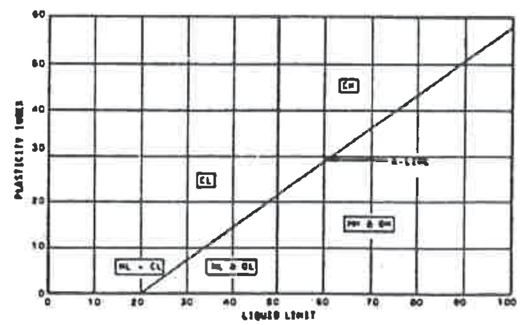
The No. 200 U.S. standard sieve size is about the smallest particle visible to the naked eye

SOIL CLASSIFICATION CHART

UNIFIED SOIL CLASSIFICATION SYSTEM

SYMBOL	TYPE OF TEST
M	MOISTURE
QD	QUICK MD TEST BASED ON ASSUMED SPECIFIC GRAVITY
MD	MOISTURE-DENSITY
CD	CHUNK DENSITY ON BULK SAMPLE
RD	RELATIVE DENSITY
COMP	COMPACTION CURVE
CI	CALIFORNIA IMPACT
CC	COMPACTED CORE
G	SPECIFIC GRAVITY
pH	HYDROGEN ION CONCENTRATION
MA (10%)	MECHANICAL ANALYSIS* (INDICATES PERCENT PASSING #200 SIEVE)
SA (10%)	SIEVE ANALYSIS (+200 ONLY) (INDICATES PERCENT PASSING #200 SIEVE)
HA	HYDROMETER ANALYSIS (-200 ONLY)
AL	ATTERBERG LIMITS (LL & PL)
SL	SHRINKAGE LIMIT
FS	FREE SWELL
SS	SHRINK-SWELL
EXP	EXPANSION
C	CONSOLIDATION
VC	VIBRATING CONSOLIDATION
P	PERMEABILITY
UC	UNCONFINED COMPRESSION
	FRICTION TEST
F/UU	1. UNCONSOLIDATED-UNDRAINED
F/CU	2. CONSOLIDATED-UNDRAINED
F/CU/M$	3. CU/MULTIPHASE**
F/CU/PP	4. CU/WITH PORE PRESSURE MEASUREMENTS
F/CD	5. CONSOLIDATED-DRAINED
	DIRECT SHEAR TEST
DS/UU	1. UNSOLIDATED - UNDRAINED
DS/CU	2. CONSOLIDATED-UNDRAINED
DS/CD	3. CONSOLIDATED-DRAINED
DS/CD/M$	4. CD/MULTIPHASE **
LV	TORVANE SHEAR (LAB VANE SHEAR)

* INCLUDES COMPLETE ANALYSIS, SIEVING AND HYDROMETER
** SERIES OF TESTS RUN ON SAMPLE



PLASTICITY CHART

- INDICATES DEPTH OF UNDISTURBED SAMPLE
- INDICATES DEPTH OF DISTURBED SAMPLE
- INDICATES DEPTH OF SAMPLING ATTEMPT WITH NO RECOVERY
- INDICATES DEPTH OF STANDARD PENETRATION TEST
- INDICATES DEPTH OF STANDARD PENETRATION TEST WITH NO RECOVERY
- INDICATES DEPTH & LENGTH OF CORING RUN
- INDICATES DEPTH OF FIELD VANE SHEAR TEST

NOTE:
UNLESS OTHERWISE NOTED SAMPLING RESISTANCE IS MEASURED IN BLOWS PER FOOT REQUIRED TO DRIVE SAMPLER 12-INCHES AFTER SAMPLER HAS BEEN SEATED 6-INCHES. THE WEIGHT & DROP OF THE DRIVING MECHANISM IS NOTED ON EACH LOG OF BORING.

KEY TO SAMPLES

D - DAMES & MOORE, TYPE D SAMPLER
P - DAMES & MOORE PISTON SAMPLER
U - DAMES & MOORE TYPE U SAMPLER
PT - PITCHER TUBE SAMPLER
NX - NX CORE SAMPLER
TW - DAMES & MOORE TYPE U SAMPLER WITH THIN WALL ATTACHMENT
SPT - STANDARD PENETRATION TEST SAMPLER
ST - SHELBY TUBE SAMPLER

KEY TO SAMPLERS

350 MISSION STREET San Francisco, California	**DAMES & MOORE CLASSIFICATION CHART**
Treadwell&Rollo *A LANGAN COMPANY*	Date 06/19/12 | Project No. 730466502 | Figure A-3

Treadwell&Rollo
A LANGAN COMPANY

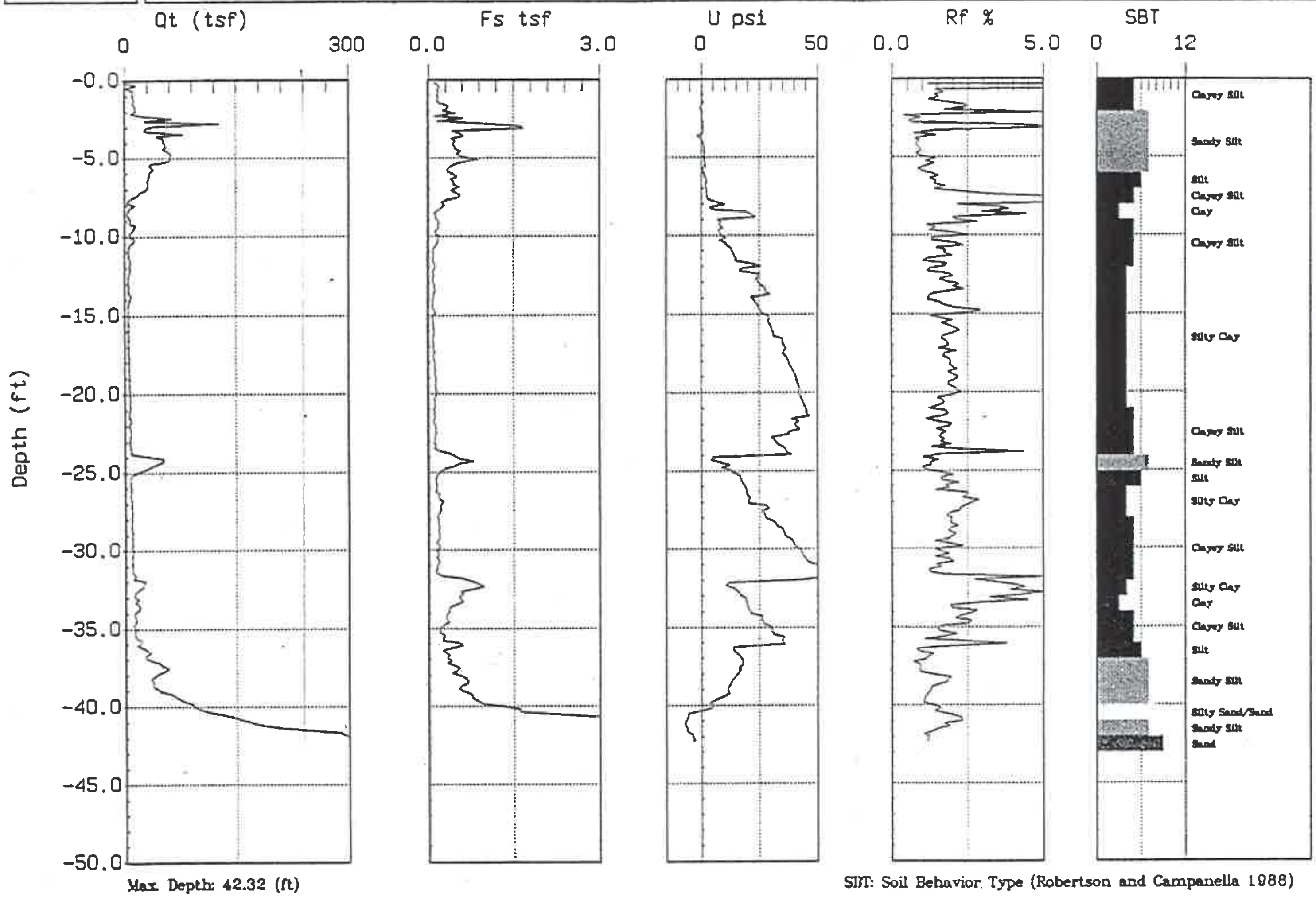
APPENDIX B

Previous Cone Penetration Test Results by Treadwell & Rollo, Inc.

TREADWELL & ROLLO

Site : 350 MISSION
Location : CPT-1

Engineer : R. ROGERS
Date : 05:23:97 12:14



Max. Depth: 42.32 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

Zero corresponds to the top of the basement floor slab.

350 MISSION STREET
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

CONE PENETRATION TEST RESULTS
CPT-1

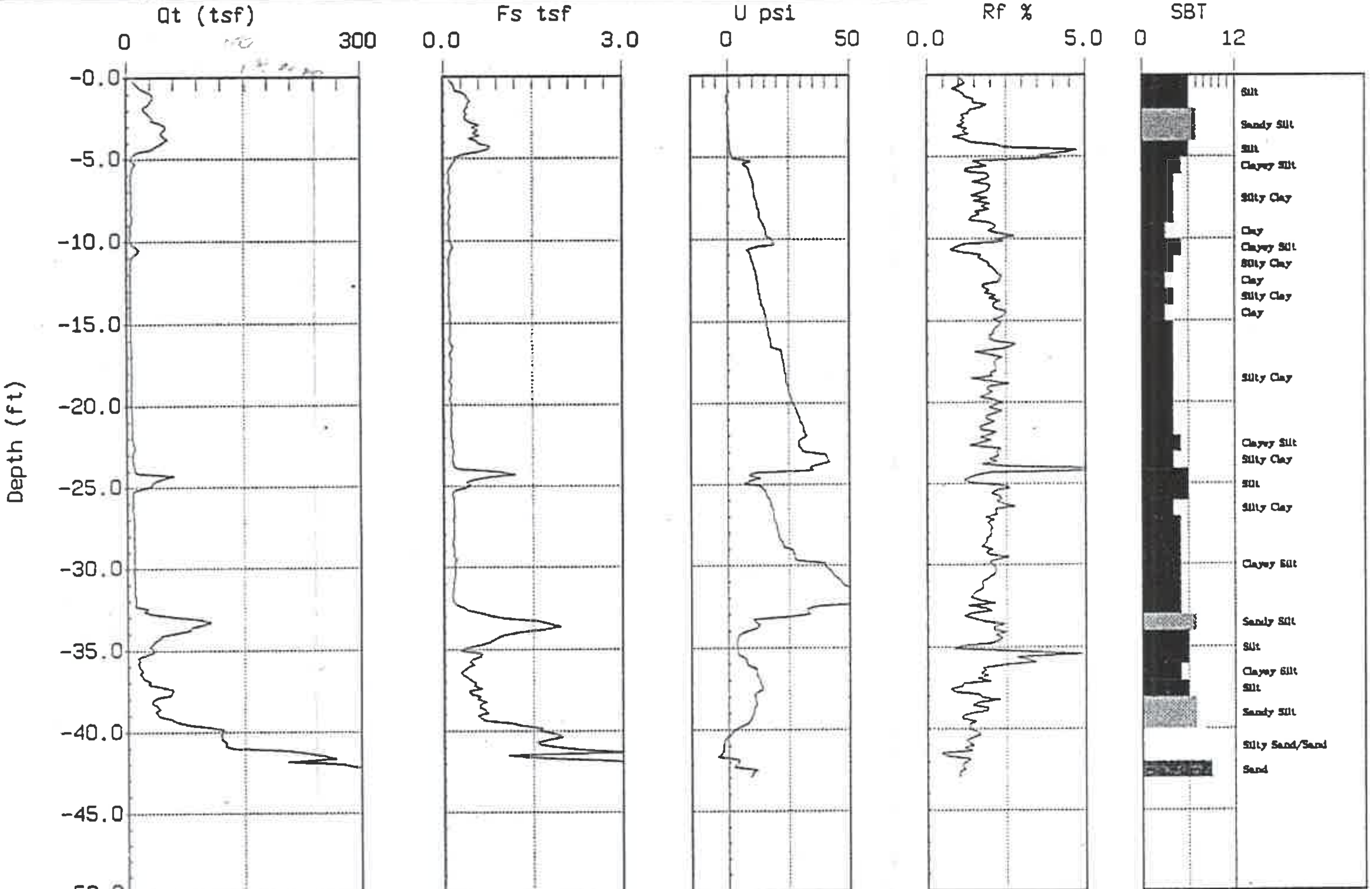
Date 04/26/12	Project No. 730466502	Figure B-1

GREGG

TREADWELL & ROLLO

Site : 350 MISSION
Location : CPT-2

Engineer : R. ROGERS
Date : 05:23:97 13:58



Max. Depth: 42.98 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

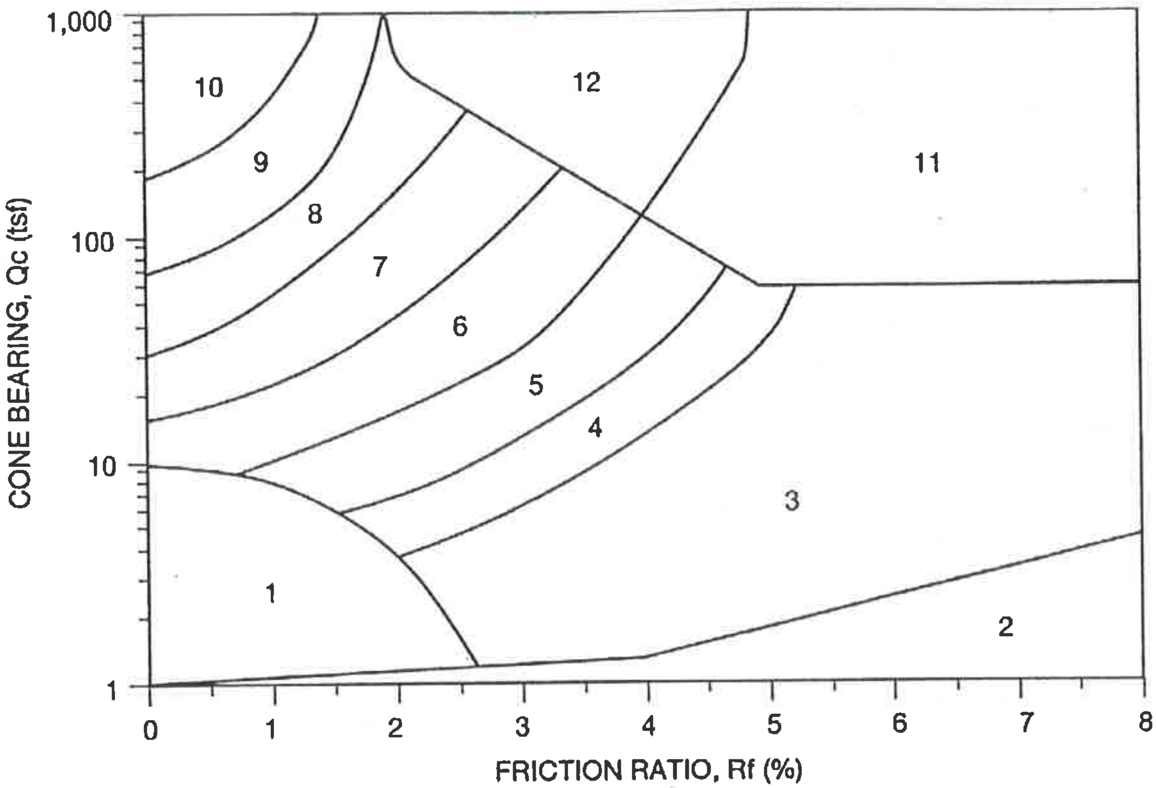
Zero corresponds to the top of the basement floor slab.

350 MISSION STREET
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

CONE PENETRATION TEST RESULTS
CPT-2

Date 04/26/12 | Project No. 730466502 | Figure B-2



ZONE	Qc/N[1]	Su Factor (Nk) [2]	SOIL BEHAVIOR TYPE [1]
1	2	15	Sensitive Fine-Grained
2	1	15	Organic Material
3	1	15	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	—	SILTY SAND to SANDY SILT
8	4	—	SAND to SILTY SAND
9	5	—	SAND
10	6	—	GRAVELLY SAND to SAND
11	1	15	Very Stiff Fine-Grained (*)
12	2	—	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented
Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc x 100 = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

References: 1. Robertson, 1986, Olsen, 1988.
2. Estimated from local experience.

350 MISSION STREET San Francisco, California	SIMPLIFIED SOIL BEHAVIOR TYPE CLASSIFICATION FOR STANDARD ELECTRONIC CONE PENETROMETER
Treadwell&Rollo A LANGAN COMPANY	Date 04/26/12 | Project No. 730466502 | Figure B-3

APPENDIX C

Previous Borings by Treadwell & Rollo, Inc.

PROJECT:	301 MISSION STREET San Francisco, California	Log of Boring B-1 PAGE 1 OF 4

Boring location: See Site Plan, Figure 2

Logged by: R. Nelson

Date started: 6/28/01 | Date finished: 6/29/01

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30-inches | Hammer type: Safety, rope & pulley

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

Ground Surface Elevation: 3.5 feet[2]

DEPTH (feet)	Sampler Type	Sample	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
0–11					GP	SANDY GRAVEL (GP) light brown, loose, dry, with concrete and brick debris	FILL						
11						CONCRETE SLAB 6-inches thick							
12–23					GP	SANDY GRAVEL (GP) light brown, loose, moist, with wood and concrete debris ▽ unstabilized groundwater level at 13 feet noted during drilling							
13	S&H			30/3"									
21	S&H			4									
23–30					CH	CLAY with SAND (CH) gray, very soft to soft, wet, with shells							
26	O			50 psi								52.9	69

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 730466502 | Figure: C-1a

TEST GEOTECH LOG 730466502 G.GPJ TR.GDT 4/27/12

PROJECT:	301 MISSION STREET San Francisco, California	Log of Boring B-1 PAGE 2 OF 4

DEPTH (feet)	Sampler Type	Sample	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
						CLAY with SAND (CH) (continued)						
31	S&H			0								
32–35												
36	O			50 psi			TxUU	1,400	685		37.1 35.0	85 87
37					CH	Consolidation Test, See Figure C-8						
38–40												
41	S&H			2							48.4	72
42–44												
44						SAND (SP) gray, very dense, wet						
46	SPT			51	SP							
47–48												
48						CLAYEY SAND (SC) gray, medium dense, wet						
51	SPT			13		LL=17, PI=9, See Figure C-1				19	24.1	
53					SC							
54–58												
58						CLAYEY SAND (SC) olive-gray, dense, wet						
59					SC							
60												

TEST GEOTECH LOG 730466502 G.GPJ TR.GDT 4/27/12

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 730466502	Figure: C-1b

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61–67	S&H			43	SC	CLAYEY SAND (SC) (continued)					16.2	119
67–78	S&H			30/5"	SC	CLAYEY SAND (SC) yellow-brown, very dense, wet						
78–88	S&H			30/5"	SM	SILTY SAND (SM) olive-brown, very dense, wet					18.7	116
88–90					CL	CLAY (CL) **[OLD BAY CLAY]** gray, very stiff, wet						

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 730466502
Figure: C-1c

TEST GEOTECH LOG 730466502 G.GPJ TR.GDT 4/27/12

PROJECT:	301 MISSION STREET San Francisco, California	Log of Boring B-1 PAGE 4 OF 4

DEPTH (feet)	Sampler Type	Sample	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
						CLAY (CL) (continued)						
91	O			100 psi			TxUU	3,000	1,910		20.5	110
92												
93												
94												
95												
96					CL							
97												
98												
99												
100						green-gray, hard						
101	S&H			34								
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at 101.5 feet below ground surface.
Boring backfilled with cement grout.
Unstabilized groundwater encountered at 13 feet during drilling.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

Treadwell&Rollo
A LANGAN COMPANY

Project No.:	Figure:
730466502	C-1d

PROJECT: **301 MISSION STREET** San Francisco, California

Log of Boring B-4

Boring location:	See Site Plan, Figure 2
Date started: 6/27/01	Date finished: 6/28/01
Drilling method:	Rotary Wash
Hammer weight/drop: 140 lbs./30-inches	Hammer type: Safety, rope & pulley
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Osterberg (O)
Logged by:	R. Nelson

Ground Surface Elevation: 3.5 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION		LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–11					GP	SANDY GRAVEL (GP) gray-brown, dry, with concrete and brick debris	FILL						
11						CONCRETE SLAB 7.5-inches thick	FILL						
12–18						RUBBLE loose, concrete, brick	FILL						
16	S&H			5									
18–24					CH	SANDY CLAY (CH) dark gray, soft, wet							
21	O			50 psi									
24–30					CH	CLAY with SAND (CH) gray, soft, wet, with shells							
26	O			50 psi								47.0	71

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 730466502

Figure: C-2a

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAY with SAND (CH) (continued)						
31	O			50 psi							33.2	86
32					CH							
33												
34						CLAYEY SAND (SC) gray, medium dense, wet						
35												
36	O			75 psi			TxUU	1,400	980	19	24.0	103
37												
38												
39					SC							
40												
41	S&H			19						24	25.4	101
42												
43												
44						SAND (SP) green-gray, very dense, wet						
45	S&H			30/5"								
46					SP							
47												
48												
49						SILTY SAND (SM) gray, medium dense, wet LL=17, PI=4, See Figure C-1						
50												
51	SPT			12						21	27.7	
52												
53					SM							
54												
55												
56												
57												
58						CLAYEY SAND (SC) green-gray, medium dense, wet						
59					SC							
60												

PROJECT:	301 MISSION STREET San Francisco, California	Log of Boring B-4

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H			28	SC	CLAYEY SAND (SC) (continued)				14	20.2	111
62												
63												
64												
65												
66												
67												
68												
69												
70												
71												
72												
73												
74												
75												
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79												
80												
81												
82												
83												
84												
85												
86												
87												
88												
89												
90												

Boring terminated at 61.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 730466502

Figure: C-2c

TEST GEOTECH LOG 730466502 G.GPJ TR.GDT 4/27/12

PROJECT:	301 MISSION STREET San Francisco, California	Log of Boring B-7 PAGE 1 OF 8

Boring location: See Site Plan, Figure 2		Logged by: L. Bedolla
Date started: 5/14/04	Date finished: 5/17/04	
Drilling method: Rotary Wash		
Hammer weight/drop: 140 lbs./30-inches	Hammer type: Safety	LABORATORY TEST DATA
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)		

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: +4 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
0–11					SP	SAND with GRAVEL (SP) gray brown, loose, dry, with brick and concrete (FILL)						
11–12						12-inches-thick Concrete Slab (FILL)						
12–17	S&H (15–16)			11	SM	SILTY SAND (SM) dark gray, medium dense, wet, with brick (FILL)						
17–19					CH	CLAY (CH) black, soft to medium stiff, wet, with rubble and organics (FILL)						
19–30					CH	CLAY (CH) gray, soft to medium stiff, wet, trace sand and shells						

Treadwell&Rollo
A LANGAN COMPANY

Project No.: 730466502

Figure: C-3a

TEST GEOTECH LOG 730466502.GPJ TR.GDT 4/27/12

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31–35					CH	CLAY (CH) (continued)						
35–37	ST			100 to 200 psi								
36–41					SM	SILTY SAND (SM) gray, medium dense, wet						
41–60					CH	CLAY with SAND (CH) gray, medium stiff, wet, trace shells						
45						no sand						
48						with sand						
55–56	S&H			4			TV		800			

DEPTH (feet)	Sampler Type	Sample	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
61–65					CH	SANDY CLAY (CH) dark gray to black, medium stiff to stiff, wet						
65–68					CL	SANDY CLAY (CL) green gray, stiff, wet						
68–70					SM	SILTY SAND (SM) yellow brown, dense, wet, pockets of clayey sand and cemented sand						
70–71.5	S&H	■		33	SM	gray yellow brown						
80–81	S&H	■		30/ 6"	SM							
81–88					SM	very dense						
88–90					CL	CLAY with SAND (CL) olive gray, medium stiff to stiff, wet OLD BAY CLAY						

DEPTH (feet)	Sampler Type	Sample	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
91	ST			100 to 400 psi		CLAY with SAND (CL)						
92												
93					SM	SILTY SAND (SM) dark gray, medium dense to dense, wet						
94												
95					CL	SANDY CLAY (CL) olive gray, stiff, wet						
96												
97												
98					CL	CLAY (CL) dark gray, stiff, wet						
99												
100												
101	ST			100 to 200 psi		with silt and fine sand					33.7	90
102							TV		950			
103												
104												
105												
106												
107												
108												
109												
110												
111	ST			100 to 180 psi		less silt and no fine sand					40.2	80
112						Consolidation Test, See Figure B-3	TV		800			
113												
114												
115												
116												
117												
118												
119												
120												

PROJECT:	301 MISSION STREET San Francisco, California	Log of Boring B-7 PAGE 5 OF 8

DEPTH (feet)	Sampler Type	Sample	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
				100 to 200 psi		CLAY (CL) (continued)						
121	ST					OLD BAY CLAY					41.3	81
122							TV		900			
123												
124												
125												
126												
127												
128												
129												
130				100 to 200 psi								
131	ST										43.1	79
132							TV		1,200			
133												
134												
135												
136												
137												
138												
139												
140				100 to 200 psi								
141	ST											
142							TV		1,300			
143												
144												
145												
146												
147												
148												
149												
150												

PROJECT:	301 MISSION STREET San Francisco, California	Log of Boring B-7

DEPTH (feet)	Sampler Type	Sample	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
				100 to 225 psi		CLAY (CL) (continued)	OLD BAY CLAY						
151	ST					Consolidation Test, See Figure B-4						42.4	78
152								TV		1,500			
153													
154													
155													
156													
157													
158													
159													
160													
161	ST			100 to 200 psi								44.1	77
162								TV		1,900			
163													
164													
165					CL								
166													
167													
168													
169													
170						green gray, very stiff, wet, trace sand and organics							
171	ST			100 to 200 psi									
172								TV		2,200			
173													
174													
175													
176													
177													
178													
179													
180													

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Project No.: 730466502

Figure: C-3f

DEPTH (feet)	Sampler Type	Sample	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
181	ST			150 to 200 psi	CL	CLAY (CL) (continued) gray						
182							TV		2,700			
191	ST			0 to 150 psi		Consolidation Test, See Figure B-5					36.7	85
192							TV		2,400			
200–201	ST			0 to 300 psi	CL	SANDY CLAY (CL) gray brown, very stiff, wet, trace organics						
202							TV		2,300			
208–209					SM	SILTY SAND (SM) gray, very dense, wet, trace organics						

OLD BAY CLAY (180 to 200 feet)

Depth scale: 181, 182, 183, 184, 185, 186, 187, 188, 189, 190, 191, 192, 193, 194, 195, 196, 197, 198, 199, 200, 201, 202, 203, 204, 205, 206, 207, 208, 209, 210

PROJECT: **301 MISSION STREET** San Francisco, California

Log of Boring B-7

DEPTH (feet)	Sampler Type	Sample	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
	S&H	■		30/ 3"	SM	SILTY SAND (SM) (continued)						
211–216												
217–220					CL	CLAY (CL) gray, hard, wet						
221–240												

Boring terminated at 220 feet below ground surface.
Boring backfilled with cement grout under the observation of the SFDPH.
Groundwater level was obscured by drilling method.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

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Project No.: 730466502
Figure: C-3h

TEST GEOTECH LOG 730466502.GPJ TR.GDT 4/27/12

PROJECT:	301 MISSION STREET San Francisco, California	Log of Boring TR-1 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2	Logged by: C. Keane
Date started: 7/5/01	Date finished: 7/5/01
Drilling method: Hand Auger	
Hammer weight/drop: ---	Hammer type: ---
Sampler: ---	

DEPTH (feet)	SAMPLES: Sample Number	Sample	Blow Count	Recovery (Inches)	OVM (ppm)	LITHOLOGY	MATERIAL DESCRIPTION
							CONCRETE SLAB
0–1							Concrete core to 6-inches, rubber membrane 1/4" thick, second concrete slab to total of 13-1/2"
1–2.5						SP	SILTY SAND brown, moist, with brick fragments — FILL
2.5–4	TR-1-3.5 TR-1-4.0	■				SP ▽	SAND grey, wet Groundwater encountered at 3 feet

Depth scale: 1–30 feet.

