OLMM
CONSULTING
ENGINEERS

701 Sutter St., 4th Floor, San Francisco, CA 94109
Tel: (415) 882-9449

1305 Franklin Street, Suite 312, Oakland, CA 94612
Tel: (510) 433-0828

March 23, 2017

Mr. Hamid Fatehi, SE
Chief Project Manager
COWI Marine
1300 Clay Street, Suite 700
Oakland, CA 94612

Subject: Building 6, Pier 70, Port of San Francisco
Update of Structural & Geotechnical Condition Assessment
(SF Port CSO#: CO-07)

Dear Hamid:

Please find attached herewith the following reports summarizing the findings and recommendations from the condition assessment of Building 6 at Pier 70. The scope of this work was outlined in Port CSO CO-07.

1. "February 2017 Pier 70, Building 6 Inspection, Rev 1", Feb. 2017, by COWI Marine
2. "Geotechnical Letter Report, Pier 70, Building 6", dated March 22, 2017, by Geotechnical Consultants, Inc.
3. "Tier-1 Seismic Evaluation, Building 6, Pier 70, Port of San Francisco, Rev. 1", dated March 23, 2017, by OLMM Consulting Engineers

Please contact me if you have questions or require further information.

Thank you.

Sincerely yours,
OLMM Consulting Engineers

Sunil Gupta, PhD, SE
President

PIER 70 BUILDING 6 INSPECTION REPORT

TITLE	February 2017 Pier 70 Building 6 Inspection, Rev 1.
DATE OF INSPECTION	Monday, February 13, 2017
TIME	7:00AM to 11:30AM
PLACE	Pier 70, San Francisco
INSPECTION BY	Jim Kearney, P.E./Soren Morch, P.E.
PREPARED BY	JWK/SRNM
PROJECT NO	A077120

ADDRESS COWI Marine North America
1300 Clay St.
7th Floor
Oakland, CA 94612

TEL 510-839-8972
FAX 510-839-9715
WWW cowi-na.com

An above water, below deck inspection of Building 6 at Pier 70, was conducted on Monday February 13th, 2017, by COWI NA. The inspection was performed from a boat in the water and on foot where possible to document existing conditions below the deck and report on current conditions. This inspection was limited to the structure that was visible above the waterline and shoreline. The tide level at the time of the inspection of Pier 70 ranged from approximately +3.5 ft MLLW to + 6.0 ft MLLW.

Figure 1 Pier 70 Building 6 as seen from the water looking west.

Findings

The structure supporting Building 6 consists of timber piles with concrete pile caps, short concrete columns, concrete cap beams, and concrete deck. There are 52 bents in the north-south direction at approximate 10 ft. spacing and 10 bents in the east-west direction at approximate 9 ft. spacing. A steel sheet pile wall is present just west of the row AA. We adopted the bent and row designations 1-52 and A-I from an earlier report to avoid confusion and added rows AA and BB which were not mentioned therein.

Timber piles located in the water and tidal zone along the shoreline are typically partially encased in concrete from the mudline. The encasements vary in height and extend to about one foot from the concrete pile cap. The concrete sleeves do not appear to be present at all between bent 1 and approximately bent 20 and only some piles are sleeved at bents 21 and 22. From bent 5 to bent 22 many piles are not visible because the soil line reaches the concrete pile cap.

Piles 19-B and 19-C are located in an area of soil that appears to be scoured by tidal action. These two piles exhibit loss of section due to either marine borer infestation, erosion, or both. While the loss is significant, we do not judge it to be critical at this time. Active borer activity was not apparent at the time of our inspection.

Timber piles are missing at 13 of 19 bents from bent 34 through bent 52. Six bents are missing more than one pile. A majority of the previously installed concrete sleeves observed were found to have cracks, or missing sections. However, the timber looked sound with few signs of previous section loss in the visible areas of the piles at the time of the inspection.

Isolated spalling, with and without exposed reinforcing, was noted in the northwest area of the soffit of the slab and cap beams.

The fill on shore west of the building is retained by a steel sheet pile wall running the full length of the building. This sheet pile wall is heavily corroded.