Boring terminated at 4.0 feet.
Boring backfilled with bentonite grout mix.
Groundwater encountered at 3.0 feet.

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Project No.: 730466502
Figure: C-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 0.074
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 0.074
Silt and Clay	Below No. 200	Below 0.074

▽ 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
- Sampling attempted with no recovery
- Core sample
- Analytical laboratory sample
- Sample taken with Direct Push sampler

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)

301 MISSION STREET
San Francisco, California

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CLASSIFICATION CHART

Date 04/26/12 | Project No. 730466502 | Figure C-5

APPENDIX D

Site-Specific Response Spectra and Time Histories

APPENDIX D

SITE-SPECIFIC RESPONSE SPECTRA AND TIME HISTORIES

This appendix presents the details of our estimation of the level of ground shaking at the site of the proposed 350 Mission building in San Francisco, California during future earthquakes. We developed site-specific response spectra and time histories for three levels of shaking corresponding to the Maximum Considered Earthquake (MCE) and the Design Earthquake (DE) in accordance with the 2010 San Francisco Building Code (SFBC) which is consistent with the definitions 2010 California Building Code (CBC) and ASCE 7-05. In addition, we developed a site-specific response spectrum for the Serviceability Level Earthquake (SLE) having a 50 percent probability of exceedance in 30 years. Spectrally scaled time histories were also developed for two levels of shaking corresponding to the MCE and the DE levels of shaking per ASCE 7-05 criteria.

For the development of MCE and DE in accordance with the 2010 SFBC criteria, we performed a Probabilistic Seismic Hazard Analysis (PSHA) and deterministic analysis to develop smooth, site-specific horizontal spectra for two levels of shaking, namely:

- MCE, which corresponds to the lesser of 2 percent probability of exceedance in 50 years or 150 percent of the median deterministic event
- DE, which corresponds to 2/3 of the MCE
- SLE, 50 percent probability of exceedance in 30 years (43-year return period).

Details regarding our study are presented in the following sections of this Appendix.

D1.0 PROBABILISTIC SEISMIC HAZARD ANALYSIS

Because the location, recurrence interval, and magnitude of future earthquakes are uncertain, we performed a PSHA, which systematically accounts for these uncertainties. The results of a PSHA define a uniform hazard for a site in terms of a probability that a particular level of shaking will be exceeded during the given life of the structure.

The probability of exceedance, $P_e(Z)$, at a given ground-motion, Z, at the site within a specified time period, T, is given as:

$$P_e(Z) = 1 - e^{-V(z)T}$$

where V(z) is the mean annual rate of exceedance of ground motion level Z. V(z) can be calculated using the total-probability theorem.

$$V(z) = \sum_i v_i \iint P[Z > z \mid m, r] f_{M_i}(m) f_{R_i|M_i}(r; m)\, dr\, dm$$

where:

ν_i = the annual rate of earthquakes with magnitudes greater than a threshold M_{oi} in source i
P [Z > z | m,r] = probability that an earthquake of magnitude m at distance r produces ground motion amplitude Z higher than z
$f_{Mi}(m)$ and $f_{Ri|Mi}(r;m)$ = probability density functions for magnitude and distance

Z represents peak ground acceleration, or spectral acceleration values for a given frequency of vibration. The peak accelerations are assumed to be log-normally distributed about the mean with a standard error that is dependent upon the magnitude and attenuation relationship used.

D1.2 Source Modeling and Characterization

The segmentation of faults, mean characteristic magnitudes, and recurrence rates were modeled using the data presented in the WGCEP (2008) and Cao et al. (2003) reports. We also included the combination of fault segments and their associated magnitudes and recurrence rates as described in the WGCEP (2008) in our seismic hazard model. Table D-1 presents the distance and direction from the site to the fault, mean characteristic magnitude, mean slip rate, and fault length for individual fault segments within 100 km from the site. We used the California fault database identified as "USGS08" in EZFRISK 7.62. We understand EZFRISK obtained this database directly from USGS and models the faults with multiple segments. Each segment is characterized with multiple magnitudes, occurrence or slip rates and weights. This approach takes into account the epistemic uncertainty associated with the various seismic sources in our model.

D1.3 Attenuation Relationships

Based on the subsurface conditions, the site is classified as a stiff soil profile, site class D. Using the subsurface information and the shear wave velocity measurements during our field investigation, we estimated the average shear wave velocity of the upper 100 feet of approximately 797 feet per second (243 meters per second).

Recently, Pacific Earthquake Engineering Research Center (PEER) embarked on the NGA project to update the previously developed attenuation relationships which were mostly published in 1997. We used the relationships by Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008) and Chiou and Youngs (2008). These attenuation relationships include the average shear wave velocity in the upper 100 feet. Furthermore, these relationships were developed using different earthquake databases, therefore, the average of the relationships was used to develop the recommended spectra.

The NGA relationships database includes the most up-to-date recorded and processed data. The NGA relationships were developed for the orientation-independent geometric mean of the data. Geometric mean is defined as the square root of the product of the two recorded components.

D1.3.1 Amplification of Long Period Response Spectra and Effects of Depth to Rock on the Results of Attenuation Relationships

On the basis of the subsurface conditions, the project site is categorized as a soil profile type S_D. Furthermore, from a review of available subsurface information in the vicinity we estimate bedrock at the site is at a depth of about 230 feet below the ground surface. It has been recognized that depth to competent rock (i.e. basin depth) has an influence on the recorded data. The basin depth effect increases the long period of recorded data. Because the recorded data used in the development of the NGA relationships are mostly from deep sediment sites (sediment depths on the order of several hundred to thousands of feet), there is a bias towards over amplification of long period spectral values for sites with shallow sediment depths. Ramin check quotation.....I highlighted change is this correct? Abrahamson (2009) indicates that "CA empirical based models not applicable to shallow soil sites". Abrahamson and Silva (2008) account for this effect by including a depth to rock with a shear wave velocity of 1 km/s, $Z_{1.0\ km/sec}$. We assumed the depth to $Z_{1.0\ km/sec}$ of about 100 meters.

San Andreas fault dominates the hazard at the project site at different periods of interest. We conclude that for the deterministic analysis the governing earthquake is a magnitude of about 8.0 occurring at a distance of about 13.3 km from the site.

D2.0 DETERMINISTIC ANALYSIS

We performed a deterministic analysis to develop the MCE spectrum at the site. In a deterministic analysis, a given magnitude earthquake occurring at a certain distance from the source is considered as input into an appropriate ground motion attenuation relationship. The MCE spectrum was defined as an earthquake on the San Andreas fault having a Moment Magnitude of about 8.0 occurring at a distance of about 13.3 km. This is consistent with the deaggregation results discussed in Section D1.4.1.

The same attenuation relationships as discussed in Section D1.3 were used in our deterministic analysis. Figure D-7 presents the median deterministic geometric mean results for rock for the San Andreas event. The average of the four relationships as well as 150 percent of the average of the median results is presented on figure D-7. As noted, these results also include average directivity.

D3.0 RECOMMENDED SPECTRA

The MCE as defined in the 2010 SFBC/ASCE 7-05 is the lesser of the PSHA spectrum having a 2 percent probability of exceedance in 50 years or the 150 percent median deterministic spectrum of the governing earthquake scenario and the DE spectrum is defined as 2/3 times the MCE spectrum.

We followed the procedures outlined in Chapter 21 of ASCE 7-05 to develop the site-specific spectra for MCE and DE. Chapter 21 of ASCE 7-05 requires the following checks:

- the deterministic spectrum used to develop the MCE shall not fall below the Deterministic Lower Limit spectrum as shown on Figure 21.2-1 of ASCE 7-05;
- the DE spectrum shall not fall below 80 percent of general design spectrum (Section 21.3 of Chapter 21 ASCE 7-05).

Figure D-8 presents soil spectra developed from rock spectra using the Abrahamson and Silva (2008) amplification factors as described in Section D1.3.1. Figure D-9 and Table D-2 present a comparison of the site-specific spectra for a 2 percent probability of exceedance in 50 years (2,475 year return period), 80 percent of the general spectrum for Site Class D. For all periods, 80 percent of the general spectrum is equal or lower than 2/3 of the MCE spectrum. Therefore, we recommend that 2/3 of the MCE spectrum be used for DE. The recommended DE spectrum is shown on Figure D-8.

TABLE D-3
Comparison of Site-specific and Code Spectra for Development of DE Spectrum
Per 2007 CBC/ASCE 7-05
S_a (g) for 5 percent damping

Period (seconds)	Recommended MCE	2/3 times MCE	80% of General Design Spectrum	Recommended DE
0.01	0.778	0.519	0.320	0.519
0.10	1.295	0.863	0.717	0.863
0.20	1.500	1.000	0.800	1.000
0.30	1.500	1.000	0.800	1.000
0.40	1.466	0.978	0.800	0.978
0.50	1.248	0.832	0.800	0.832
0.75	0.971	0.647	0.647	0.647
1.00	0.728	0.485	0.485	0.485
2.00	0.374	0.249	0.243	0.249
3.00	0.266	0.177	0.162	0.177
4.00	0.200	0.133	0.121	0.133
5.00	0.161	0.108	0.097	0.108
6.00	0.129	0.086	0.081	0.086

MCE and DE spectra are presented on Figure D-9 and digitized values of the recommended spectrum are presented in Table D-4 for a damping ratio of 5 percent.

Because site-specific procedure was used to determine the recommended MCE and DE response spectra, the corresponding values of S_{MS}, S_{M1}, S_{DS} and S_{D1} per Section 21.4 of ASCE 7-05 should be used as shown in Table D-6.

TABLE D-6
Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g's)
S_{MS}	1.500
S_{M1}	0.748*
S_{DS}	1.000
S_{D1}	0.498*

D4.0 TIME HISTORIES

D4.1 Design Ground Motion Time Histories

To develop time histories that are compatible with the 2010 SFBC recommended MCE spectrum, we performed time domain spectral scaling following the procedure outlined by Kircher & Associates (1996). The scaling factors for each pair of time histories are obtained such that they satisfy the criteria in ASCE 7-05 section 16.1.3.2 where the average of the scaled square root of the sum of the squares (SRSS) of the suite of the time histories (seven) does not fall below 1.3 times the target (MCE) spectrum in the period range of interest by more than 10 percent. The spectral period range of interest is defined as 0.2T to 1.5T where T is the first mode structural period. As discussed with SOM, the project structural engineer, we understand the first mode structural period is about 3.1 seconds. Hence, the spectral period range considered to develop the scaling factors is 0.62 to 4.65 seconds.

The selection of a recorded time history is an important step in developing the ground motion. The intent in this selection process is to choose time histories that have a similar magnitude, distance and fault mechanism as that of the controlling target spectrum. In discussions with SOM and the peer review panel we developed a list of twenty five potential earthquake time histories for the project as presented in Table D-7. These recordings have forward, backward, and neutral directivities and capture the different characteristics. The records were obtained from the NGA PEER data base website. From this list we recommended ten pairs of time histories used in spectral scaling as presented in Table D-8. Figure D-10 presents a comparison of the unscaled ten pairs of spectra with the recommended MCE spectrum in log-log scale.

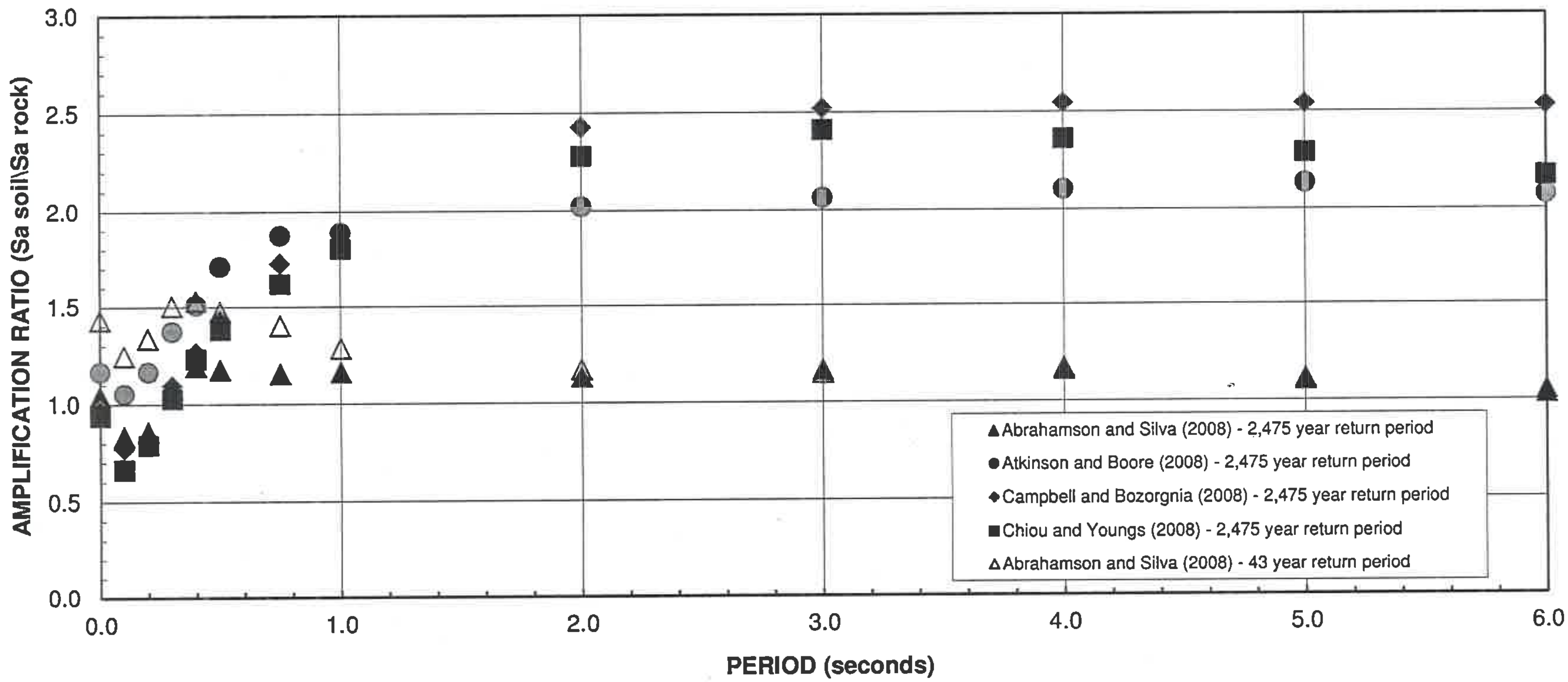
TABLE D-8
Earthquake Time Histories Used for Scaling to DE and MCE Spectra

Earthquake	Magnitude	Time History
Loma Prieta	6.9	Los Gatos PC
Duzce	7.1	Duzce
Denali	7.9	Pump Station #10
Kocaeli	7.5	Duzce
Kocaeli	7.5	Yarimca
Kocaeli Northridge	6.7	Sylmar
Manjil	7.4	Abbar
Imperial Valley	6.5	Holtville
Chi Chi	7.6	TCU065
Chi Chi	7.6	CHY101

The scaling factors developed following the procedure outlined by Kircher & Associates (1996) for the MCE target spectrum are presented in Table D-9.

TABLE D-9
Scaling Factors for MCE Time Histories

Time History	MCE
Los Gatos PC	0.89
Duzce, Duzce	1.02
Pump Station #10	0.68
Duzce, Kocaeli	1.40
Yarimca	1.16
Sylmar	0.67
Abbar	1.56
Holtville	2.07
TCU065	0.70
CHY101	0.78



Damping Ratio = 5%

Notes:

1. Results include average directivity

350 MISSION STREET San Francisco, California		
AMPLIFICATION RATIOS PSHA (Sa soil/ Sa rock)		
Date 04/25/12	Project No. 730466502	Figure D-1

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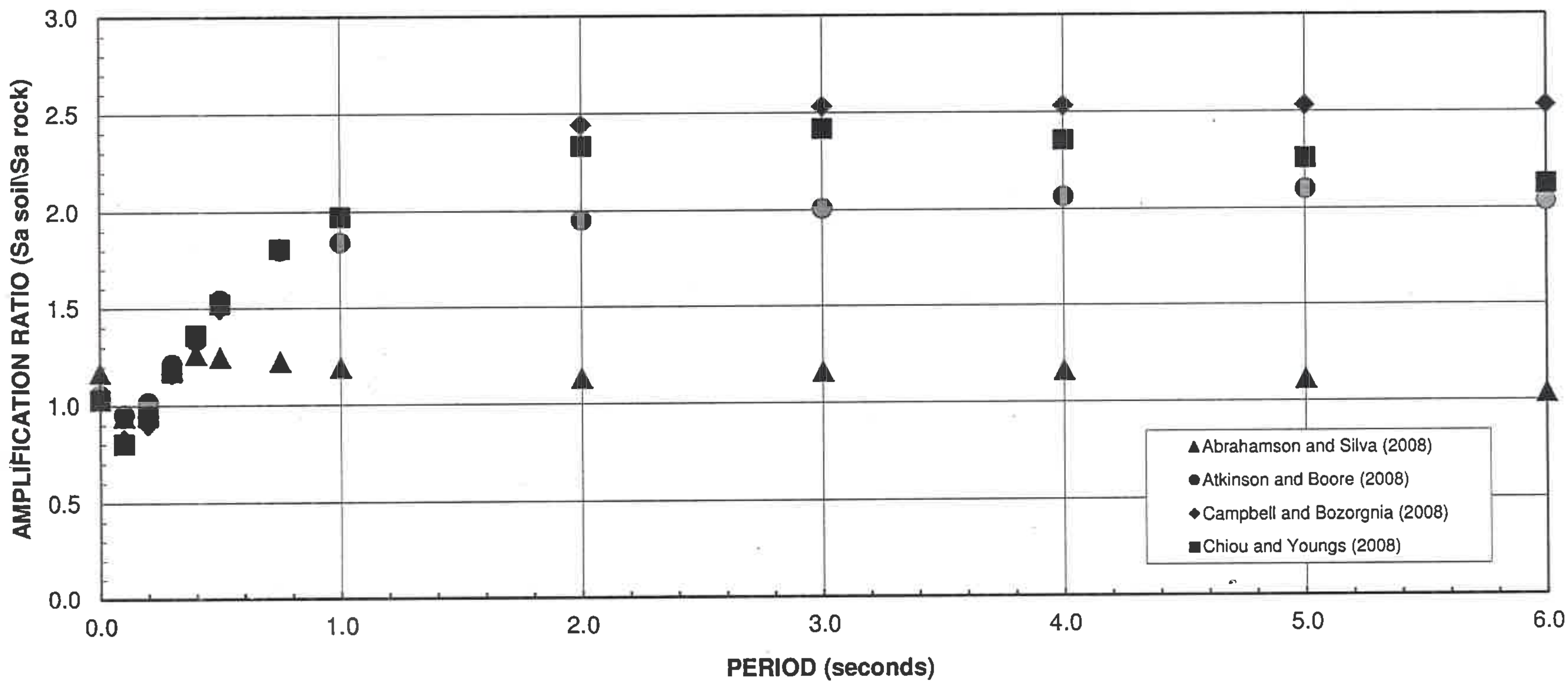

Damping Ratio = 5%

Notes:

1. Results include average directivity

350 MISSION STREET San Francisco, California		
AMPLIFICATION RATIOS DETERMINISTIC **(Sa soil/ Sa rock)**		
Date 04/25/12	Project No. 730466502	Figure D-2
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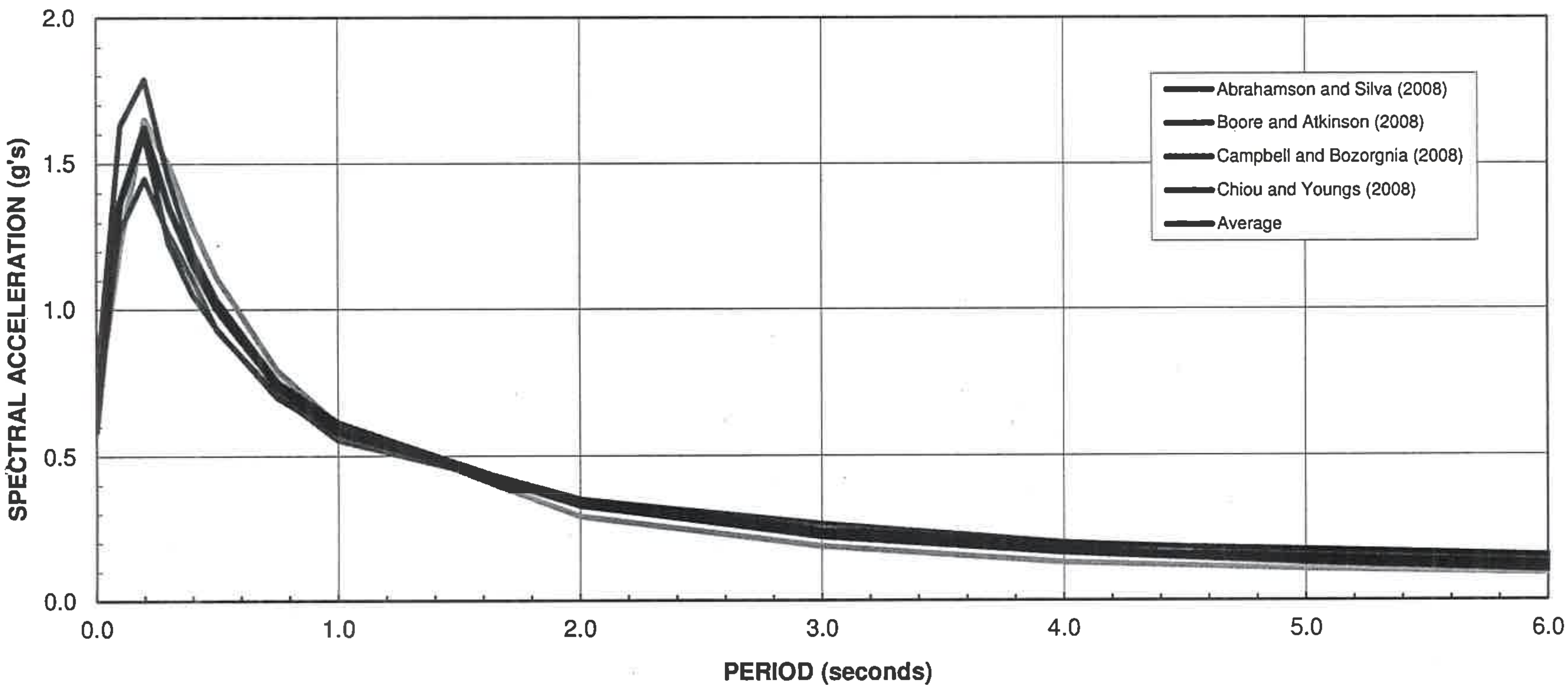

Damping Ratio = 5%

Notes:

1. Estimated Vs_{30} 760 meters per second
2. Avg. Directivity
3. $Z_{1.0\ Km/sec}$ 12 m from Rincon Hill Recording Station (NGA Flatfile)