These defects are shown on Drawing SK-A and SK-B. A summary of findings is shown in Table 1 below:

Table 1 Summarized Findings

Location	Estimated extent of damage
Missing piles	24
Piles with broken sleeve	142
Damaged soffit	60 sf
Damaged cap beams	120 lf
Exposed piles without sleeve	97
Significant loss of section at timber piles	2
Corroded steel sheet pile	100% of inspected area

Figure 2 *Photo showing bents 37 to 39, the first row of piles is row I, cracked concrete sleeves are seen on the majority of the timber piles and the four pile cluster at I-37 is missing three piles.*

Figure 3 *Photo showing spall damage with corroded rebar to cap beam at row 51 A-B*

Figure 4 *Photo showing typical soffit spall damage with corroded rebar between bents 45-46 row A*

Figure 5 *Steel sheet pile at row BB with timber pile at bent 45 row AA shown in front.*

Figure 6 *Bents 35-35.5 36 row A. Assumed piles encased in concrete and joined with a cap beam.*

Figure 7 *Large concrete encasements at row A located at all bents with four-pile clusters at row I. It is assumed that there are four piles in these encasements to match the outside row, but these were not visible.*

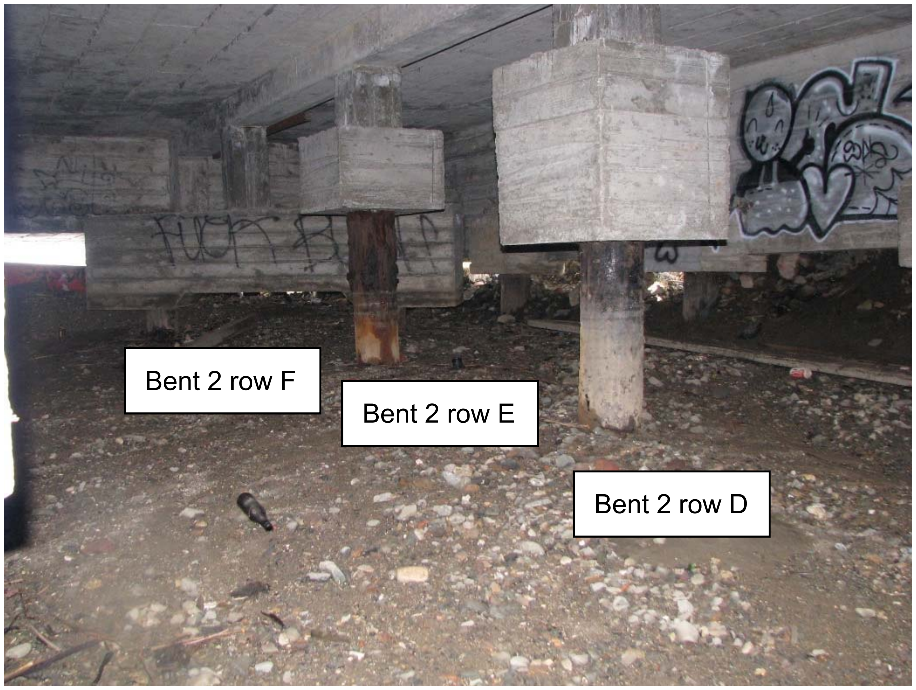


Figure 8 The southeast corner of the building is supported by piles out of the water as shown above. These piles are not encased in concrete. Bent 2 row D, E and F. Note that row F has an atypical pile arrangement with two piles and a cap beam supporting a column. The soil in this area was damp but did not appear to be regularly inundated by the tide.

Figure 9 Piles at 19 B and C, Surface damage due to borers, erosion, or both. Soil is

Conclusion

The missing piles are a critical deficiency.

The sheet pile wall is heavily corroded.

With a limited number of exceptions, the concrete substructure appears to be in sound condition.