350 MISSION STREET
San Francisco, California

RESULTS OF PSHA - 2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS

Date 04/25/12	Project No. 730466502	Figure D-3

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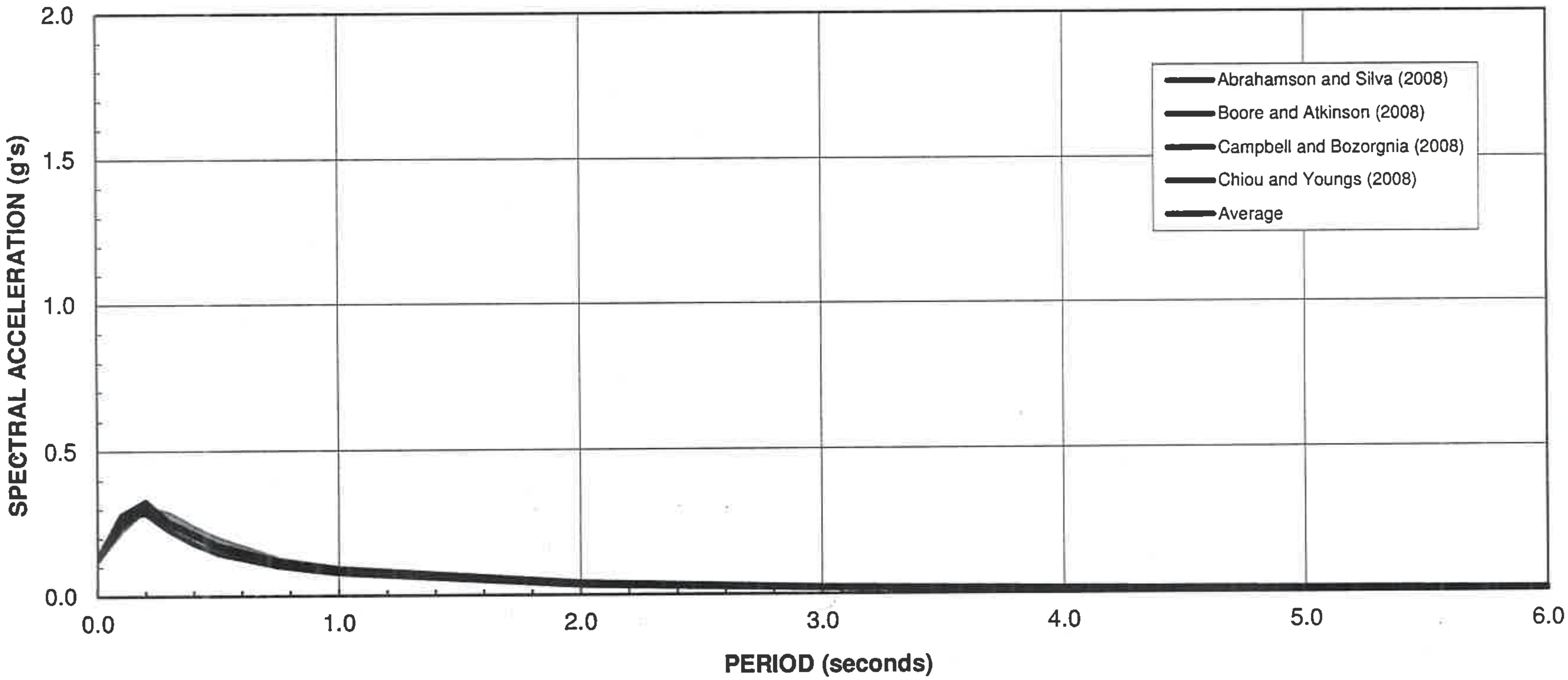

Damping Ratio = 5%

Notes:

1. Estimated Vs_{30} 760 meters per second
2. Avg. Directivity
3. $Z_{1.0\ Km/sec}$ 12 m from Rincon Hill Recording Station (NGA Flatfile)

350 MISSION STREET San Francisco, California
RESULTS OF PSHA - 50 PERCENT PROBABILITY OF EXCEEDANCE IN 30 YEARS
Date 04/25/12 | Project No. 730466502 | Figure D-4
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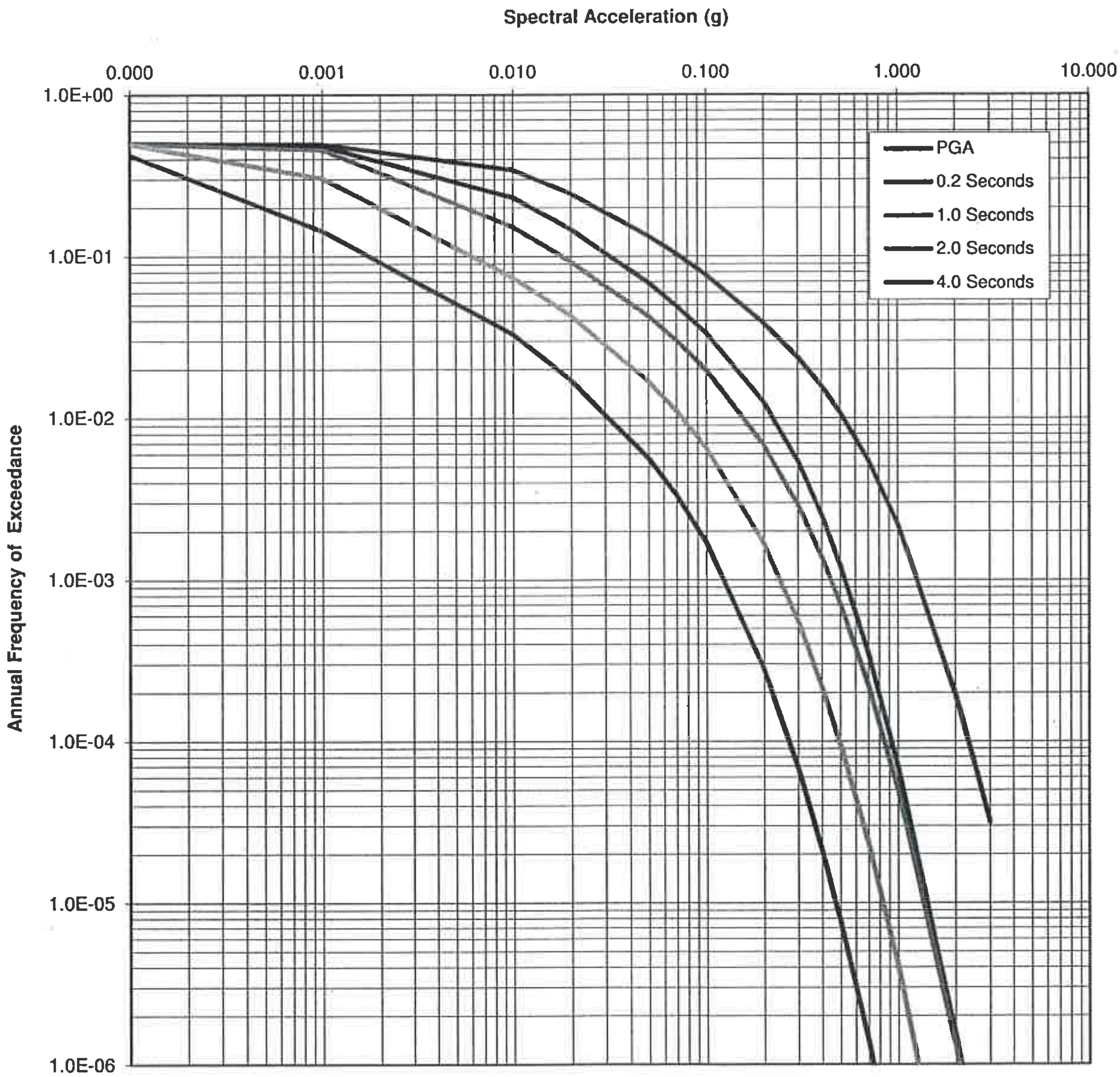
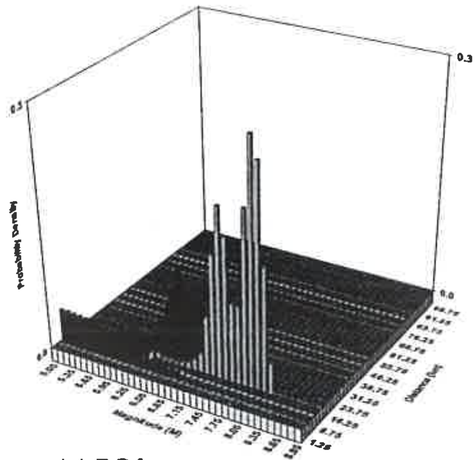
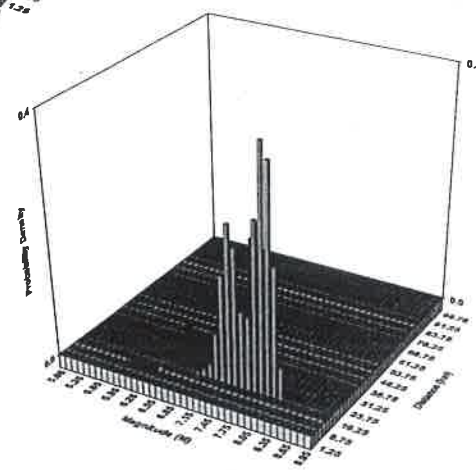


Note: Seismic hazard curves are for the mean of the four rock attenuation relationships [Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008) and Chiou and Youngs (2008)]

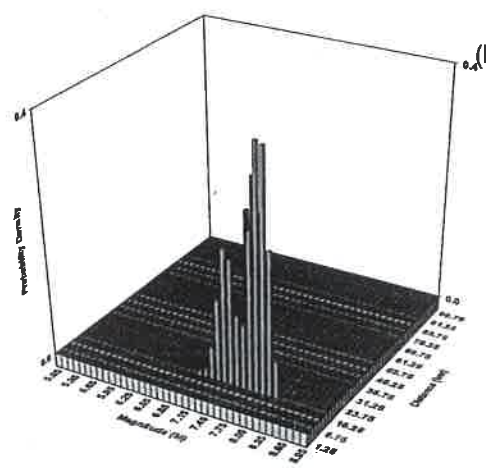
350 MISSION STREET San Francisco, California		
SEISMIC HAZARD FOR PGA, 0.2, 1.0, 2.0 AND 4.0 SECONDS		
Date 04/25/12	Project No. 730466502	Figure D-5
Treadwell&Rollo A LANGAN COMPANY		



(a) PGA



(b) S_a, T = 1.0 seconds



(c) S_a, T = 4.0 seconds

350 MISSION STREET
San Francisco, California

2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS DEAGGREGATION

Date 04/25/12	Project No. 730466502	Figure D-6

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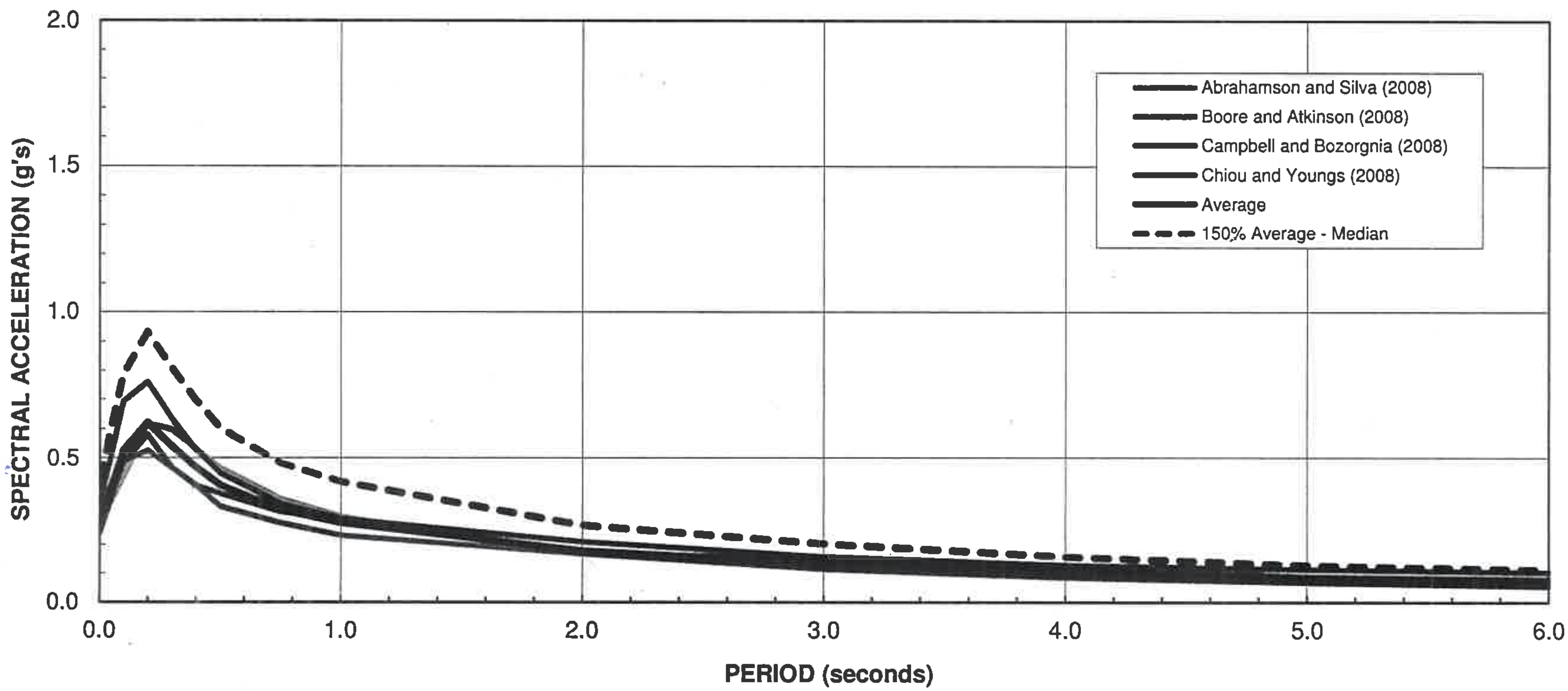

Damping Ratio = 5%

Notes:

1. Estimated Vs_{30} 760 meters per second
2. Deterministic Results correspond to a Moment Magnitude 8.0 occuring on the San Andreas Fault about 13.3 km from the site and are for the geometric mean
3. $Z_{1.0\ Km/sec}$ 12 m from Rincon Hill Recording Station (NGA Flatfile)
4. Average Directivity

350 MISSION STREET
San Francisco, California

MEDIAN DETERMINISTIC RESULTS - ROCK

Date 04/25/12	Project No. 730466502	Figure D-7

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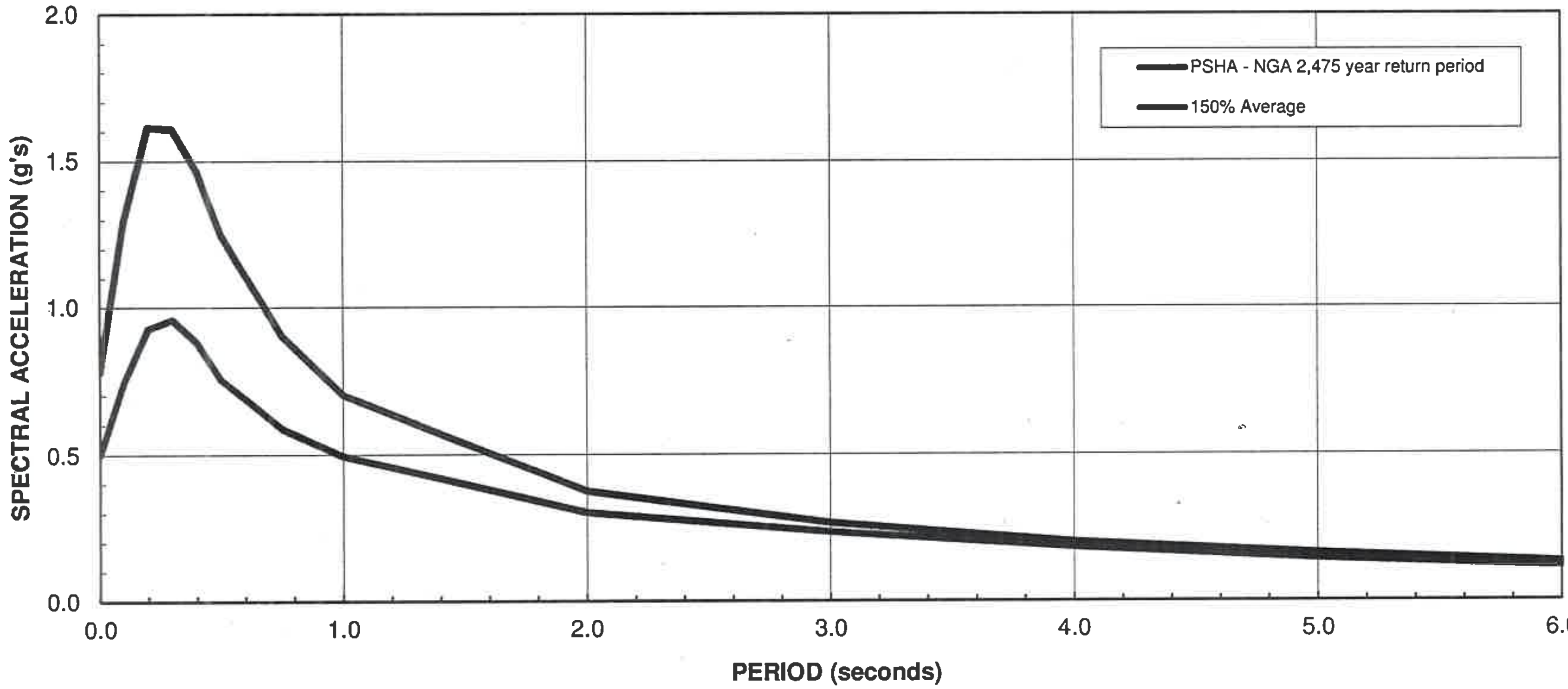

Damping Ratio = 5%

Notes:

1. Spectra developed using Abrahamson and Silva (2008) amplification Factors
2. Deterministic Results correspond to a Moment Magnitude 8.0 occuring on the San Andreas Fault about 13.3 km from the site and are for the geometric mean
3. Include Average Directivity

350 MISSION STREET San Francisco, California		
PSHA AND DETERMINISTIC SPECTRA - SOIL		
Date 04/25/12	Project No. 730466502	Figure D-8

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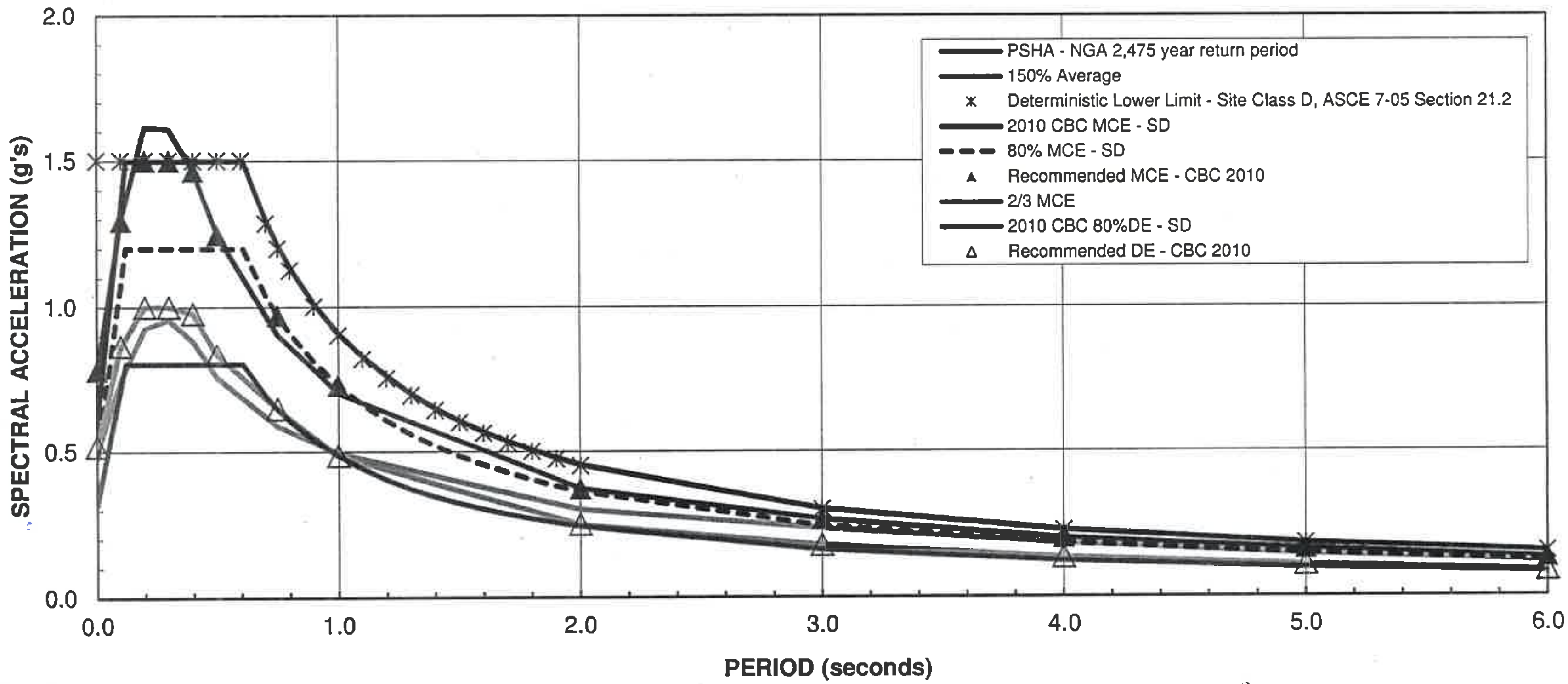

Notes:

Damping Ratio = 5%

1. Estimated Vs_{30} 243 meters per second
2. Deterministic Results correspond to a Moment Magnitude 8.0 occuring on the San Andreas Fault about 13.3 km from the site
3. PSHA and Determinsitic spectra include average directivity
4. Soil Spectra developed from rock spectra using Abrahamson and Silva (2008) ampification factors

350 MISSION STREET
San Francisco, California

COMPARISON OF PROBABILISTIC AND DETERMINISTIC RESULTS - SOIL

Date 04/25/12	Project No. 730466502	Figure D-9

Treadwell&Rollo
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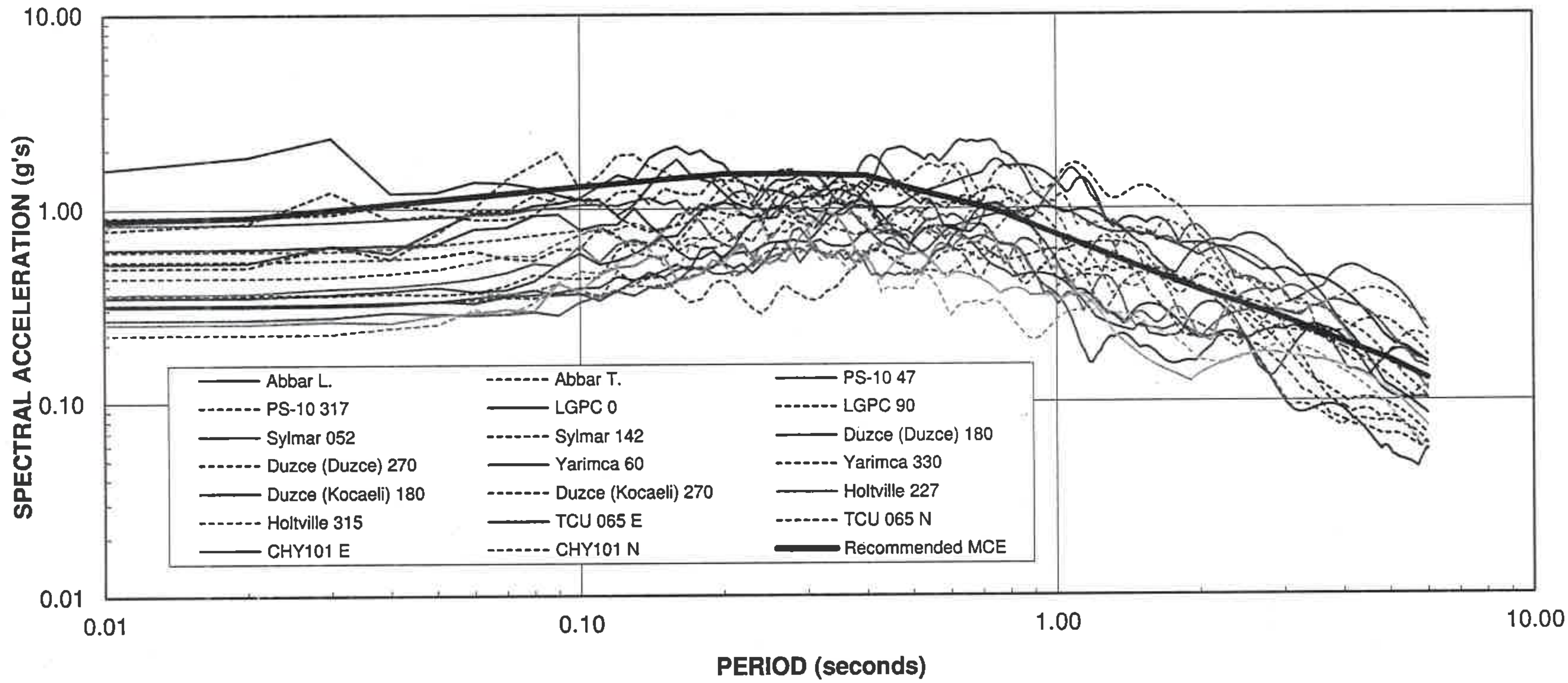
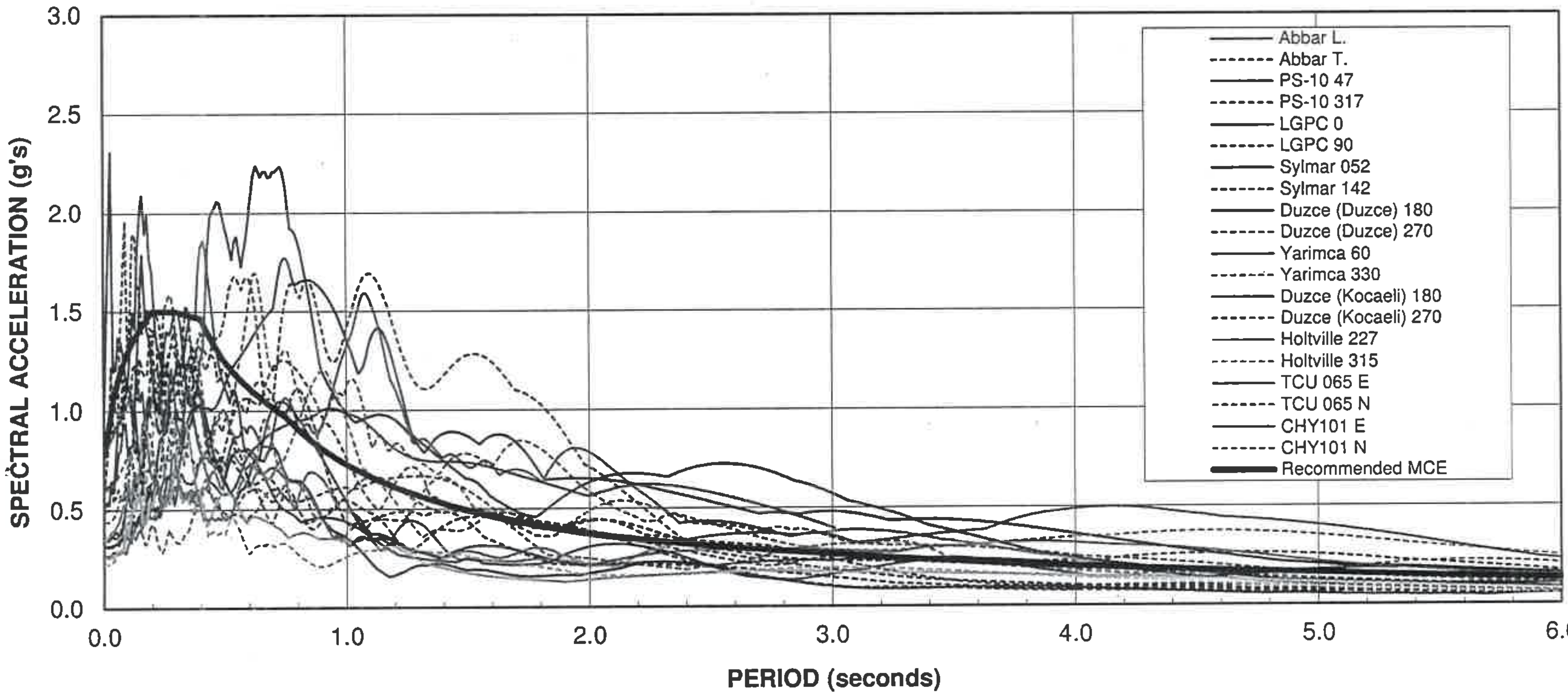


Damping Ratio = 5%

350 MISSION STREET San Francisco, California		
UNSCALED SPECTRA		
Date 04/25/12	Project No. 730466502	Figure D-10
Treadwell&Rollo *A LANGAN COMPANY*		



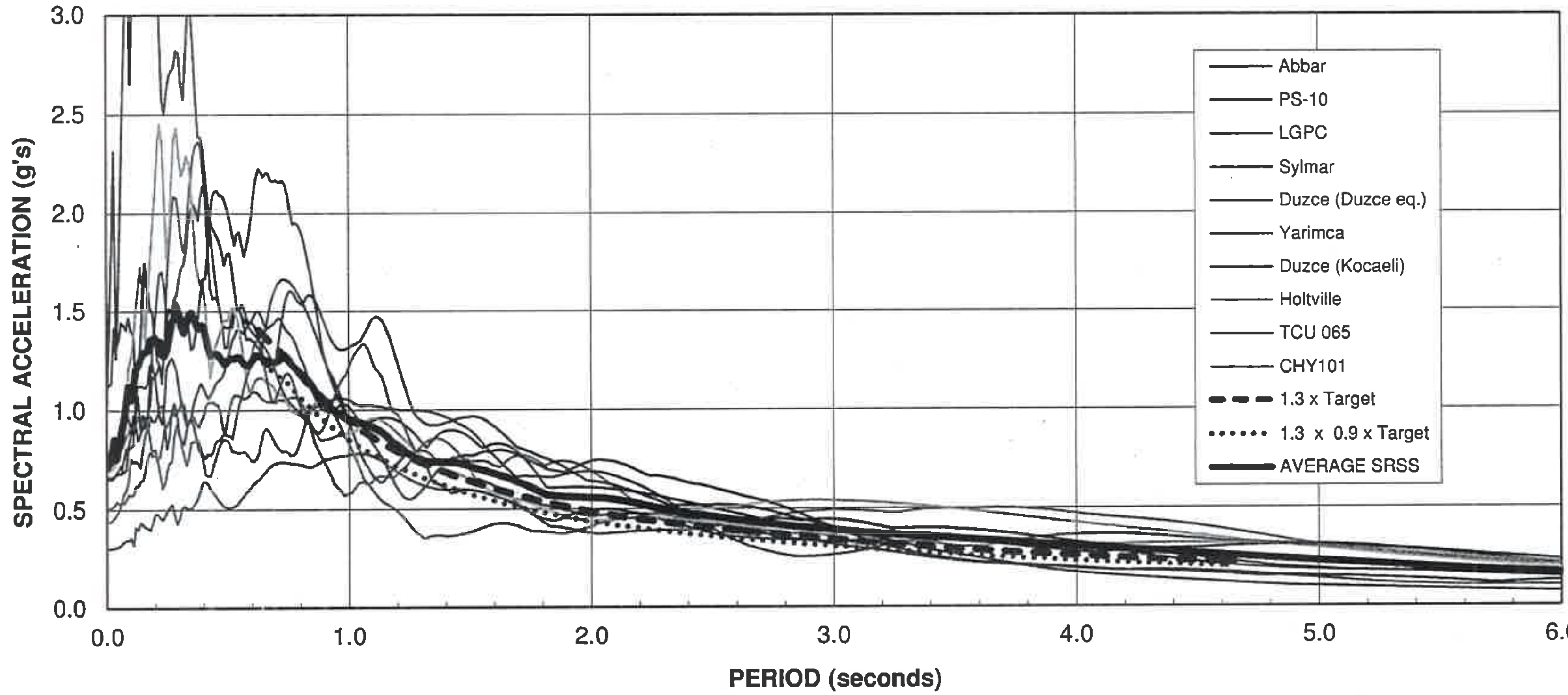
Damping Ratio = 5%

350 MISSION STREET
San Francisco, California

UNSCALED SPECTRA

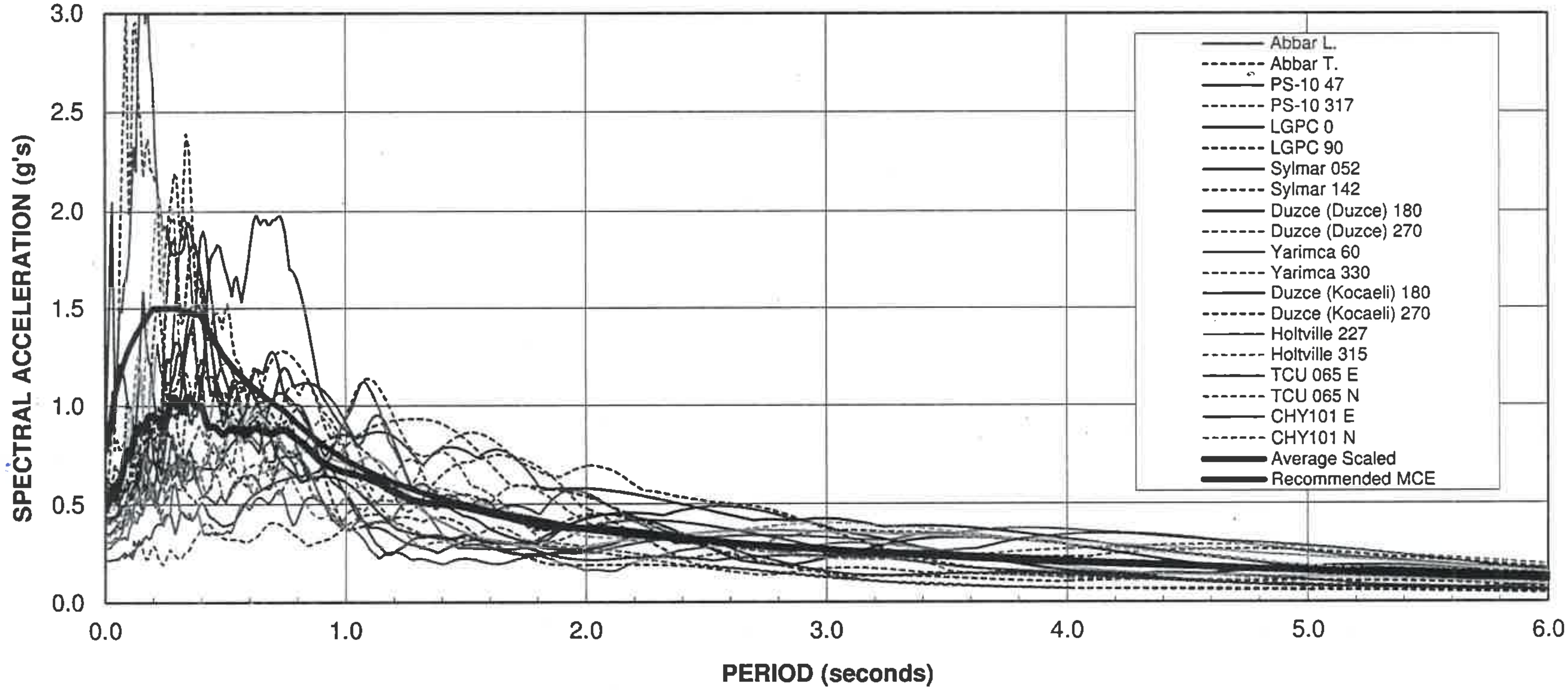
Date 04/25/12	Project No. 730466502	Figure D-11

Treadwell&Rollo
A LANGAN COMPANY



Damping Ratio = 5%

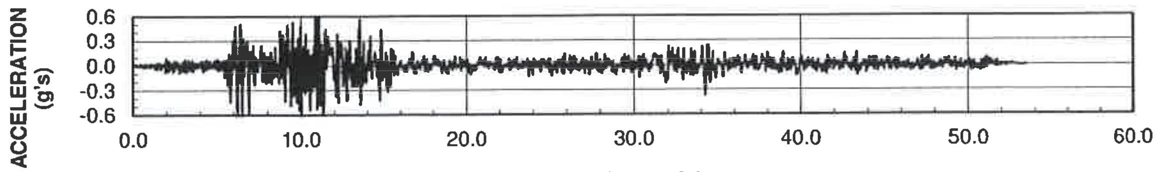
350 MISSION STREET San Francisco, California		
SCALED MCE SRSS SPECTRA		
Date 04/25/12	Project No. 730466502	Figure D-12
Treadwell&Rollo *A LANGAN COMPANY*		

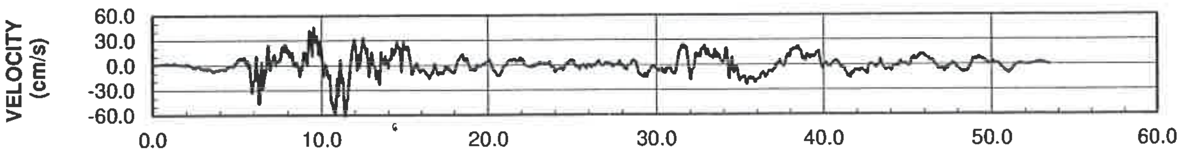


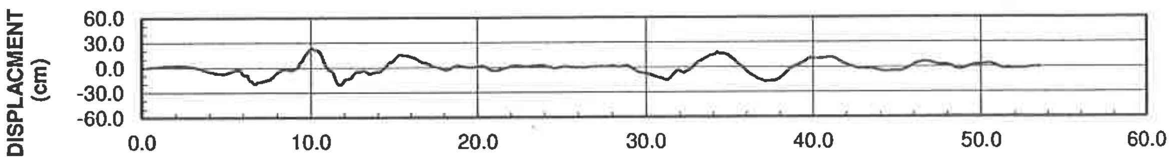
Damping Ratio = 5%

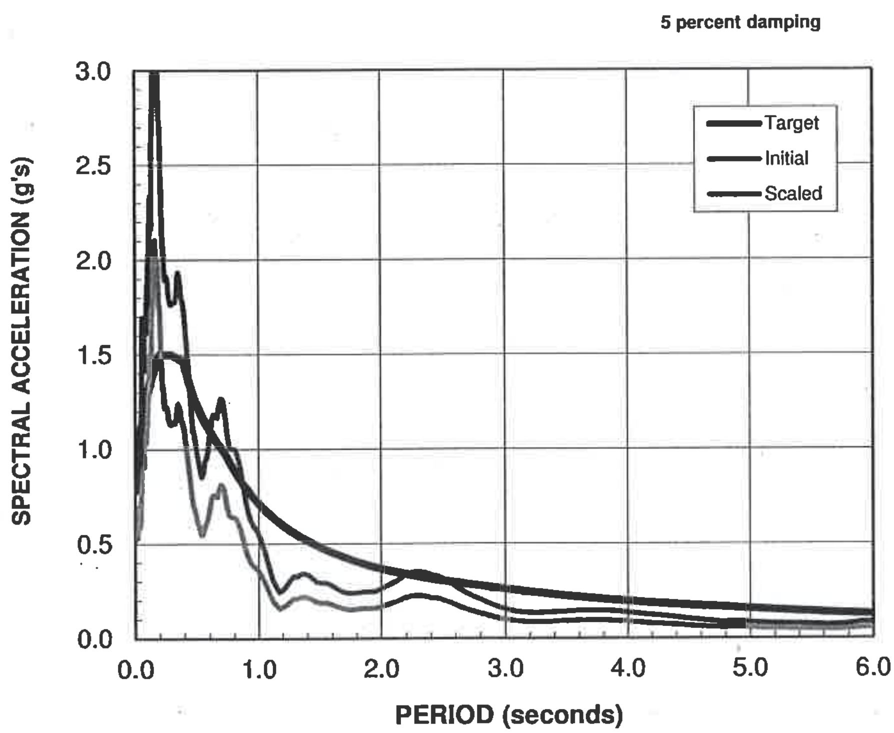
350 MISSION STREET San Francisco, California		
SCALED MCE SPECTRA		
Date 04/25/12	Project No. 730466502	Figure D-13

Treadwell&Rollo
A LANGAN COMPANY

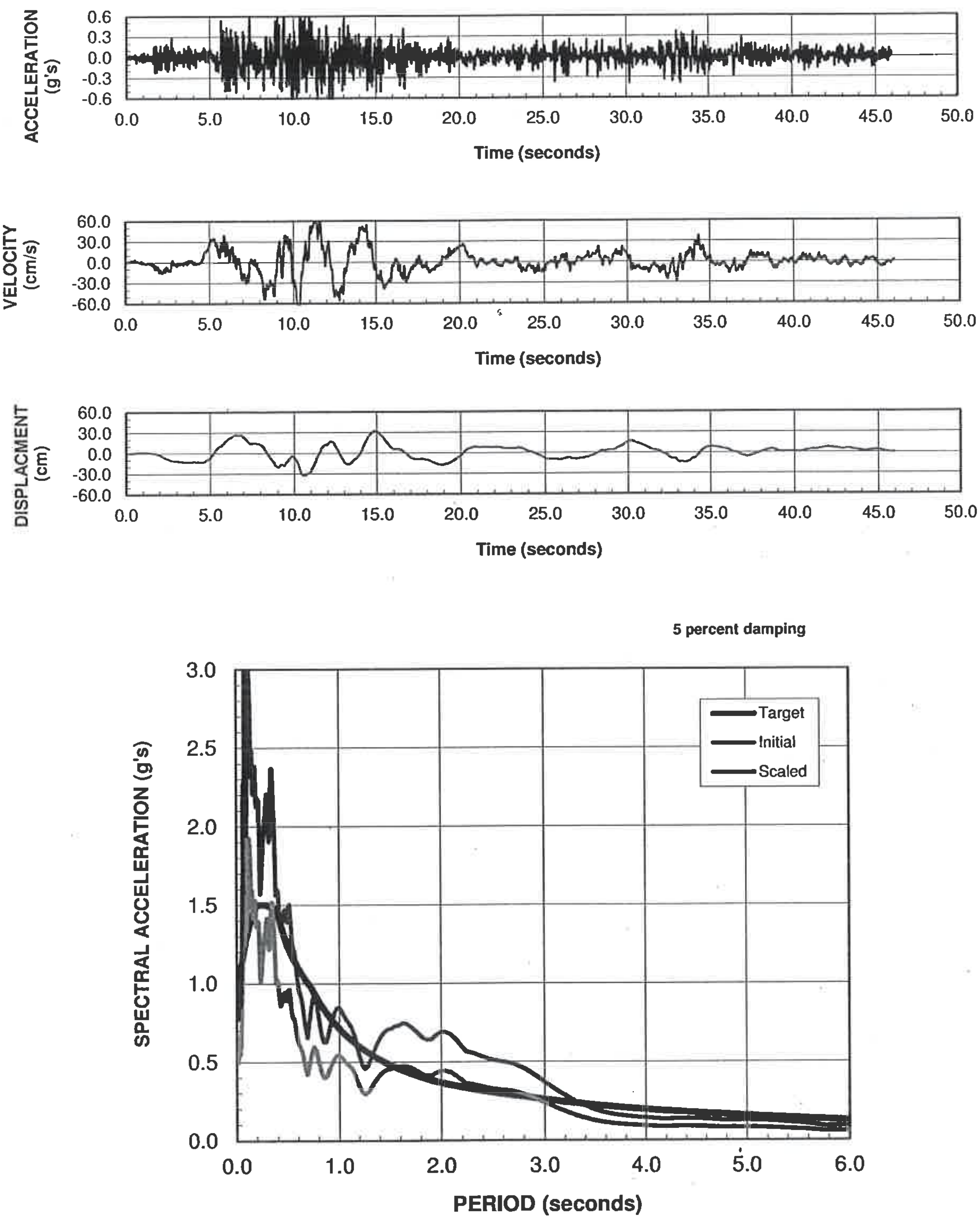








350 MISSION STREET San Francisco, California	MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1990 MANJIL EARTHQUAKE ABBAR L.
Treadwell&Rollo A LANGAN COMPANY	Date 04/25/12 | Project No. 730466502 | Figure D-14



350 MISSION STREET
San Francisco, California

Treadwell&Rollo
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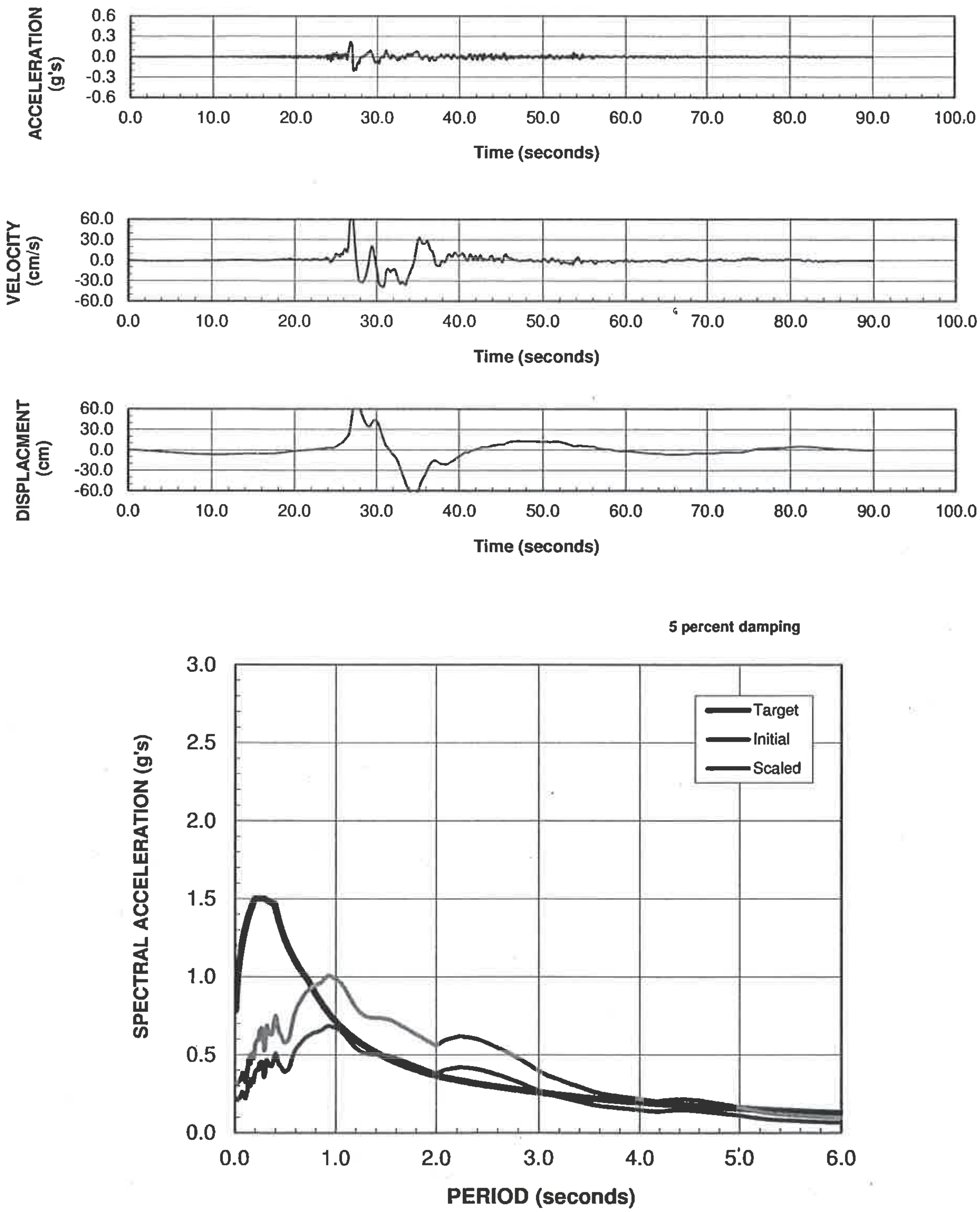
MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1990 MANJIL EARTHQUAKE ABBAR T.