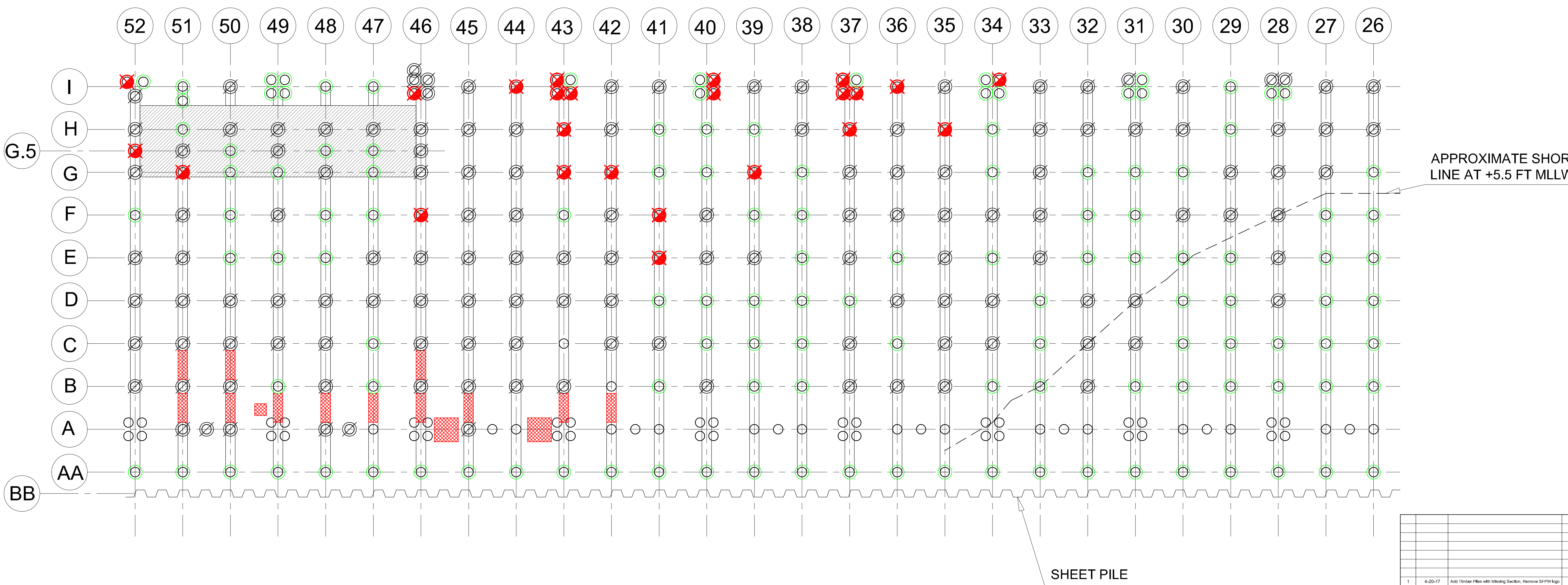
The exceptions to that observation are:

- Spall to concrete beams and soffit along the northwestern section of the building near row A.
- Existing concrete sleeves around the tops of the timber piles are cracking, however timber piles appear sound below the concrete in the areas visible during inspection.

Recommendations

Based on as-is conditions

- Replace missing 24 piles.
- Repair spalled concrete at slab soffit and beams.
- Develop repair plan for corroded steel sheet pile wall at row BB. Possibly install new wall west of row BB and tie into loading dock. Let original wall corrode.
- Install grout filled fiberglass sleeves at piles 19-B and 19-C to restore lost section and preclude further damage to the piles.
- To extend the useful life of the building and protect against future marine borer damage, new sleeves or wraps could be installed on exposed timber piles subject to immersion. However, extensive marine borer damage was not apparent on the most of piles observed, with or without previous concrete sleeves. A program of continued monitoring, including an underwater inspection, could be instituted and damage addressed as and when it occurs. These inspections would also be able to track new damage to the concrete slab, soffit, beams, and caps.
- Vehicular traffic or storage of any material should be barred from areas over, or within one bent of, missing piles until the piles are replaced.



LEGEND:

- AREA WITH LOWERED SOFFIT
- SPALLED CONCRETE CAP OR SOFFIT
- EXPOSED TIMBER PILE WITHOUT CONCRETE SLEEVE
- CRACKED OR BROKEN CONCRETE SLEEVE
- MISSING PILE
- PILE WITH CONCRETE SLEEVE
- TIMBER PILE W/ LOSS OF SECTION
- TIMBER PILE

1 SK-A GENERAL PLAN 26-52
SCALE: NTS

TRUE NORTH

REFERENCE NORTH

REV	DATE	DESCRIPTION	BY	CHK
1	4-20-17	Add Timber Piles with Missing Section, Remove SFPW logo	juba	JWK

CLIENT

CONSULTANT

COWI Marine North America
A Business Unit of COWI North America, Inc.
1300 Clay Street, 7th Floor Oakland, CA 94612
Tel.: 510.839.8972 Fax: 510.839.9715
Website: www.cowi-na.com

PROJECT TITLE
PIER 70 BUILDING 6 INSPECTION

CONFORMED SET

DRAWING TITLE
OVERALL SITE PLAN

SCALE	DESIGNED	APPROVED
AS SHOWN	SRNM	-
DRAWN: SRNM	CHECKED: -	DATE: 2-14-2017
JOB NO.: A077120	DRAWING NO.: SK-A	REV.: 0