Date 04/25/12	Project No. 730466502	Figure D-15

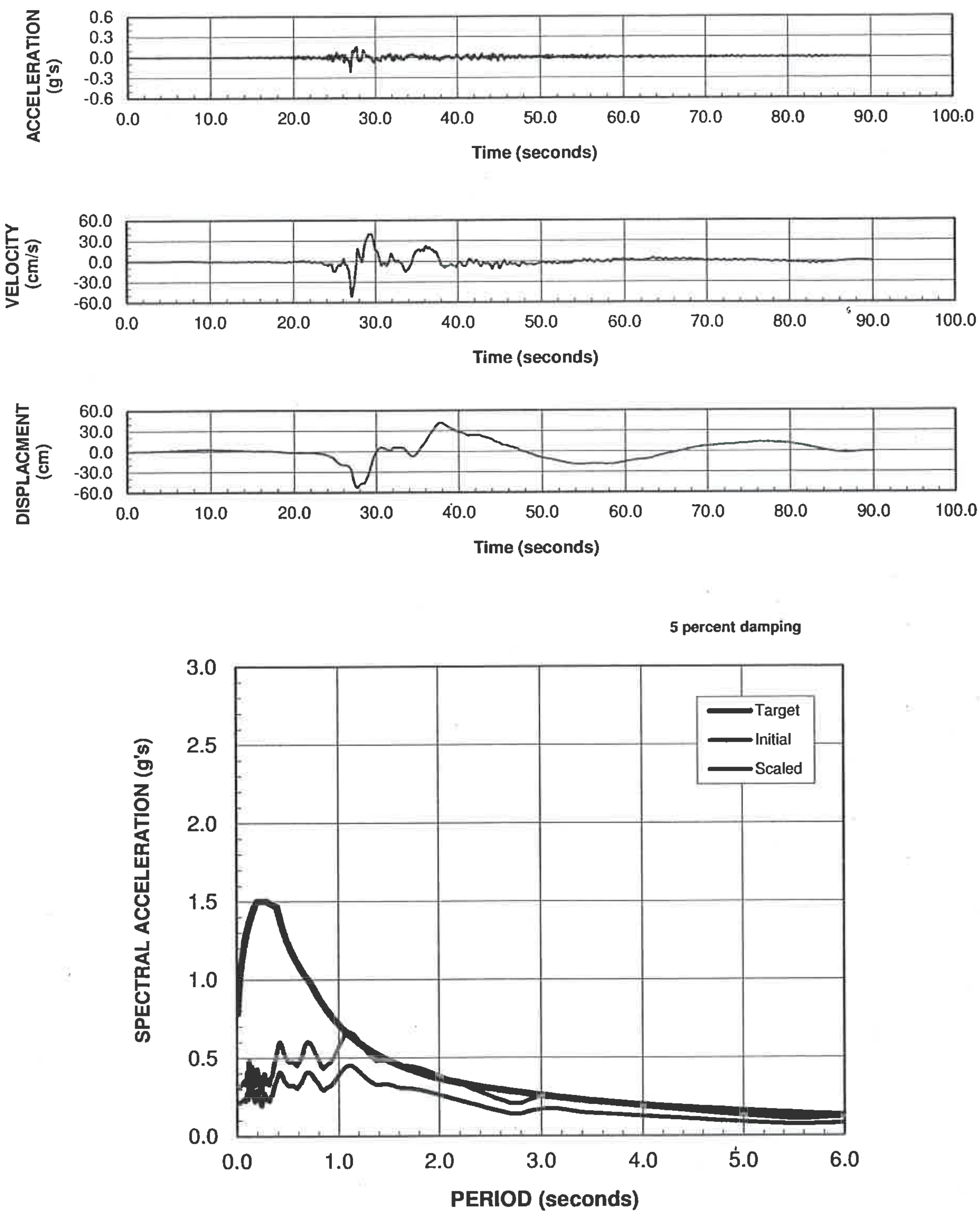


350 MISSION STREET
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 2002 DENALI EARTHQUAKE PS-10 47 Deg.

Date 04/25/12	Project No. 730466502	Figure D-16

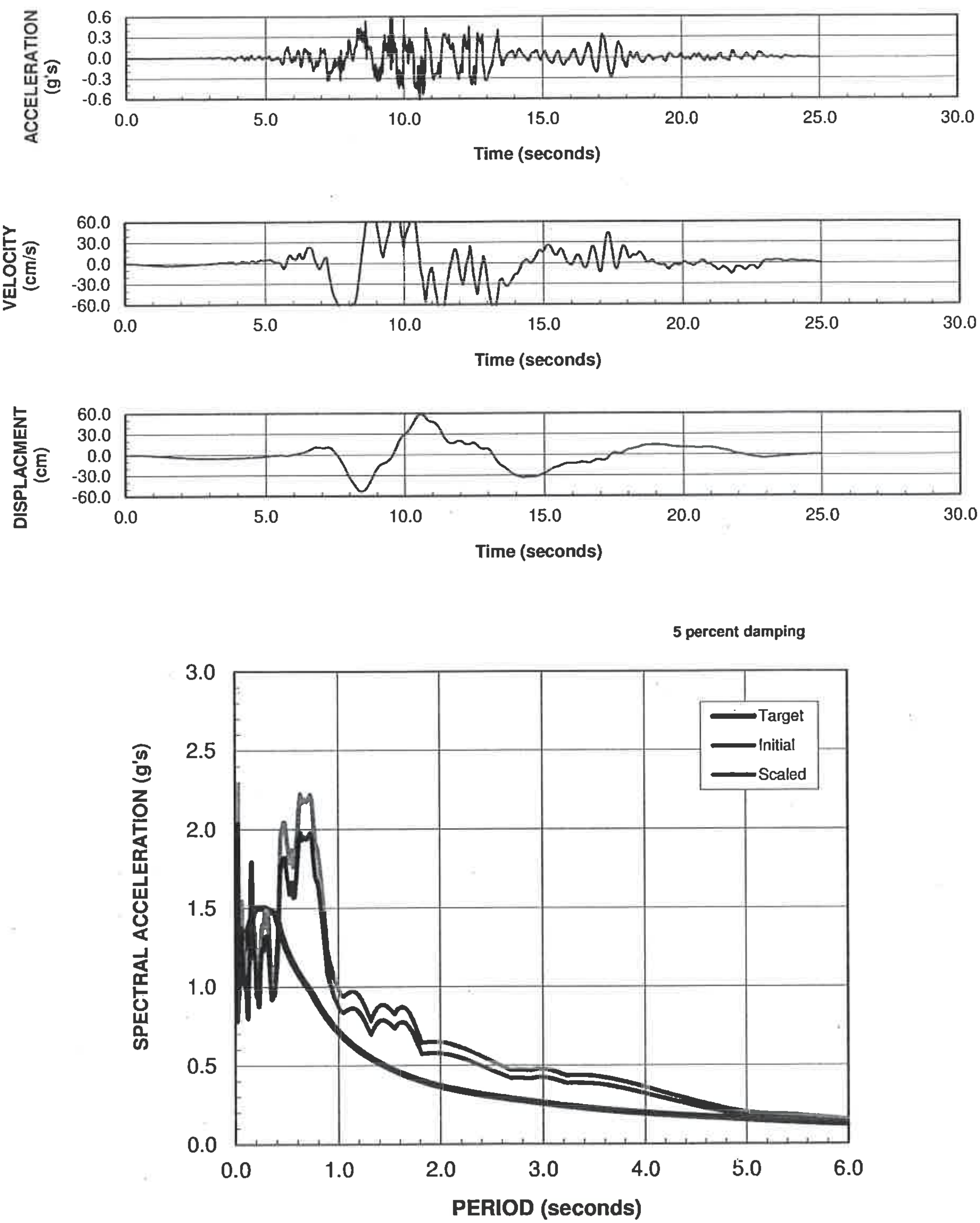


350 MISSION STREET
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 2002 DENALI EARTHQUAKE PS-10 317 Deg.

Date 04/25/12	Project No. 730466502	Figure D-17

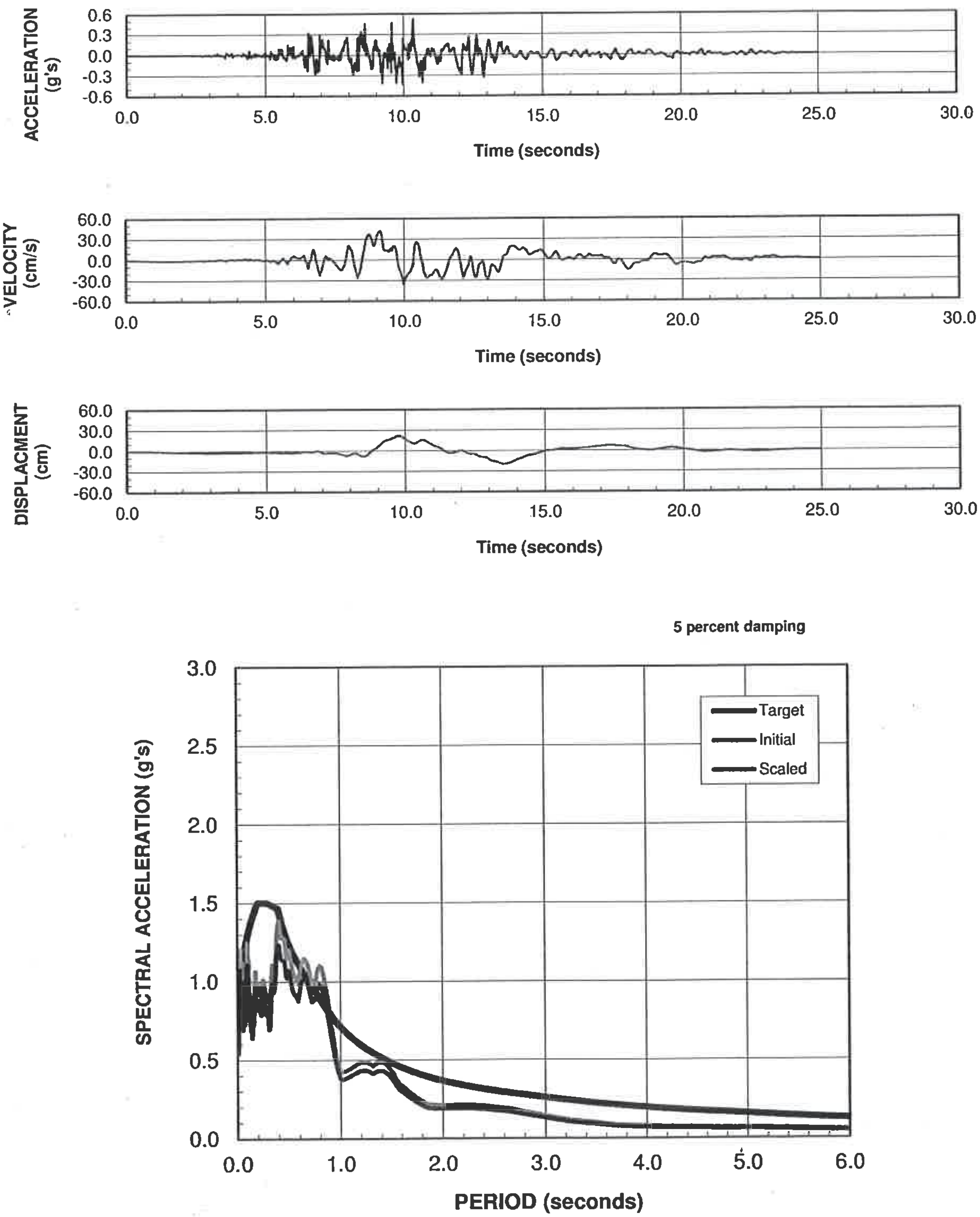


350 MISSION STREET
San Francisco, California

Treadwell&Rollo
A LANGAN COMPANY

MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1989 LOMA PRIETA EARTHQUAKE LGPC 0 Deg.

Date 04/25/12	Project No. 730466502	Figure D-18

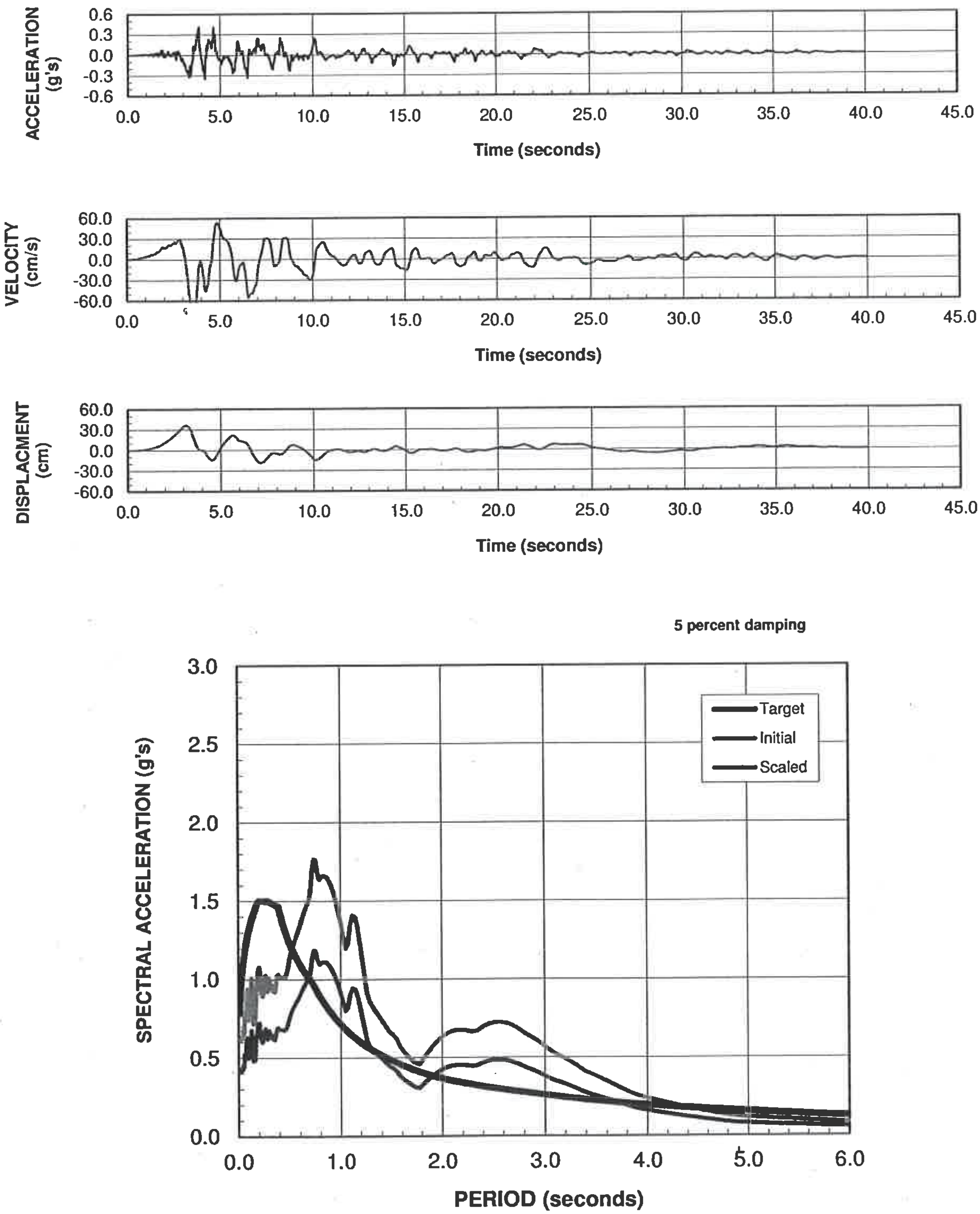


350 MISSION STREET
San Francisco, California

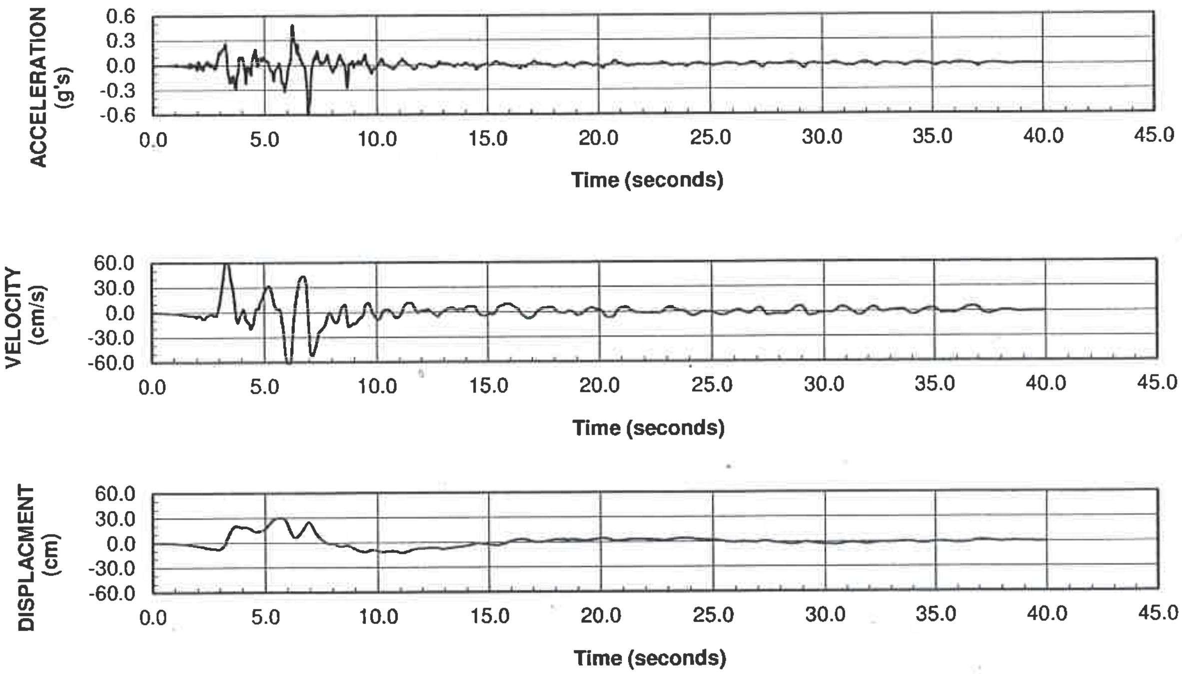
Treadwell&Rollo
A LANGAN COMPANY

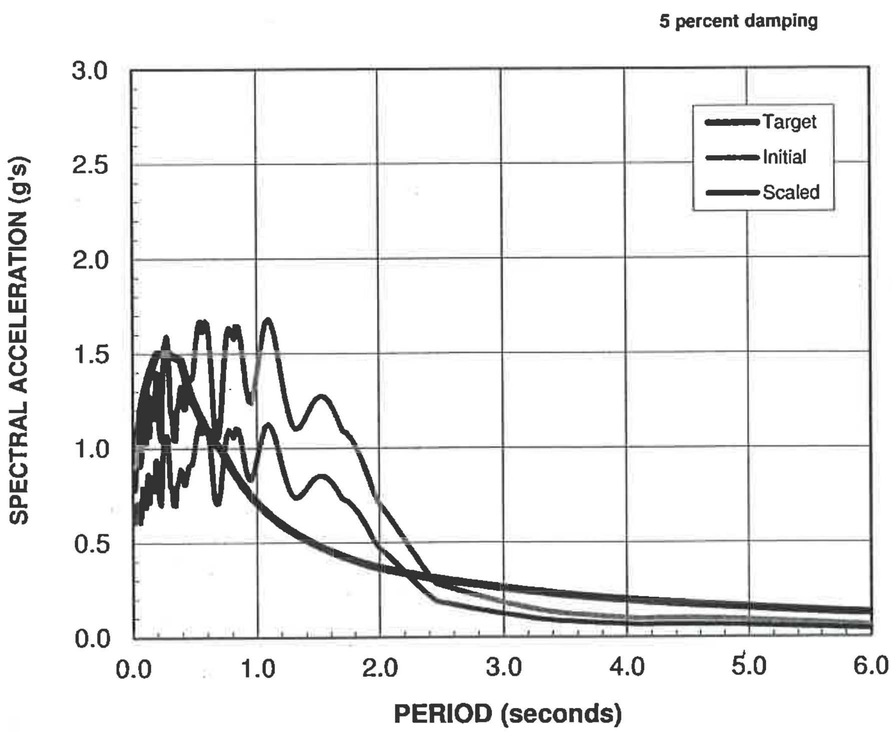
MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1989 LOMA PRIETA EARTHQUAKE LGPC 90 Deg.

Date 04/25/12	Project No. 730466502	Figure D-19

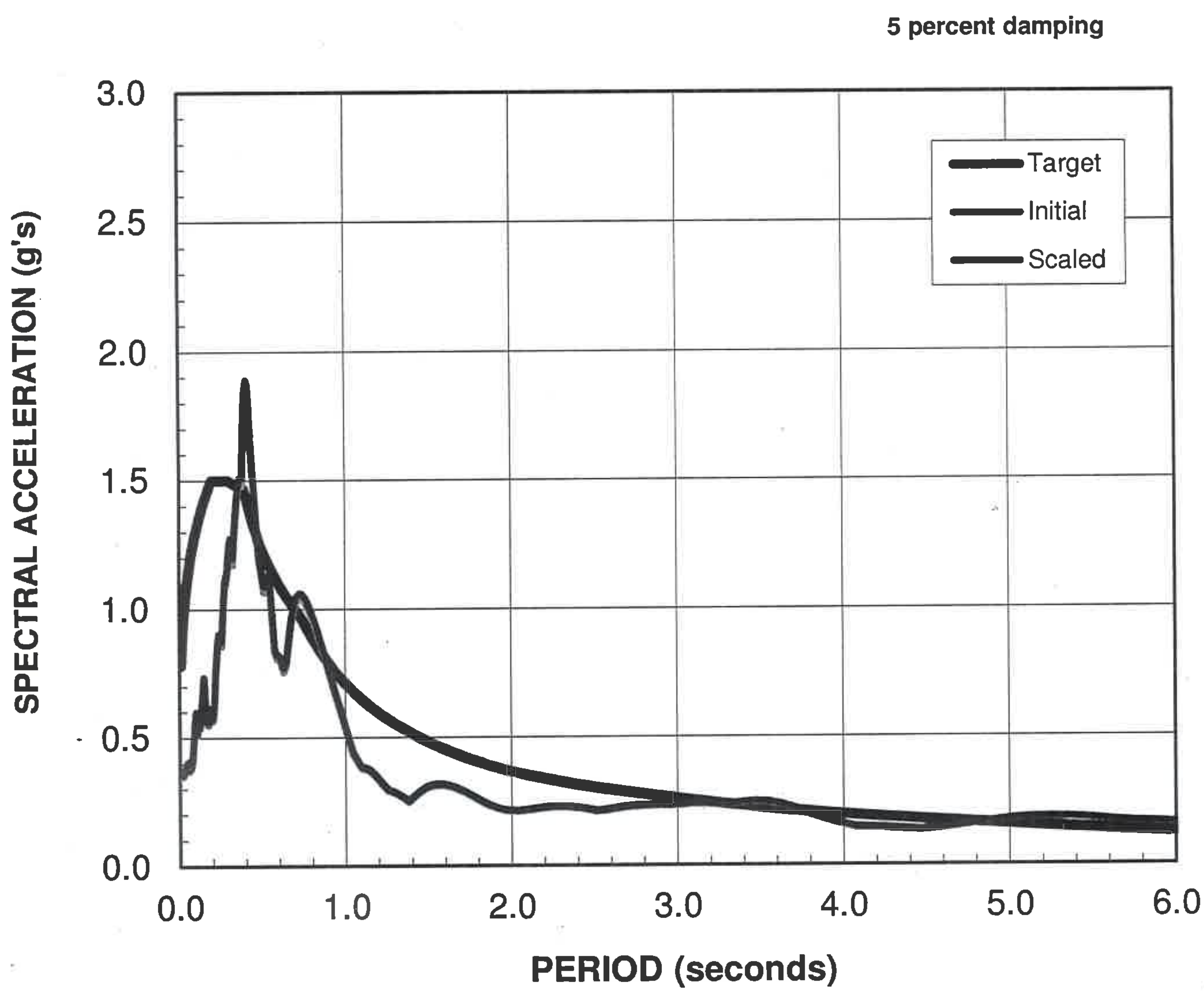


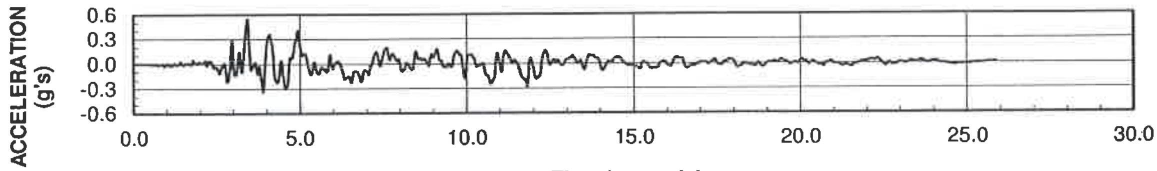
350 MISSION STREET San Francisco, California	MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1994 NORTHRIDGE EARTHQUAKE SYLMAR 52 Deg.		
Treadwell&Rollo A LANGAN COMPANY	Date 04/25/12	Project No. 730466502	Figure D-20

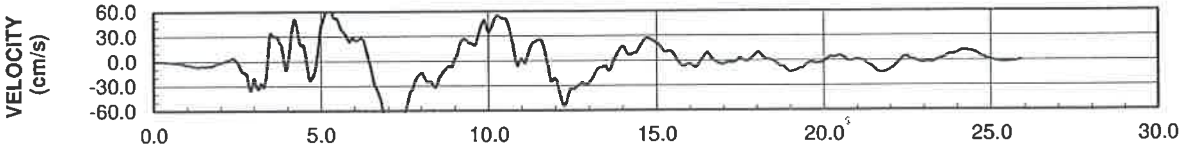


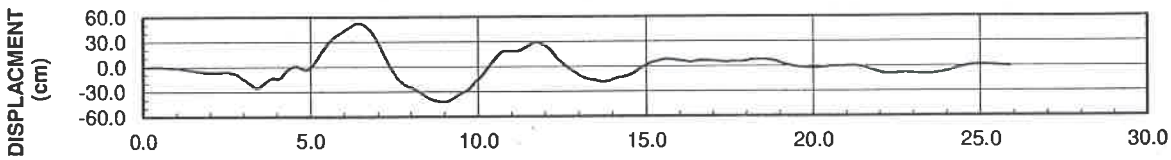


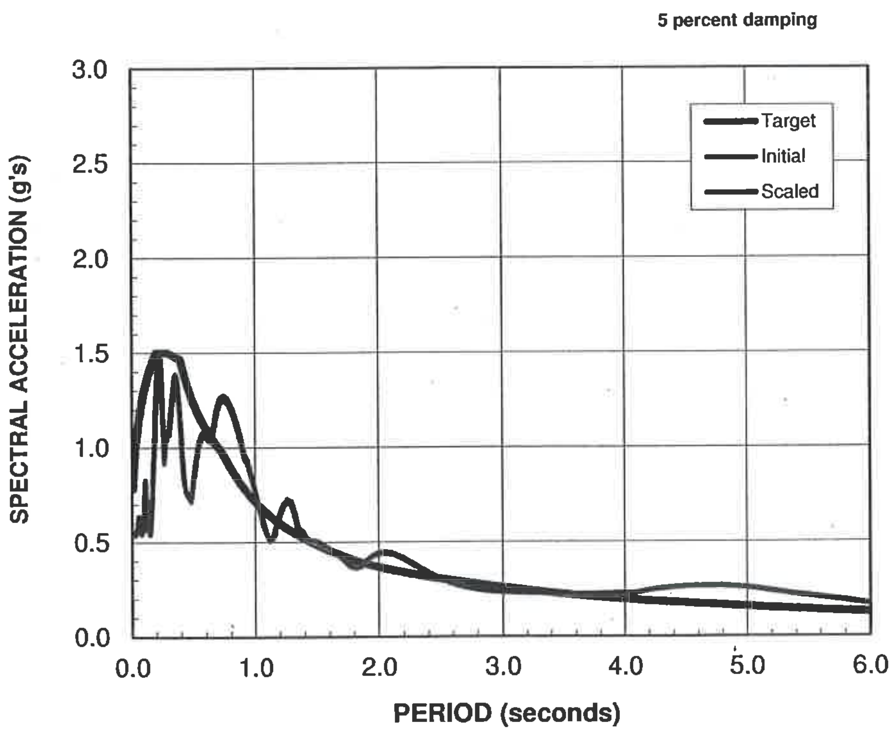
350 MISSION STREET San Francisco, California	**MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1994 NORTHRIDGE EARTHQUAKE SYLMAR 142 Deg.**
Treadwell&Rollo *A LANGAN COMPANY*	Date 04/25/12 | Project No. 730466502 | Figure D-21


5 percent damping
Target
Initial
Scaled
3.0
2.5
2.0
1.5
1.0
0.5
0.0
SPECTRAL ACCELERATION (g's)
0.0
1.0
2.0
3.0
4.0
5.0
6.0
PERIOD (seconds)