PLOTTED BY: JIM KEARNEY PLOT DATE: 4/20/2017 2:29:23 PM

FILE LOCATION: \\COWI.NET\PROJECTS\A075000\A077120\CAD\PIER 70 BLDG 6\A077120 PLAN R1.DWG

PLOTTED BY: JIM KEARNEY PLOT DATE:4/20/2017 2:31:51 PM
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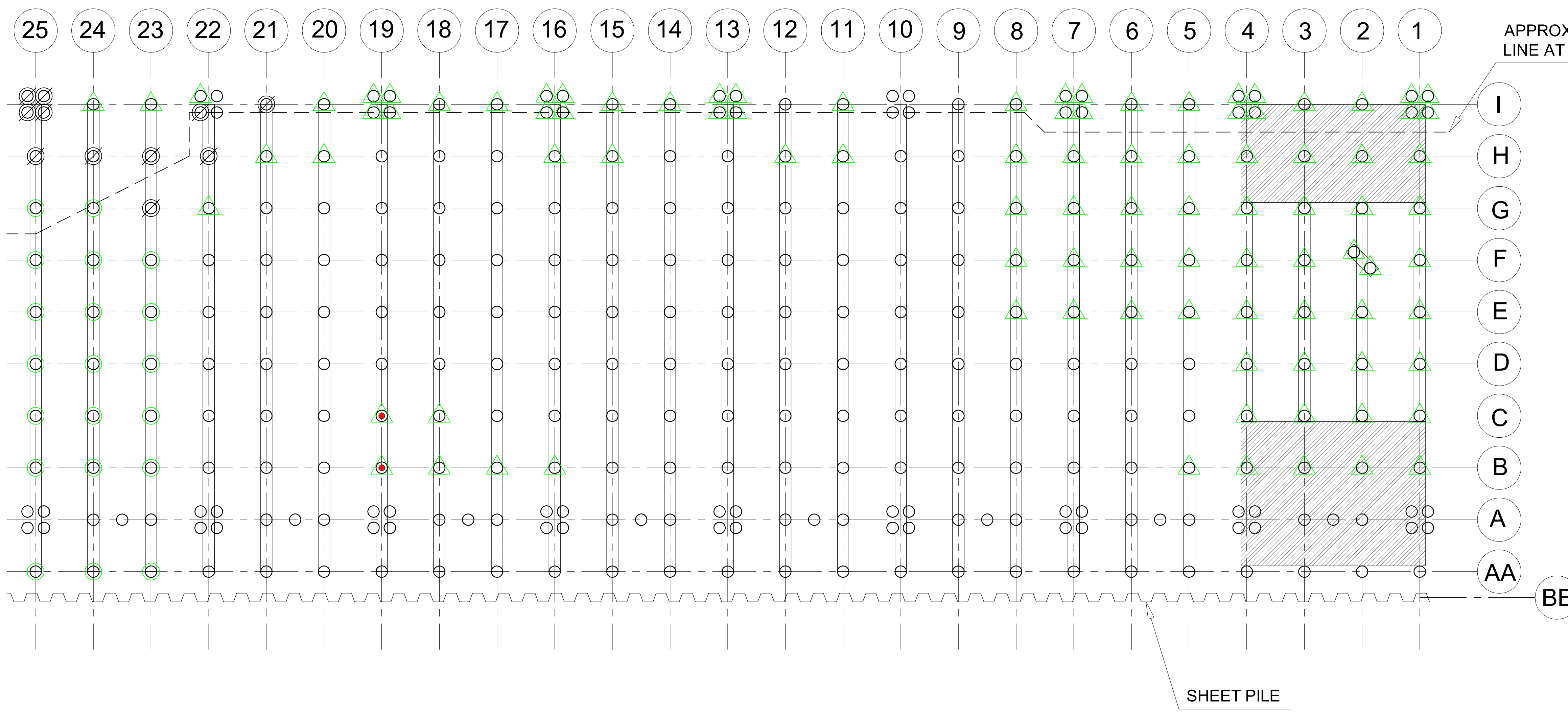

LEGEND:

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SPALLED CONCRETE CAP OR SOFFIT

EXPOSED TIMBER PILE WITHOUT CONCRETE SLEEVE

CRACKED OR BROKEN CONCRETE SLEEVE

MISSING PILE

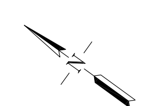
PILE WITH CONCRETE SLEEVE

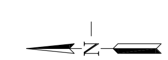
TIMBER PILE W/ LOSS OF SECTION

TIMBER PILE

1 SK-B GENERAL PLAN 1-25

SCALE: NTS


TRUE NORTH


REFERENCE NORTH

REV	DATE	DESCRIPTION	BY	CHK
0				

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COWI Marine North America
A Business Unit of COWI North America, Inc.
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Tel.: 510.839.8972 Fax: 510.839.9715
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SCALE	DESIGNED	APPROVED
AS SHOWN	SRNM	-
DRAWN	CHECKED	DATE
SRNM	-	2-14-2017
JOB NO.	DRAWING NO.	REV.
A077120	SK-B	0