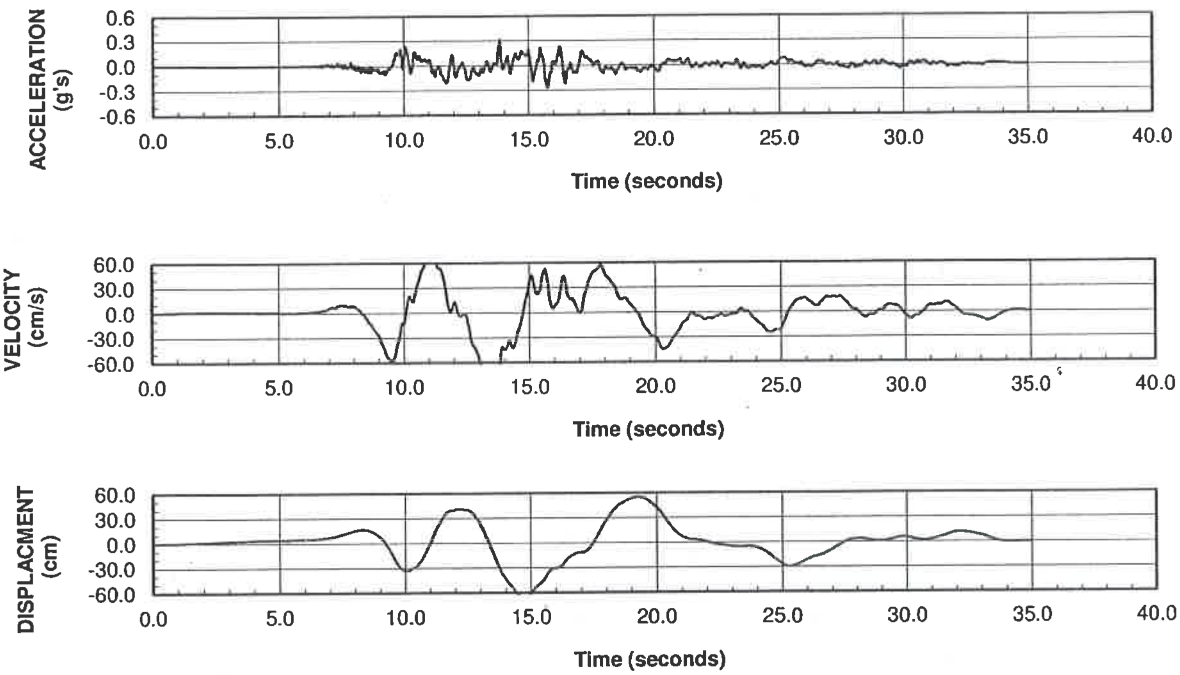


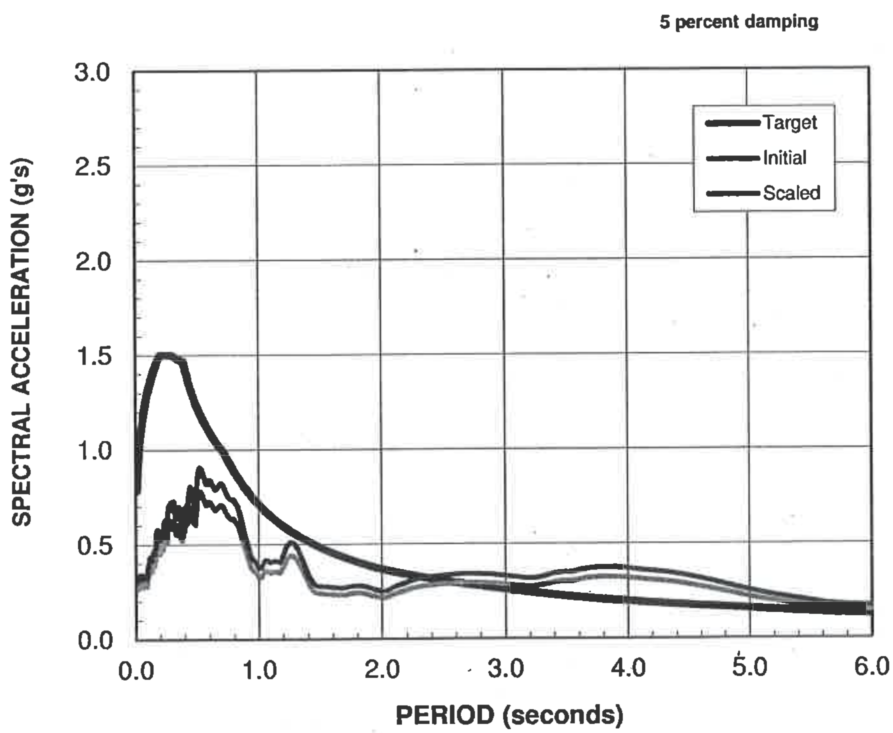




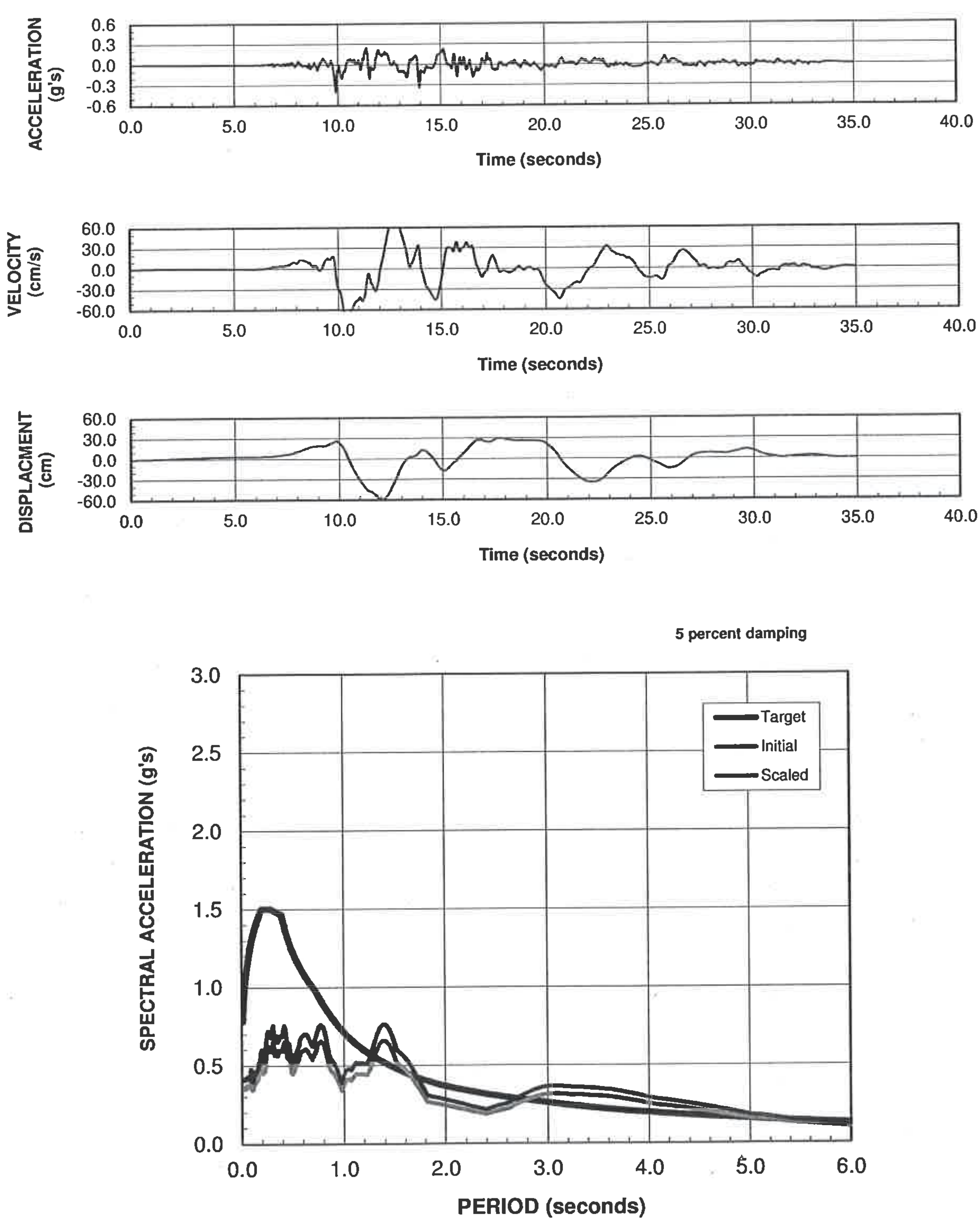


350 MISSION STREET San Francisco, California	**MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1999 DUZCE EARTHQUAKE DUZCE 180 Deg.**
Treadwell&Rollo *A LANGAN COMPANY*	Date 04/25/12 | Project No. 730466502 | Figure D-23

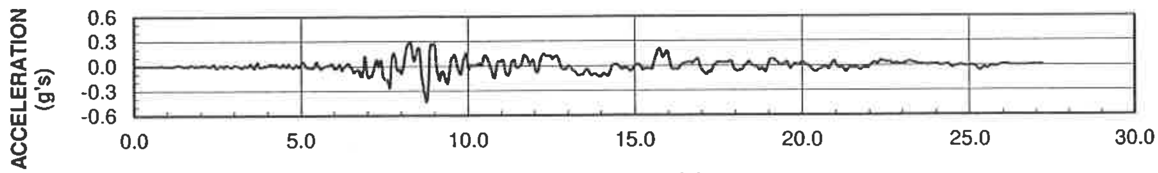


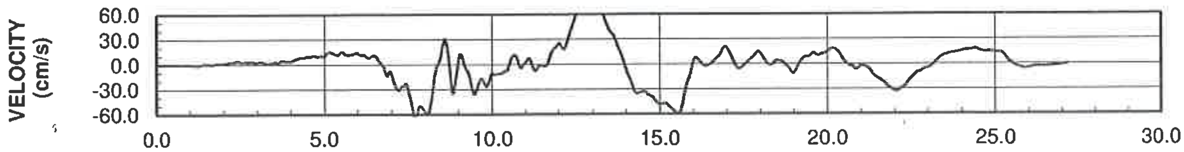


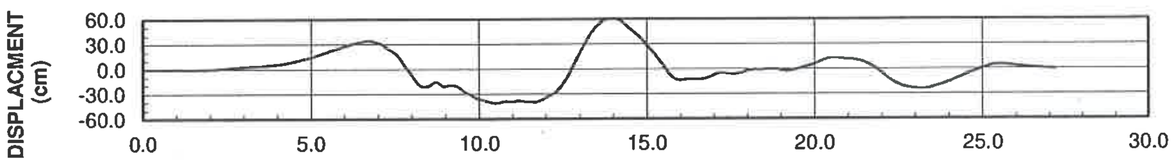
350 MISSION STREET San Francisco, California	**MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1999 KOCAELI EARTHQUAKE YARIMCA 60 Deg.**
Treadwell&Rollo *A LANGAN COMPANY*	Date 04/25/12 | Project No. 730466502 | Figure D-24

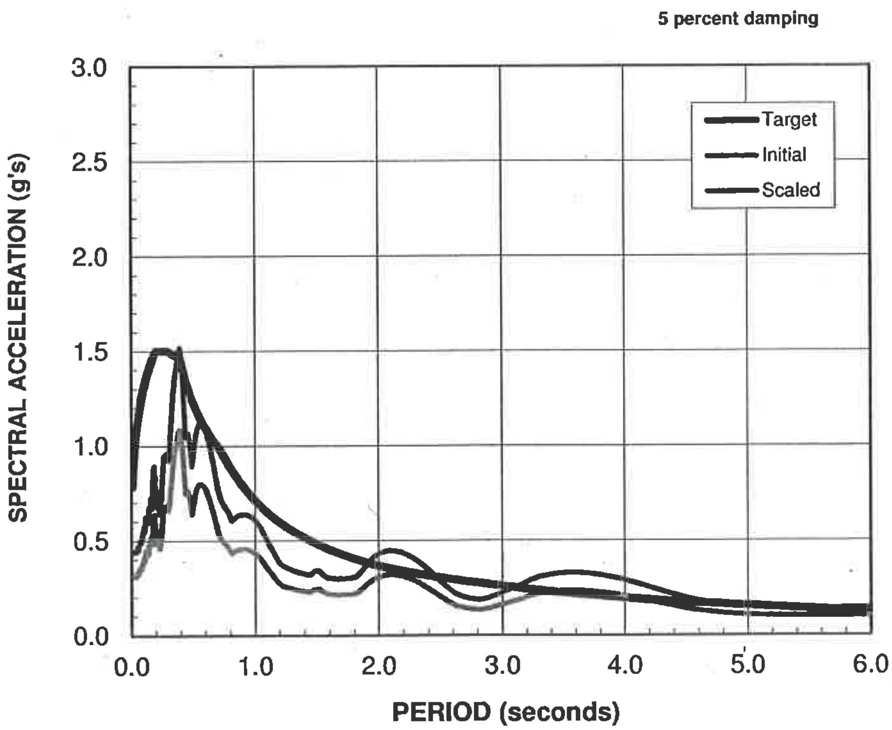


350 MISSION STREET San Francisco, California	**MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1999 KOCAELI EARTHQUAKE YARIMCA 330 Deg.**
Treadwell&Rollo *A LANGAN COMPANY*	Date 04/25/12 | Project No. 730466502 | Figure D-25

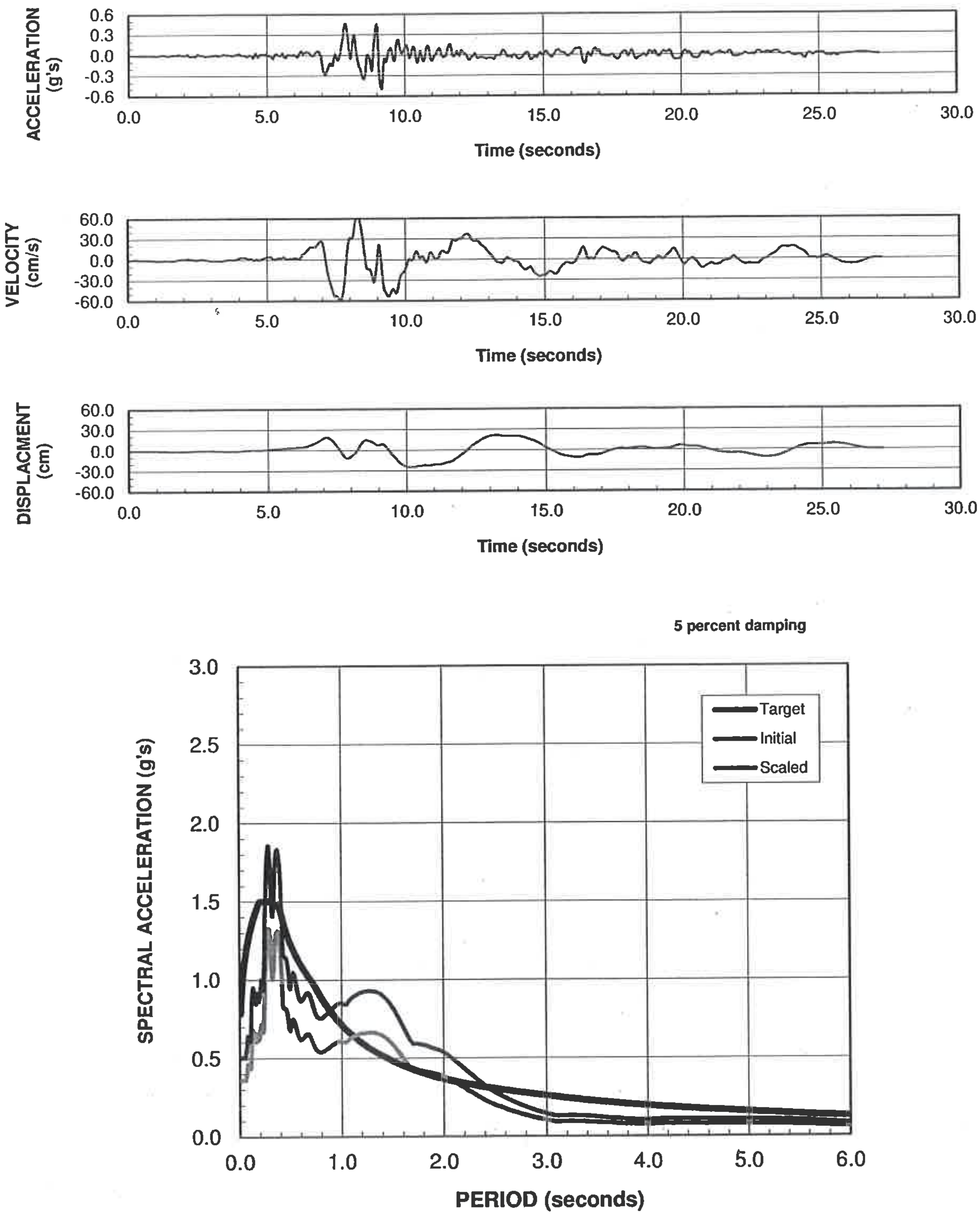








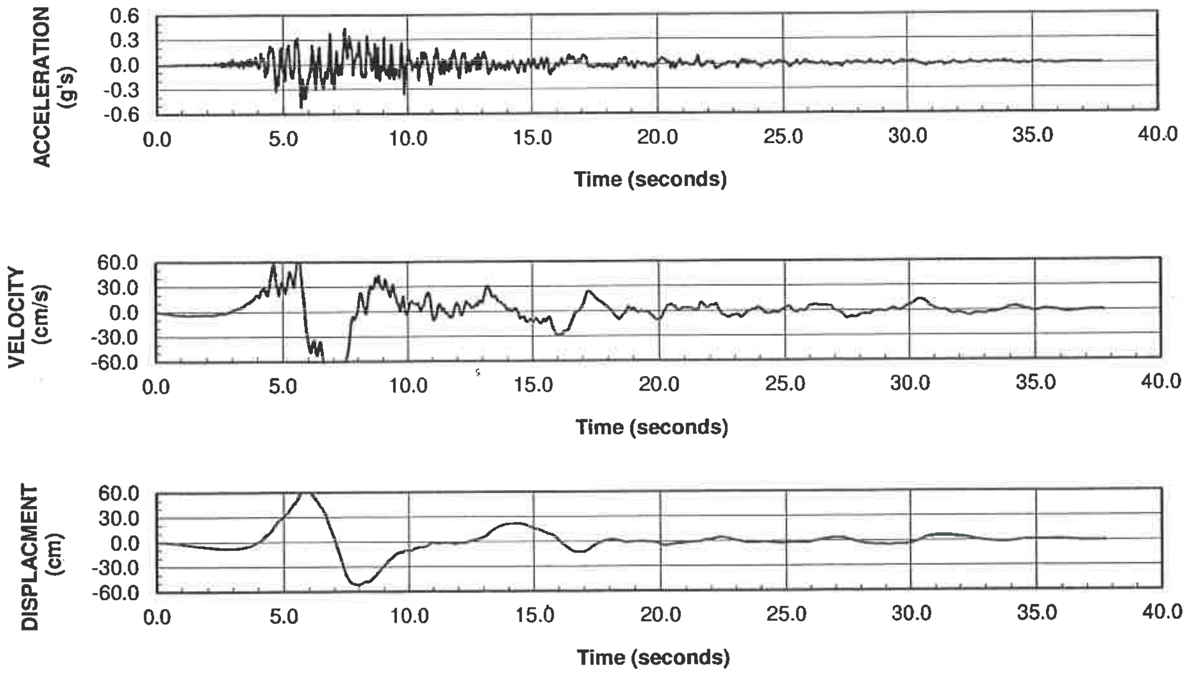
350 MISSION STREET San Francisco, California	**MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1999 KOCAELI EARTHQUAKE DUZCE 180 Deg.**
Treadwell&Rollo *A LANGAN COMPANY*	Date 04/25/12 | Project No. 730466502 | Figure D-26

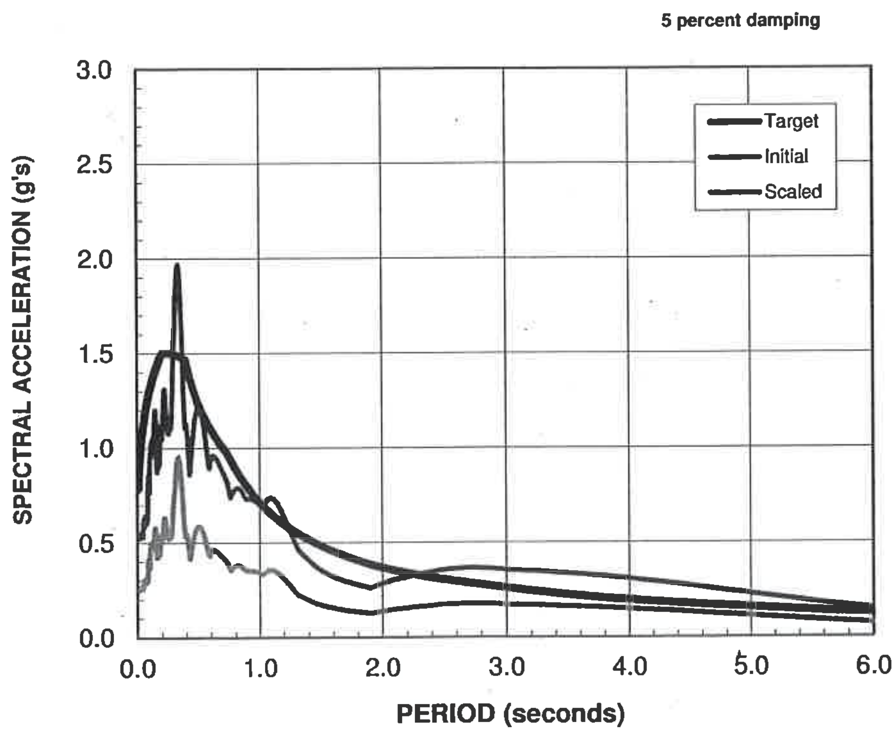


350 MISSION STREET San Francisco, California	MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1999 KOCAELI EARTHQUAKE DUZCE 270 Deg.