Pier 70 Building 6
Replacement Pile Sleeve Repair Length Estimates for Damaged Concrete Pile Sleeves

Bent	Row	Distance from b.o. Cap to Mudline, Ft.*	Fiberglass Pile Sleeve Length
19	B**	3	4.5
19	C**	4	5.5
21	I	8	9.5
22	I-3	6	7.5
22	H	8	9.5
23	H	8	9.5
23	G	8	9.5
24	H	8	9.5
25	I-1	6	7.5
25	I-2	6	7.5
25	I-3	6	7.5
25	I-4	6	7.5
25	H	8	9.5
26	I	8	9.5
26	H	8	9.5
27	I	7	8.5
27	H	9	10.5
27	G	9	10.5
28	I-1	7	8.5
28	I-2	7	8.5
28	H	9	10.5
28	G	8.5	10
28	F	8	9.5
28	E	3	4.5
28	D	2	3.5
29	G	9	10.5
29	F	8	9.5
30	I	8	9.5
30	H	10	11.5
30	F	9	10.5
31	I-1	8	9.5
31	H	10	11.5
31	D	8	9.5
31	C	6	7.5
32	I	10	11.5
32	H	10	11.5
32	D	8.5	10
32	C	8	9.5
32	B	6	7.5
33	I	10	11.5
33	H	10	11.5
33	G	9.5	11
33	F	9	10.5
33	E	9	10.5
34	F	10	11.5
34	D	9	10.5
34	C	9	10.5

*Lengths are approximated based on interpolation of measurements at perimter. Verify all dimensions in field.

Bent	Row	Distance from b.o. Cap to Mudline, Ft.*	Fiberglass Pile Sleeve Length
35	I	11	12.5
35	G	10.5	12
35	F	10	11.5
35	E	9.5	11
35	D	9.5	11
35	C	9	10.5
35	B	8.5	10
36	H	10	11.5
36	G	10.5	12
36	F	10	11.5
36	D	9.5	11
36	B	9	10.5
37	G	11.5	13
37	F	10	11.5
37	E	10	11.5
37	C	9	10.5
37	B	9	10.5
38	I	12	13.5
38	F	12	13.5
39	I	13	14.5
39	E	11.5	13
40	F	12	13.5
40	E	11.5	13
40	B	9	10.5
41	I	14	15.5
41	C	10	11.5
42	I	14	15.5
42	H	13	14.5
42	F	12	13.5
42	E	11.5	13
42	D	11	12.5
42	C	10	11.5
43	E	11.5	13
43	D	11	12.5
43	B	9	10.5
44	H	12	13.5
44	G	11	12.5
44	F	10.5	12
44	E	10	11.5
44	D	9.5	11
44	C	9	10.5
44	B	9	10.5
45	I	13	14.5
45	H	12	13.5
45	G	11	12.5
45	F	10.5	12
45	E	10	11.5
45	D	10	11.5
45	C	9.5	11
45	B	9	10.5
45	A	8	9.5

Bent	Row	Distance from b.o. Cap to Mudline, Ft.*	Fiberglass Pile Sleeve Length
46	I-1	11	12.5
46	I-2	11	12.5
46	I-3	11	12.5
46	I-4	11	12.5
46	H	12	13.5
46	G.5	11.5	13
46	G	11	12.5
46	E	10	11.5
46	D	10	11.5
46	C	9.5	11
46	B	9	10.5
47	H	12	13.5
47	E	10	11.5
47	D	9.5	11
47.5	A	8	9.5
48	H	12	13.5
48	G	11	12.5
48	D	9.5	11
48	C	9	10.5
48	B	8.5	10
48	A	8	9.5
49	H	12	13.5
49	G.5	12	13.5
49	F	10.5	12
49	D	9.5	11
49	C	9	10.5
50	I	12	13.5
50	H	11.5	13
50	D	9	10.5
50	C	9	10.5
50	B	8.5	10
50	A	8	9.5
50.5	A	8	9.5
51	G.5	11	12.5
51	F	10	11.5
51	E	9.5	11
51	D	9	10.5
51	C	9	10.5
51	B	8.5	10
51	A	8.5	10
52	I	10	11.5
52	H	11	12.5
52	G	11	12.5
52	E	10	11.5
52	D	9.5	11
52	C	9	10.5
52	B	8.5	10

** Piles 19-B and 19-C were not previously sleeved, but are the only piles that showed significant section loss.

Generally, the timber piles exposed by missing portions of existing concrete sleeves did not exhibit extensive marine borer damage or other section loss.

Assume 14 inch diameter piles, average. So 18 inch diameter fiberglass sleeves, if a sleeving program is chosen, should be used for estimating.

The table above does not, with the exception of Piles 19 B and C, include piles which were never previously fitted with concrete sleeves but are exposed between the mudline and the pile cap and subject to immersion.

GEOTECHNICAL CONSULTANTS, INC.
Geotechnical Engineering • Geology • Hydrogeology

COWI OLMM JV
1300 Clay Street, 7th Floor
Oakland, California 94612

March 22, 2017
Project No. SF16027

Attention: Mr. Hamid Fatehi, P.E., S.E.

Subject: Geotechnical Letter Report
Pier 70, Building 6
San Francisco, California

Dear Mr. Fatehi:

In accordance with our proposal dated December 16, 2016, we performed a limited geotechnical assessment of Building 6 at Pier 70 at the Port of San Francisco to support a structural conditions assessment being performed by COWI OLMM JV. Our services included a review of available historical geotechnical reports and geologic maps for the site and vicinity, a site reconnaissance on February 27, 2017, discussions about the building's conditions with project team members, and preparation of this geotechnical letter report.

Building and Site Conditions. Building 6 is a light warehouse structure measuring 512 feet long by 72 feet wide and 52 feet tall, and was built in 1941 as part of the World War II shipbuilding effort (Carey & Co., 2014). The structure is situated at an angle to the shoreline and to the rest of the Pier 70 structures with approximately one-half of the structure overwater and one-half on land (***Photo 1***). The entire structure is supported by timber piles which were likely creosote-treated to protect against marine organisms. Most of the timber piles within the tidal zone are also protected with a concrete sleeve. A sheet pile wall, likely constructed to retain the fill soils for the adjacent parking area, is present on the west side of Building 6.

Our review of available historical data indicate that during the late 1800s and early 1900s, Union Iron Works developed the land, and used rock quarried from nearby Irish Hill and Potrero Point to fill the tidal flats formerly occupying the site (Treadwell & Rollo, 2012; Carey & Co., 2014). Based on a 1935 aerial photograph (Treadwell & Rollo, 2012; Pacific Aerial Surveys), the 1935 shoreline was not much altered just prior to the construction of Building 6 except a soil wedge at the southern portion of the building. The grade is level at approximately elevation 12 feet (NAVD 88) on the west side of the structure, and the shoreline edge and mudline appears to descend gently bayward.

Photo 1 – Pier 70 - Building 6

Subsurface Conditions. Our understanding of the subsurface conditions is based on a review of the preliminary geotechnical investigation report for Pier 70 (Treadwell & Rollo, 2012) and geologic maps (CDMG, 1969; Schlocker, 1974; USGS, 2000), and by a site reconnaissance performed on February 27, 2017. No additional subsurface investigation was performed for this limited geotechnical assessment. Based on this review, the site is underlain by artificial fill and young bay mud which overlies Franciscan Complex bedrock.

The historical data indicate that the artificial fill at the Building 6 site is variable and consists of predominantly sand and gravel with cobbles and layers of clayey sand and clayey gravel. Industrial waste and debris including slag, glass, metal, asphalt, concrete, brick, nuts, and nails are contained throughout the artificial fill depth. The artificial fill appears to be fairly well compacted, especially considering the era of fill placement during the late 1800s to early 1900s. This conclusion is based on SPT blow counts from historical borings and observation of the near surface soils along the shoreline edge (***Photo 2***).

Photo 2 – Exposed Fill Soils under Building 6 at Rows 15 and 16

Young bay mud underlies the artificial fill at at a depth of about 35 to 40 feet below existing ground surface. The young bay mud is a soft to medium stiff, weak, compressible, fat clay and was deposited in San Francisco Bay over approximately the last 8,000 years (CDMG, 1969). As indicated on Section B-B' on Figure 11 of Treadwell & Rollo's report (2012), the young bay mud thickens significantly bayward from approximately 23 feet thick at the south end of Building 6 to over 80 feet thick approximately 300 feet offshore.

The young bay mud overlies the sloping Franciscan Complex bedrock surface at the site. The depth to bedrock at Building 6 is estimated to be 50 to 100 feet below ground surface (Treadwell & Rollo, 2012; Schlocker, 1974). The bedrock surface slopes down toward the east.