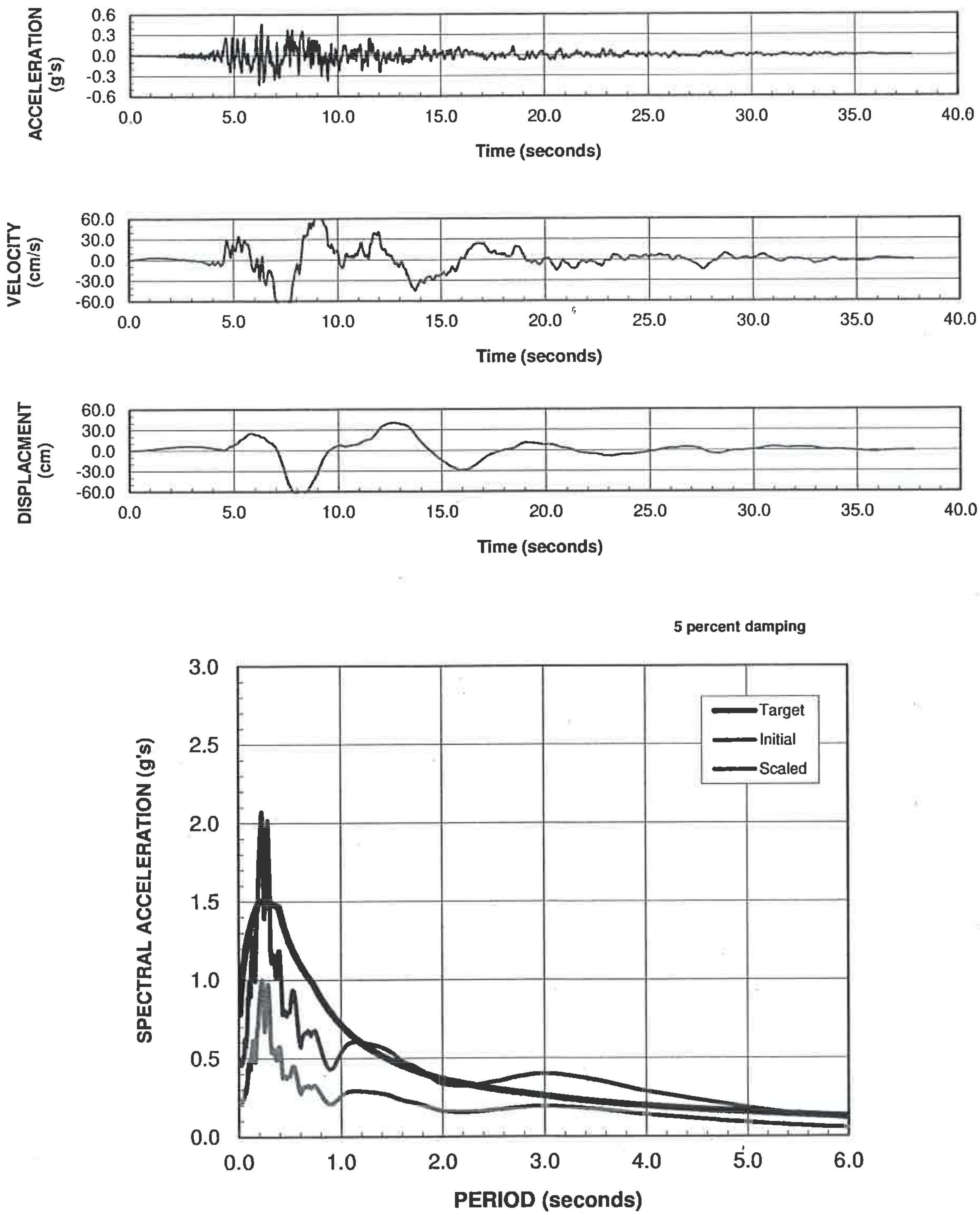
Treadwell&Rollo
A LANGAN COMPANY

Date 04/25/12	Project No. 730466502	Figure D-27

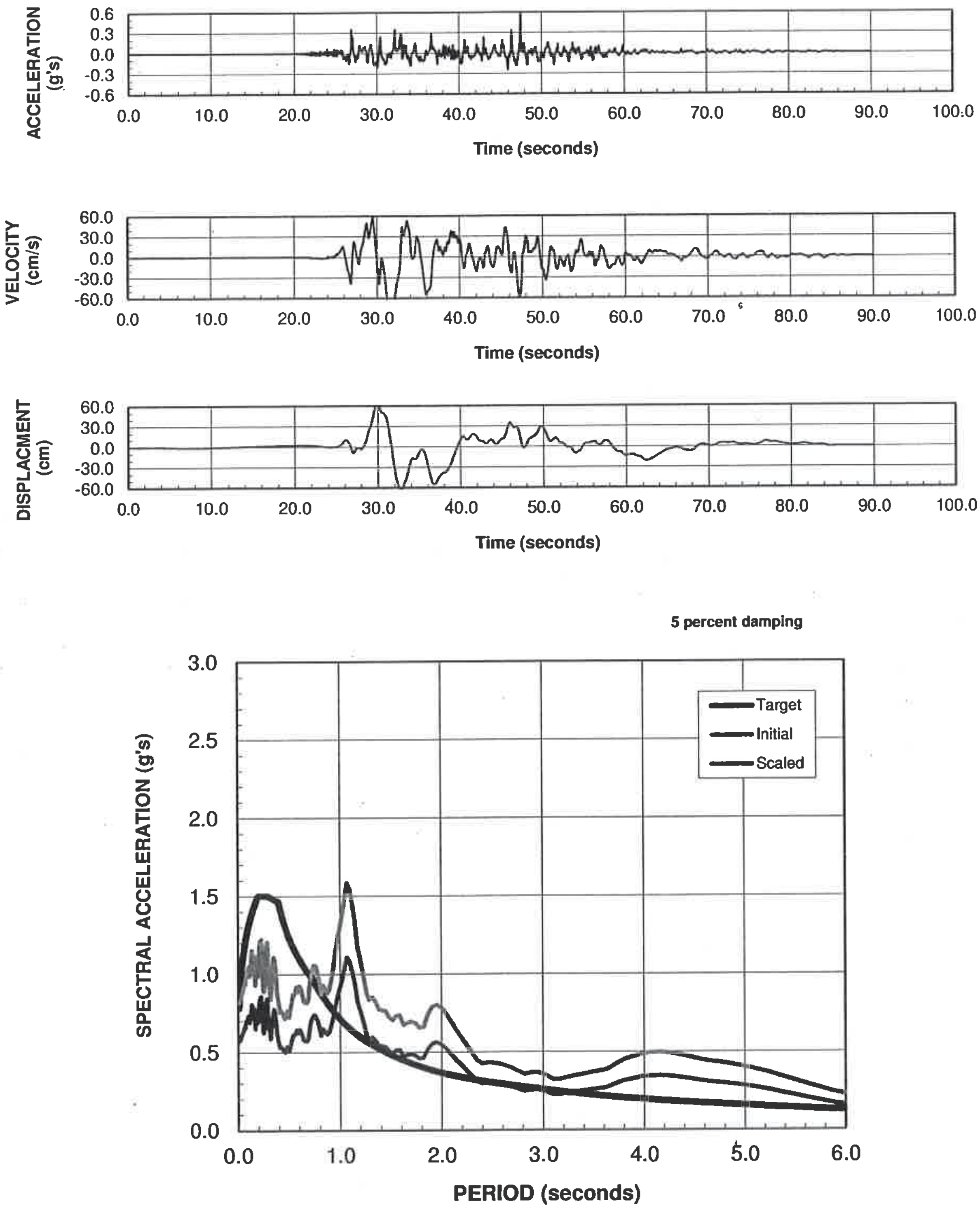




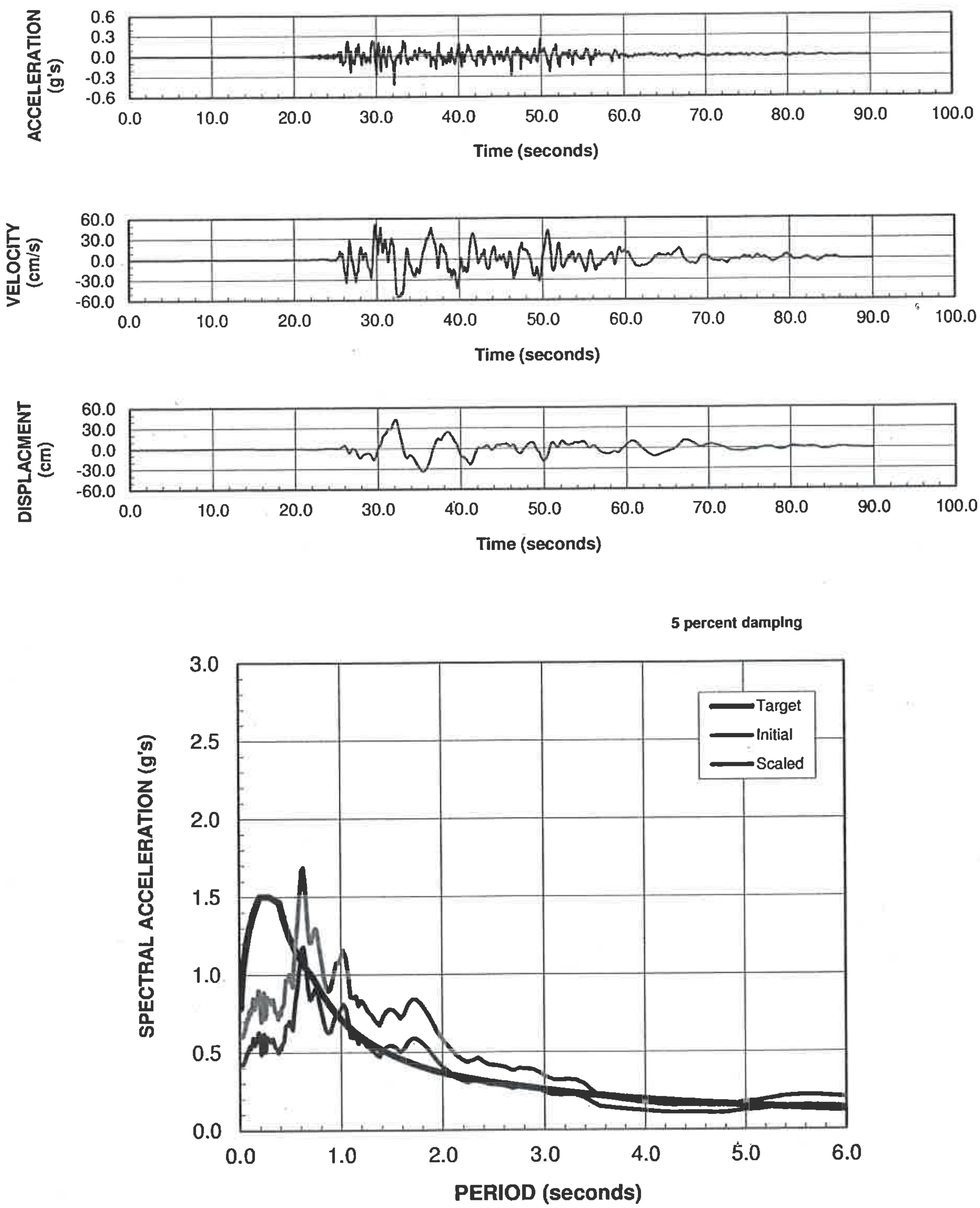
350 MISSION STREET San Francisco, California	**MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1979 IMPERIAL VALLEY EARTHQUAKE HOLTVILLE 225 Deg.**
Treadwell&Rollo *A LANGAN COMPANY*	Date 04/25/12 | Project No. 730466502 | Figure D-28



350 MISSION STREET San Francisco, California	**MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1979 IMPERIAL VALLEY EARTHQUAKE HOLTVILLE 315 Deg.**
Treadwell&Rollo *A LANGAN COMPANY*	Date 04/25/12 | Project No. 730466502 | Figure D-29



350 MISSION STREET San Francisco, California	**MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1999 CHI CHI EARTHQUAKE TCU 065 E.**
Treadwell&Rollo *A LANGAN COMPANY*	Date 04/25/12 | Project No. 730466502 | Figure D-30



350 MISSION STREET
San Francisco, California

Treadwell&Rollo
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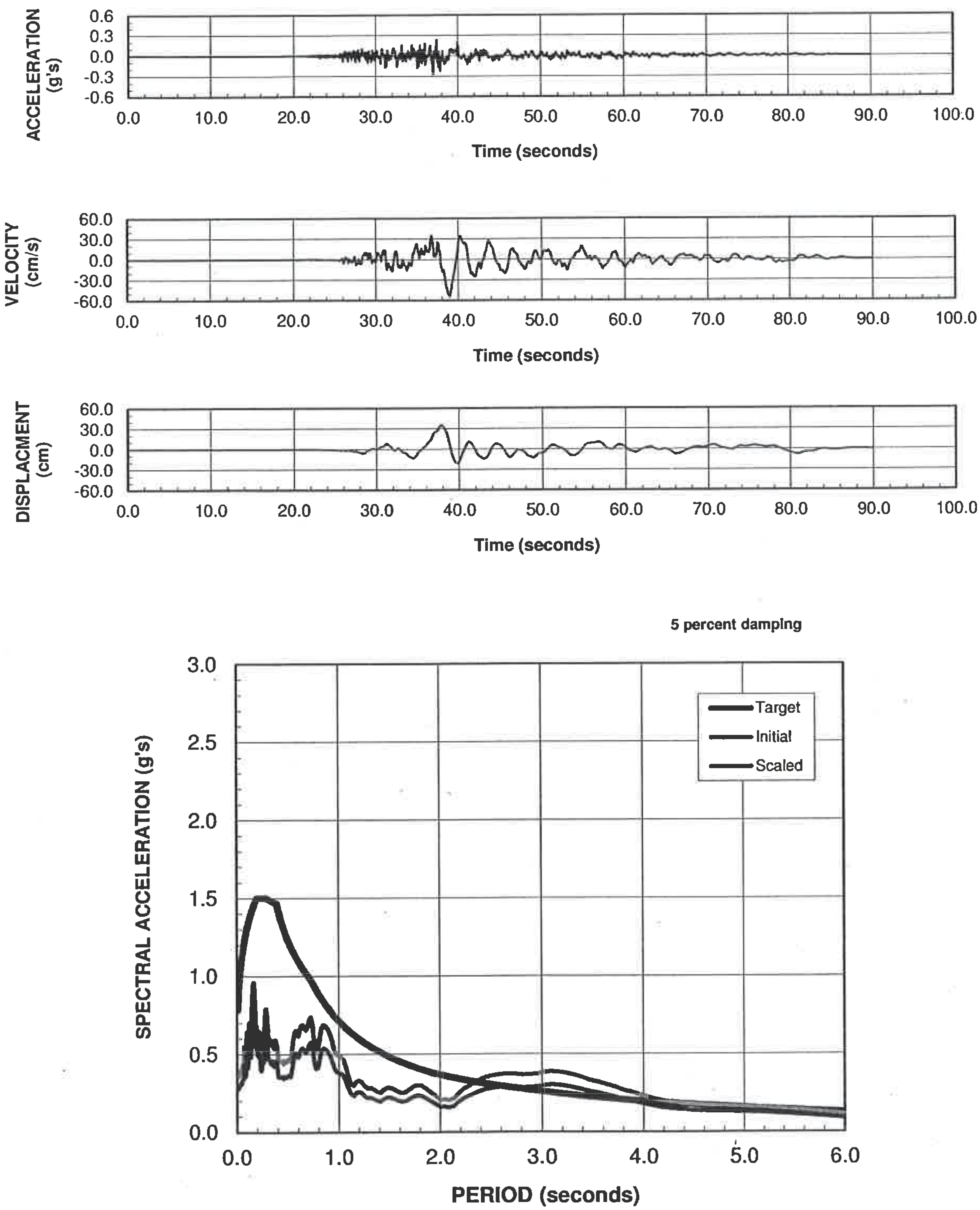
MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1999 CHI CHI EARTHQUAKE TCU 065 N.

Date 04/25/12	Project No. 730466502	Figure D-31



350 MISSION STREET
San Francisco, California

Treadwell&Rollo
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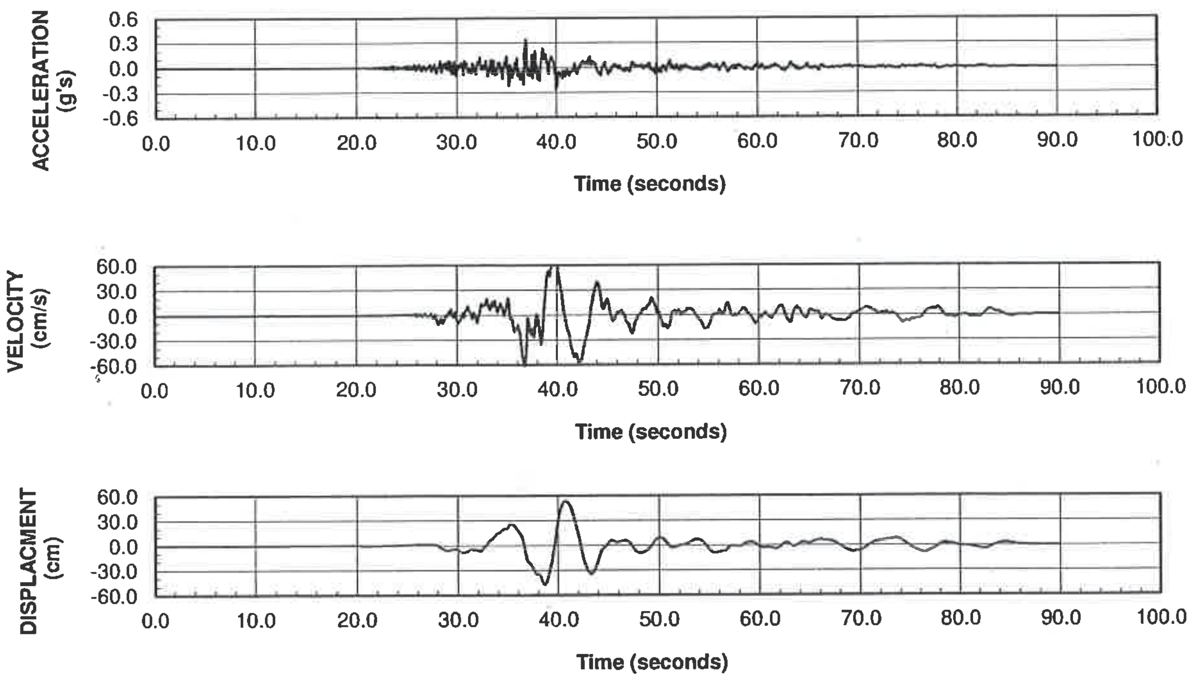
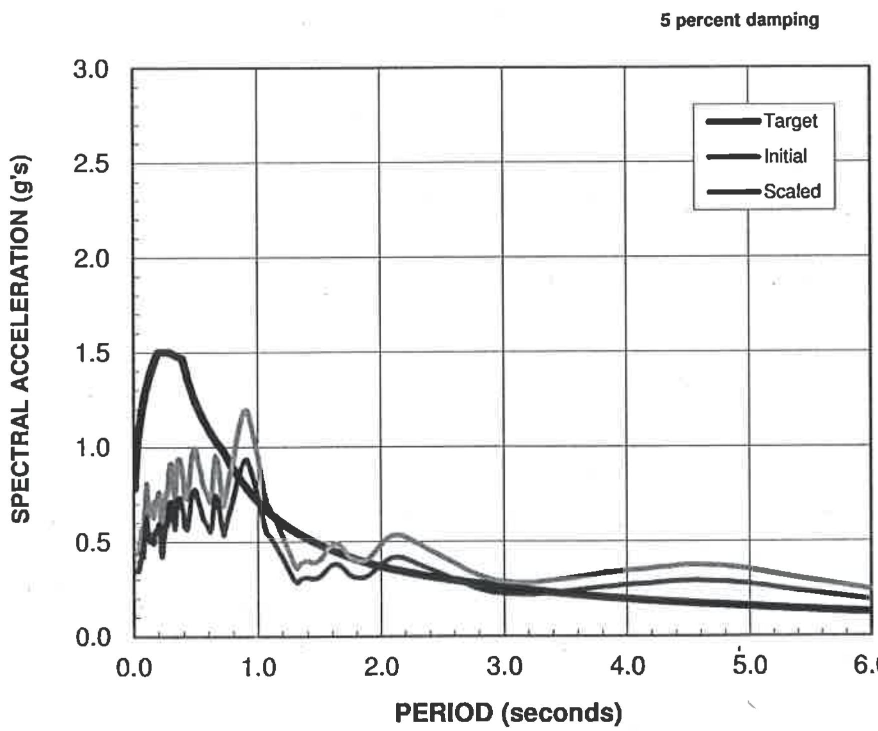
MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1999 CHI CHI EARTHQUAKE CHY 101 E.

Date 04/25/12	Project No. 730466502	Figure D-32





350 MISSION STREET
San Francisco, California

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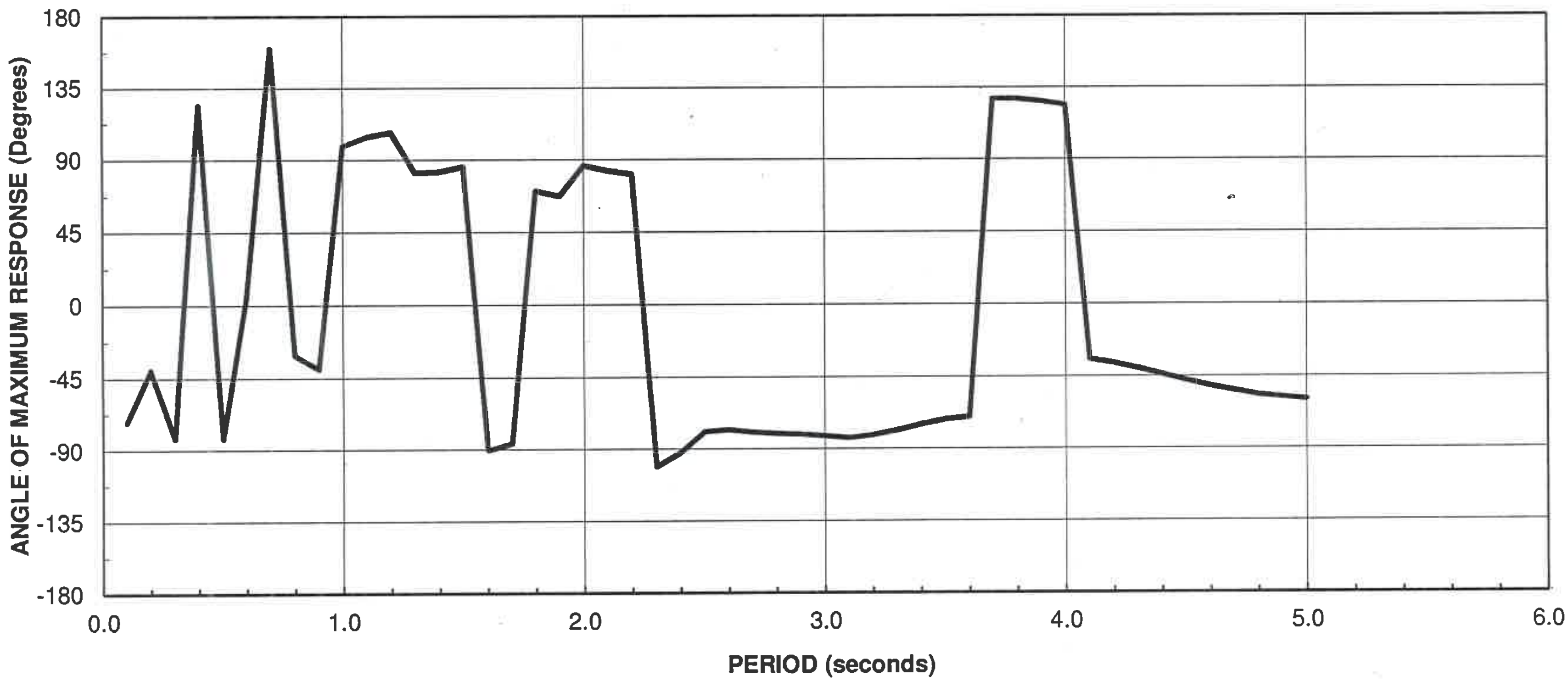
MCE SCALED TIME HISTORY AND RESPONSE SPECTRUM 1999 CHI CHI EARTHQUAKE CHY 101 N.

Date 04/25/12	Project No. 730466502	Figure D-33



Damping Ratio = 5%

350 MISSION STREET San Francisco, California		
ANGLE OF TWO DIRECTIONAL MAXIMUM ACCELERATION RESPONSE - ABBAR		
Date 04/25/12	Project No. 730466502	Figure D-34

Treadwell&Rollo
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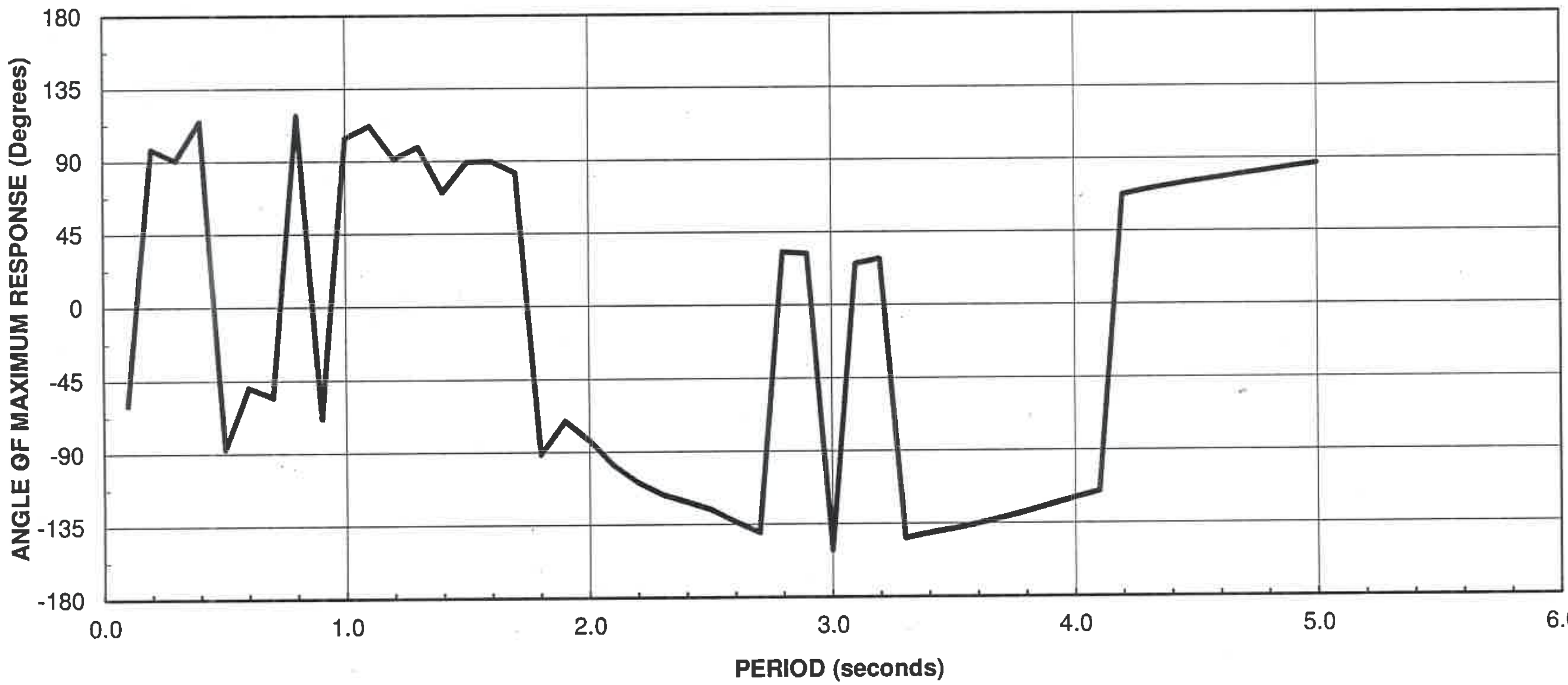
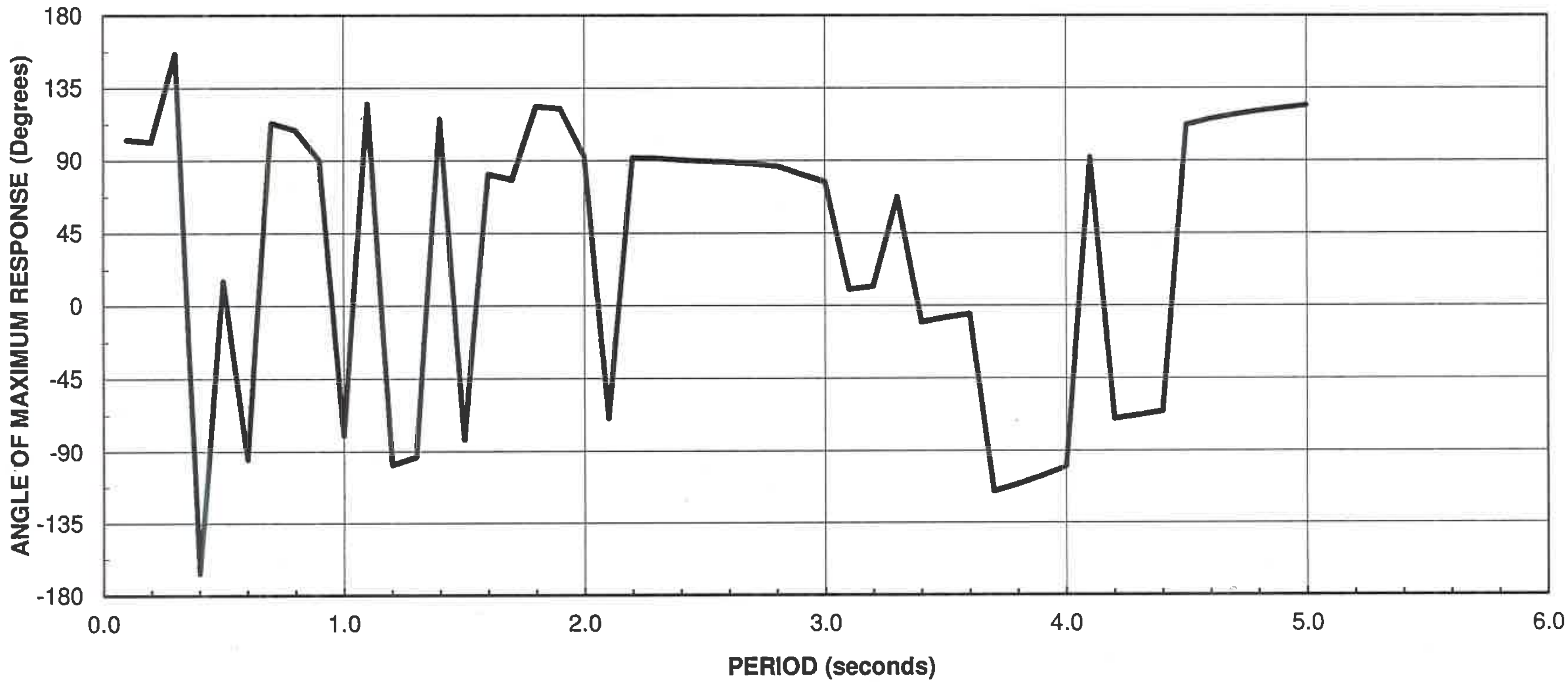


Damping Ratio = 5%

350 MISSION STREET San Francisco, California		
ANGLE OF TWO DIRECTIONAL MAXIMUM ACCELERATION RESPONSE - CHY101		
Date 04/25/12	Project No. 730466502	Figure D-35
Treadwell&Rollo A LANGAN COMPANY		



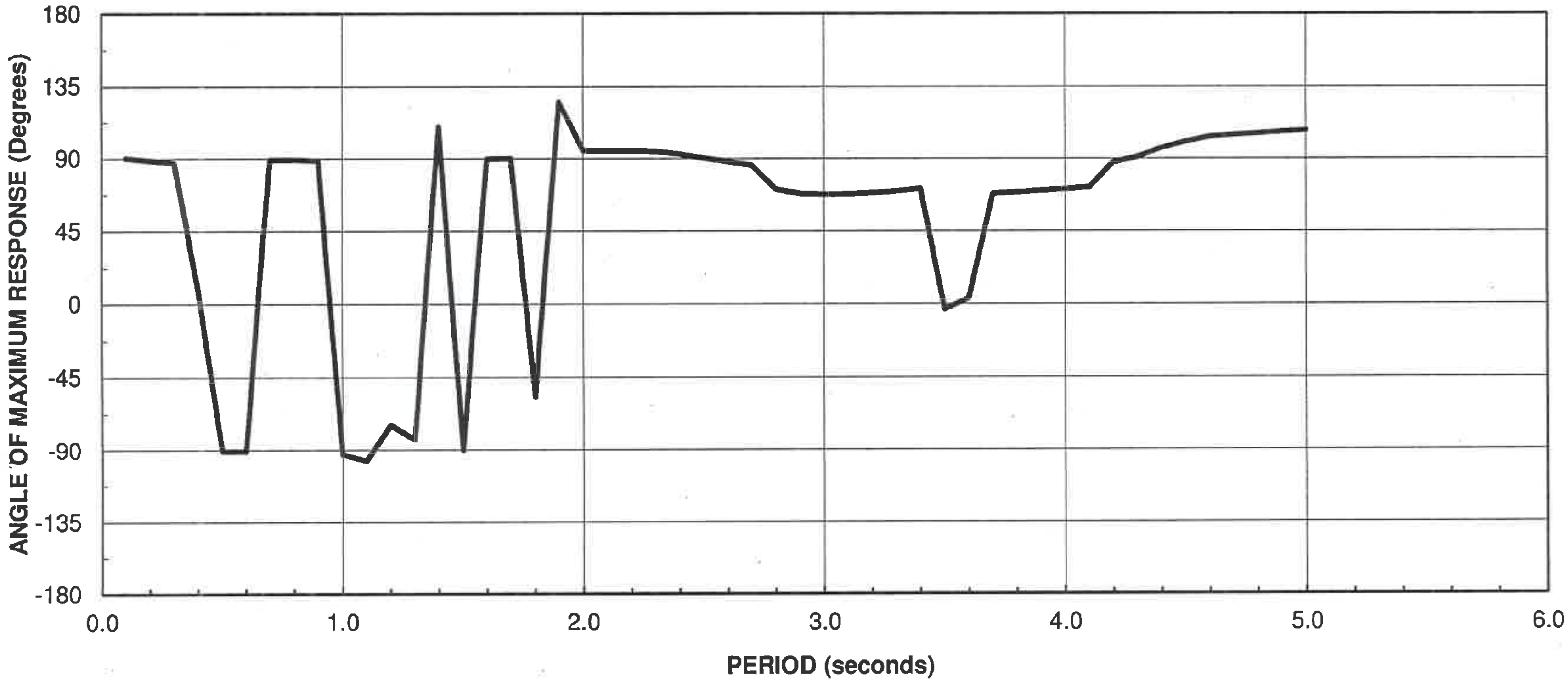
Damping Ratio = 5%

350 MISSION STREET
San Francisco, California

ANGLE OF TWO DIRECTIONAL MAXIMUM ACCELERATION RESPONSE - DUZCE (DUZCE EQ.)