Geologic Hazards. As the site is located in the seismically active San Francisco Bay Area, a major earthquake on one of the regional active faults can cause strong ground shaking. The San Andreas fault and Hayward fault are the two closest faults at distances of approximately 12 km to the southwest and 17 km to the northeast, respectively. The San Andreas and Hayward faults are capable of generating earthquakes with a maximum moment magnitude of Mw 7.9 and

Mw 6.9, respectively. In accordance with ASCE 7-10 (ASCE, 2013), the peak ground acceleration for evaluation of geologic hazards at the site is estimated at 0.47 g. The site is classified as Site Class E (soft clay). Fault rupture hazards are negligible due to the absence of known active or potentially active faults at the Building 6 site and vicinity.

Other geologic hazards that may affect the site include liquefaction and lateral spread caused by strong ground shaking, inundation by tsunami, and flooding from sea level rise.

Liquefaction is a phenomenon wherein a temporary, partial loss of shear strength occurs in a soil due to increases in pore pressure that result from cyclic loading during earthquakes. Saturated, loose to medium dense sands and silty sands are most susceptible to liquefaction. Consequences of liquefaction can include ground settlements, foundation failure, sand boils, and lateral spreading. Some isolated layers within the artificial fill may be prone to liquefaction due to their loose and granular nature. Based on the density and grain size characteristics of the majority of the fill, however, we do not foresee that liquefaction will be pervasive across the site.

Lateral spreading is a liquefaction-induced ground deformation failure in which near-surface soil layers typically break into blocks that progressively move along a plane of weakness downslope or toward a nearby free face such as a stream channel, river embankment, or a shoreline. Underground facilities and structural elements (e.g., pipelines, spread footings, pile foundations, etc.) that extend through or across a zone of lateral spreading may be pulled apart or sheared. For lateral spreading to occur, liquefaction would need to be triggered along a continuous layer of loose to medium dense granular soil (SPT blow counts less than about 15). This does not appear likely based on a review of limited subsurface information. If nearby borings are indicative of the soil types and densities across the site, the risk of lateral spread is low. It is also noted that no evidence of liquefaction or lateral spread was documented in the area after the 1906 San Francisco Earthquake (Youd and Hoose, 1978) nor after the 1989 Loma Prieta Earthquake (Holzer, 1998).

Inundation by tsunami and flooding due to sea level rise are likely to affect the site as Building 6 is in a low-lying area along the waterfront. Assessing the degree of inundation and impacts of these hazards is beyond the scope of this limited geotechnical assessment.

Discussion and Conclusions. Building 6 is in fair to good condition from geotechnical and foundation support perspectives with some deficiencies noted by previous investigators (ABR Engineers, 2000; Port of San Francisco, 2003; Carey & Co. and OLMM, 2008). During their recent above water, below deck inspection, COWI OLMM JV noted that 24 timber piles were missing from the northeastern quadrant of the building. Although this deficiency should be corrected, the missing piles do not appear to have led to building distress thus far. As-built drawings and pile driving records were not available for Building 6, and therefore it is unknown whether the existing timber piles within the northeastern quadrant reach a hard stratum or if they terminate within the young bay mud. We surmise they may terminate within the young bay mud because of the number of missing piles, though the missing piles could be a result of damage to the piles after installation. At this preliminary stage, we recommend 18-inch diameter by 0.5-inch thick wall steel pipe piles, driven open ended, to bear in the underlying dense sand and clay strata or bedrock. The required length of piles is estimated to be 120 feet. It may be assumed that one steel pipe pile will be needed for each missing timber pile, although this will need to be confirmed during design. Because of the existing superstructure, pile caps and possibly remnants of the missing timber piles, the replacement piles will likely be driven offset from their original location and incorporated into the structure by increasing the size of the pile caps and grade beams. Piles driven from inside the structure will need to be driven in sections and welded together because of the overhead clearance limitations.

During our site reconnaissance, we observed that a sheet pile wall along the western side of Building 6 is corroding (***Photo 3***). This was also noted by COWI OLMM JV during their inspection. The sheet pile wall provides protection of the soils behind the wall from wave erosion and provides some lateral stability to the shoreline. The sheet pile wall can also provide added protection from lateral spread movements in the event that site soils are prone to liquefaction and lateral spreading. The lateral extent of the sheet pile wall on land toward the south side of the building and depth of the sheet pile wall is not known. A replacement bulkhead wall can be considered to extend the useful life of this structure.