Date 04/25/12	Project No. 730466502	Figure D-36

Treadwell&Rollo
A LANGAN COMPANY



Damping Ratio = 5%

350 MISSION STREET
San Francisco, California

ANGLE OF TWO DIRECTIONAL MAXIMUM ACCELERATION RESPONSE - DUZCE (KOCAELI EQ.)

Date 04/25/12	Project No. 730466502	Figure D-37

Treadwell&Rollo
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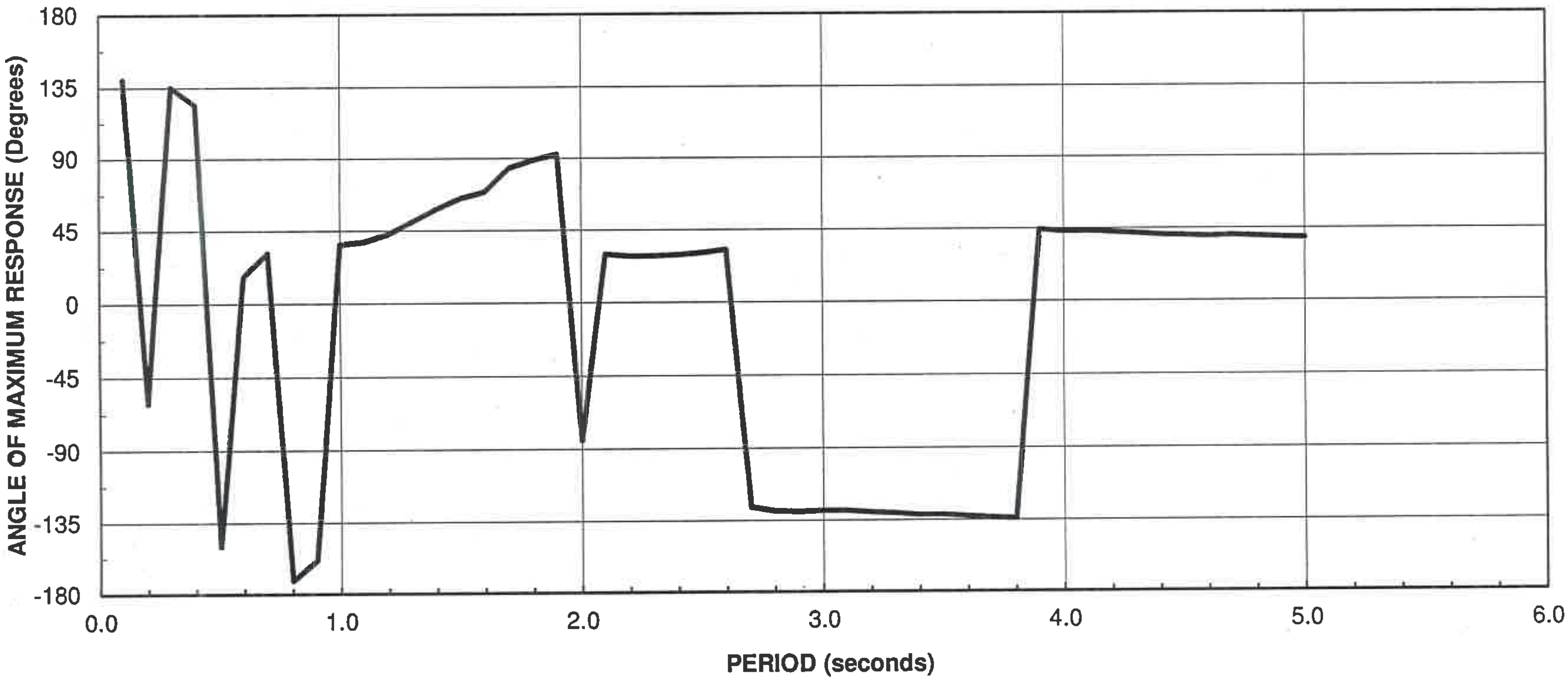

Damping Ratio = 5%

350 MISSION STREET San Francisco, California		
ANGLE OF TWO DIRECTIONAL MAXIMUM ACCELERATION RESPONSE - HOLTVILLE		
Date 04/25/12	Project No. 730466502	Figure D-38

Treadwell&Rollo
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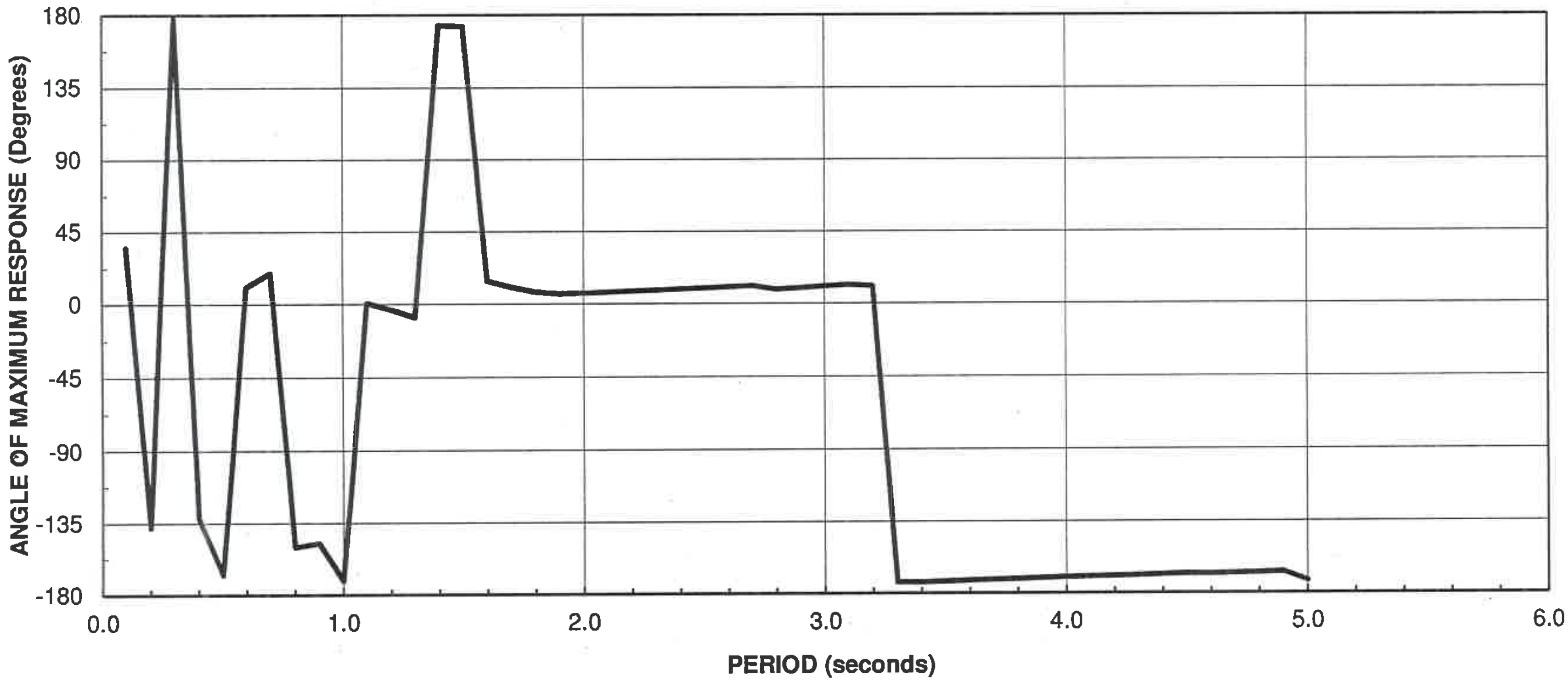

Damping Ratio = 5%

350 MISSION STREET
San Francisco, California

ANGLE OF TWO DIRECTIONAL MAXIMUM ACCELERATION RESPONSE - LGPC

Date 04/25/12	Project No. 730466502	Figure D-39

Treadwell&Rollo
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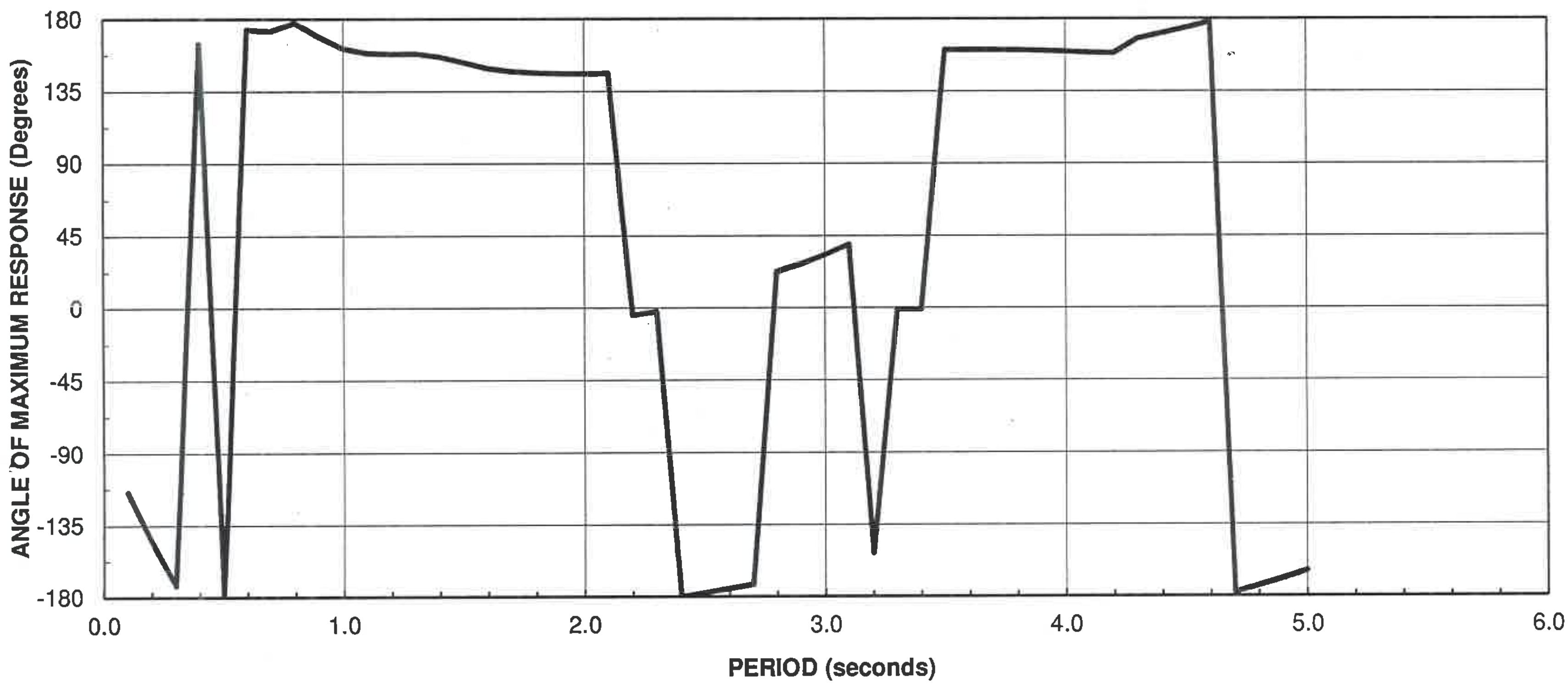
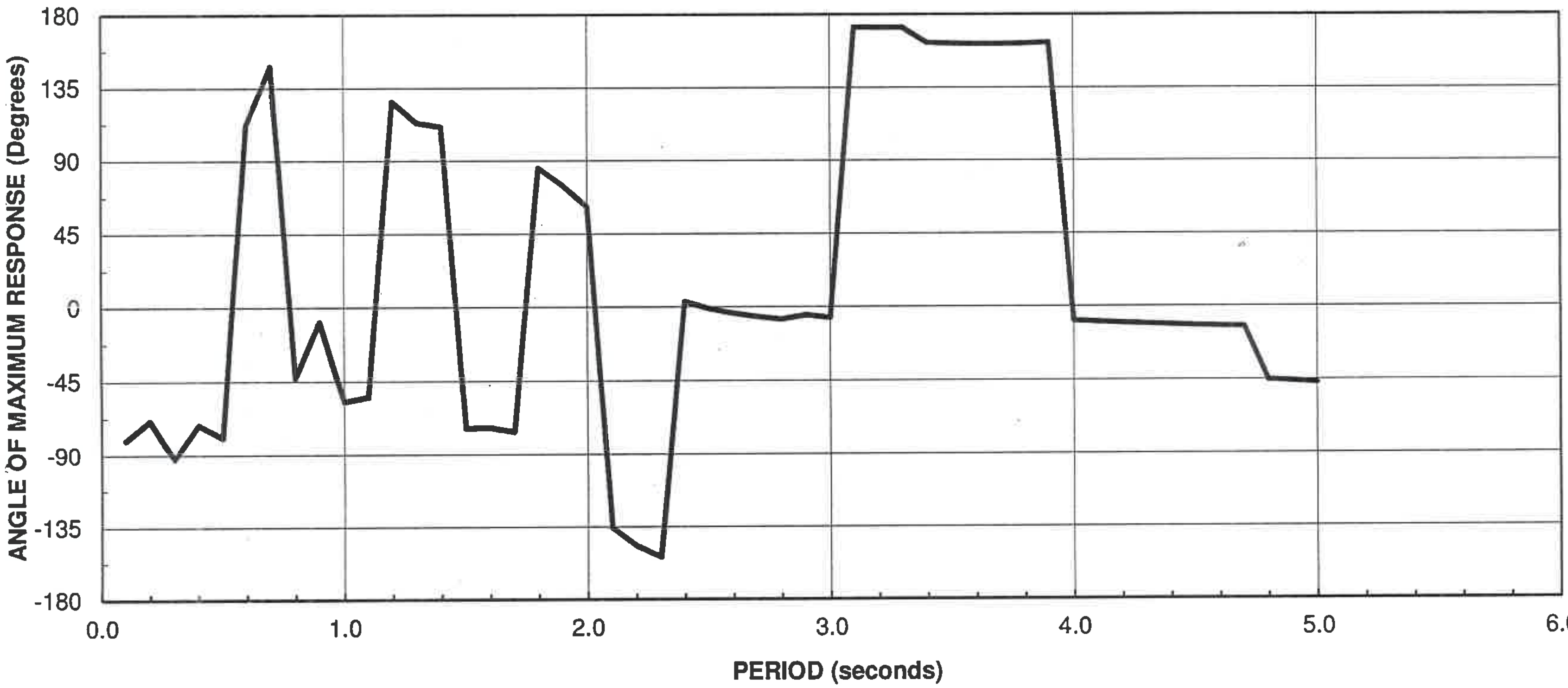


Damping Ratio = 5%

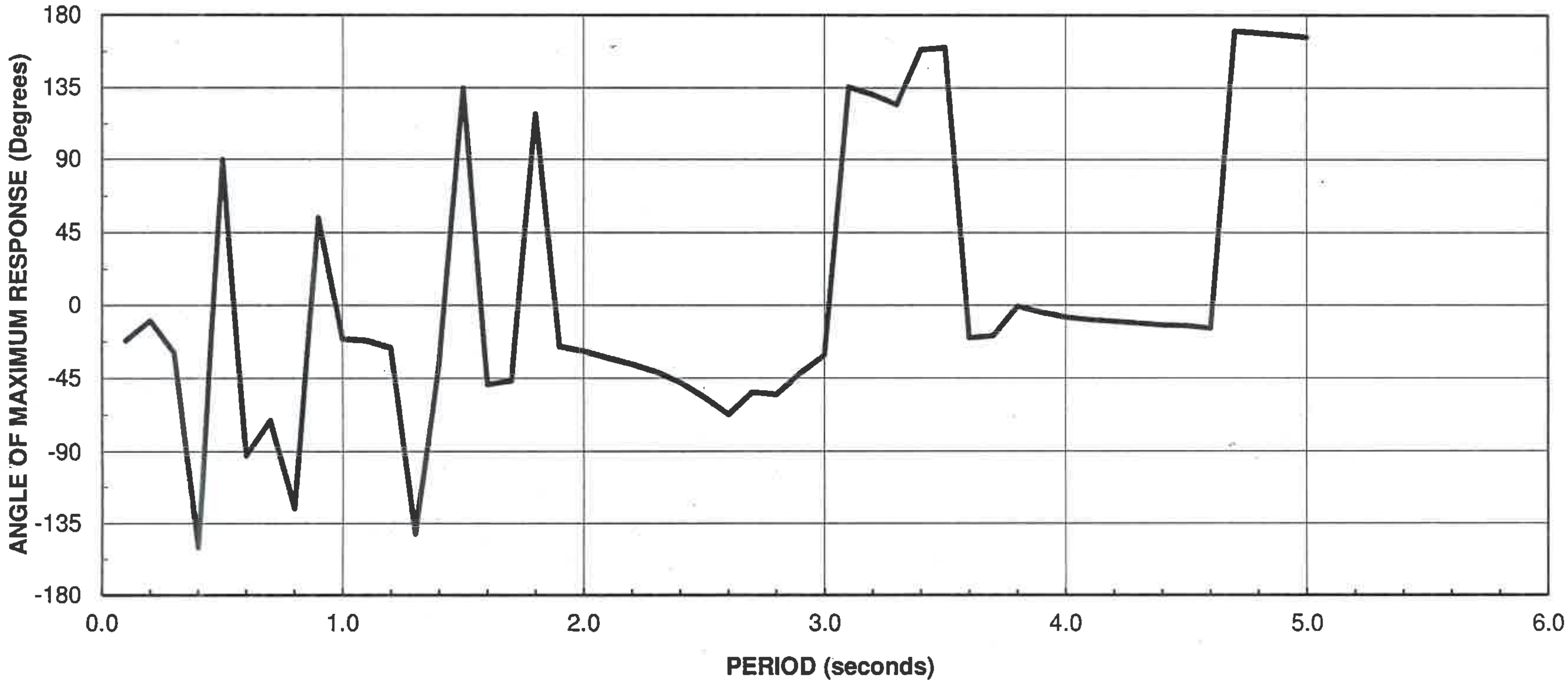
350 MISSION STREET San Francisco, California
ANGLE OF TWO DIRECTIONAL MAXIMUM ACCELERATION RESPONSE - PS-10
Date 04/25/12 · Project No. 730466502 · Figure D-40
Treadwell&Rollo *A LANGAN COMPANY*



Damping Ratio = 5%

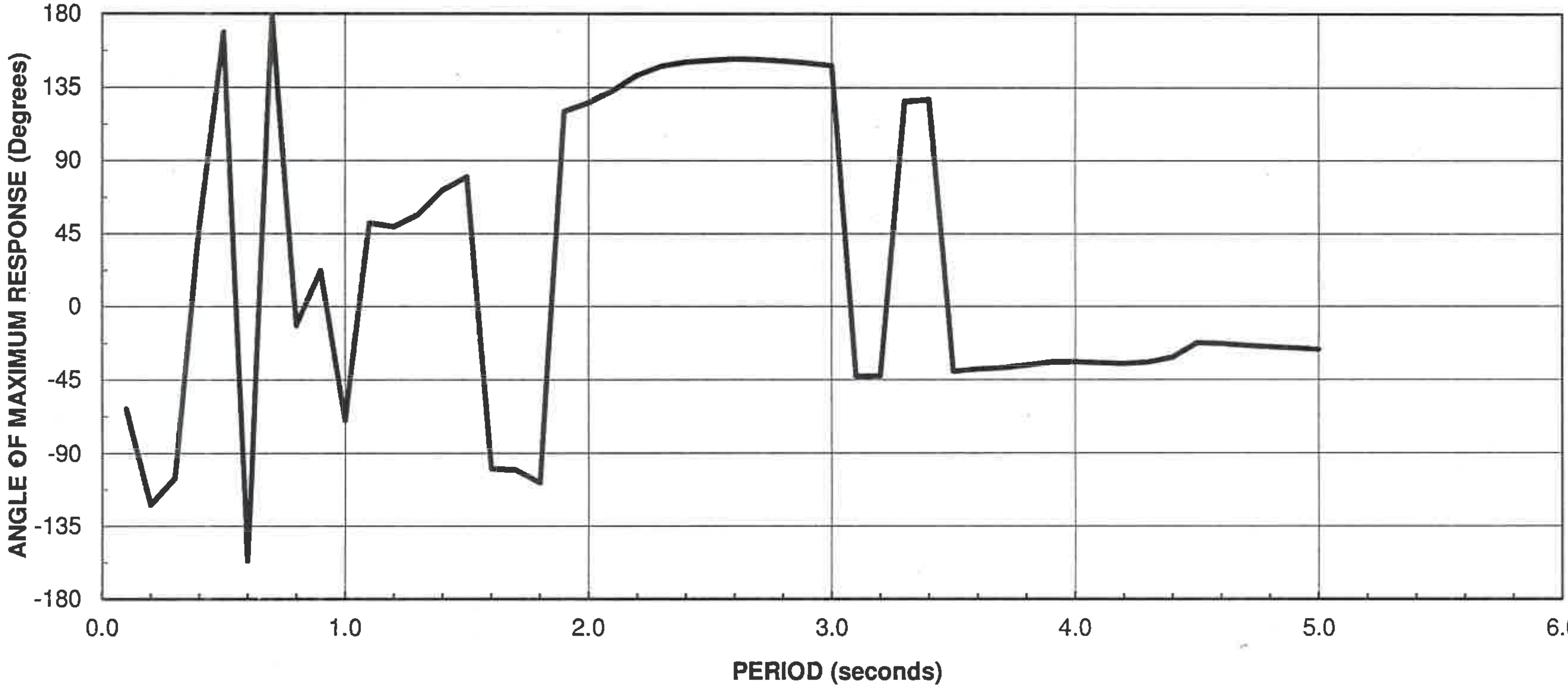
350 MISSION STREET San Francisco, California		
ANGLE OF TWO DIRECTIONAL MAXIMUM ACCLERATION RESPONSE - SYLMAR		
Date 04/25/12	Project No. 730466502	Figure D-41

Treadwell&Rollo
A LANGAN COMPANY



Damping Ratio = 5%

350 MISSION STREET San Francisco, California		
ANGLE OF TWO DIRECTIONAL MAXIMUM ACCELERATION RESPONSE - TCU065		
Date 04/25/12	Project No. 730466502	Figure D-42
Treadwell&Rollo A LANGAN COMPANY		



Damping Ratio = 5%

350 MISSION STREET
San Francisco, California

ANGLE OF TWO DIRECTIONAL MAXIMUM ACCELERATION RESPONSE - YARIMCA

Date 04/25/12	Project No. 730466502	Figure D-43

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