It is evident that there has been past settlement of soils from consolidation of the underlying young bay mud. Any improvements that include increasing surficial loads, such as the placement of fill to raise site grades, will subject the site to additional long-term, consolidation related settlements. This should be evaluated and addressed during design of site improvements.

Photo 3 – Corroded Sheet Pile Wall under Building 6 at Row BB, Row 38

Limitations. The findings, discussion and conclusions presented herein are professional opinions based on geotechnical and geologic data and the project as described. The conclusions are based on limited geotechnical subsurface information. Additional geotechnical exploration should be performed during design of improvements to better assess liquefaction and lateral spread potential and geotechnical parameters for foundation design.

Submitted by:
GEOTECHNICAL CONSULTANTS, INC.

Deron J. van Hoff

Deron J. van Hoff, P.E., G.E.
Associate Geotechnical Engineer

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Tier 1 Seismic Evaluation
Building 6, Pier 70
Port of San Francisco, Rev. 1

OLMM Job No.: 2017-01

Prepared for COWI|OLMM JV

Prepared by:

OLMM Consulting Engineers
San Francisco • Oakland
(415) 882-9449 (510) 433-0828

Submittal Date:
March 23, 2017

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701 Sutter Street, 4th Floor
San Francisco, CA 94109

1305 Franklin Street, Suite 312
Oakland, CA 94612

TIER 1 SEISMIC EVALUATION
BUILDING 6, PIER 70
Port of San Francisco

INDEX

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San Francisco, CA 94109

1305 Franklin Street, Suite 312
Oakland, CA 94612

OLMM
CONSULTING
ENGINEERS

701 Sutter St., 4th Floor, San Francisco, CA 94109
Tel: (415) 882-9449

1305 Franklin Street, Suite 312, Oakland, CA 94612
Tel: (510) 433-0828

BUILDING 6, PIER 70 –TIER 1 SEISMIC EVALUATION

This report summarizes the findings and recommendation of Tier-1 seismic assessment of Building 6 at Pier 70 for the Port of San Francisco. The scope of this study was outlined in our proposal dated December 12, 2016. The structural assessment included a site visit and a review of available reports. The primary purpose of this assessment was to update the Tier 1 screening performed in 2008 per current ASCE 41-13 requirements and update any findings and recommendations. Existing structural or architectural drawings showing the member sizes, thickness, or connections were not available for our review.

1.0 BUILDING DESCRIPTION

Building 6 is a light steel industrial building originally constructed in 1941 (Reference 7). It is located in the Union Iron Works Historic District near Dogpatch (Figure 1, Appendix B). The building is approximately 72 ft. wide, 512 ft. long and 52 ft. tall. The building consists of riveted steel members forming a single story rectangular building with gabled roof. The exterior of the building is comprised of corrugated metal panels supported by steel girts and sag rods. Image 1 in Appendix A shows an exterior view of the building. The building sits partially on land and partially over water. The north half of the building extends into the bay.

The vertical load carrying structural system of the building comprises of metal panels at the roof supported on steel channel purlins. The steel channels are supported on steel trusses which in turn are supported by built-up steel I-columns, refer to Image 2 in Appendix A. The trusses and steel columns are spaced at roughly 30 ft. o.c. along the length of the building, forming 17 bays. Built-up I-beam running along the length of the warehouse on each longitudinal face likely supported gantry crane(s) in the past. The ground floor slab consists of 8" thick concrete slab (Image 5, Appendix A) spanning between concrete beams and timber piles with concrete pile caps (see COWI report, Reference 8). Timber piles are encased in concrete.

The lateral force resisting system for the building is comprised of corrugated metal roof diaphragm transferring seismic forces to steel braced frames on east and west faces of the building in the longitudinal direction. On the west face of the building, braces consist of single diagonals with two steel angles in every bay (see Image 3, Appendix A). On the east face of the building, braces are located in every other bay (see Image 4, Appendix A). The braces form an "X" pattern and are composed of 4 steel angles, two in each direction. The two steel angles comprising the diagonal brace are spaced roughly 1 ft. apart and connected by a steel plate approximately 3 ft. on center. The clear story above the crane girders has X-bracing in every bay on both east and west faces. In the transverse direction, the lateral force resisting system appears to consist of truss moment frames utilizing the gabled roof truss with an additional diagonal angle brace at the column support to create a moment connection to the columns (see Image 2, Appendix A).

2.0 BASIS OF ASSESSMENT

Seismic evaluation of the Building 6 was performed using the Tier 1 screening process prescribed in ASCE/SEI 41-13, American Society of Civil Engineers, *"Seismic Evaluation and Retrofit of Existing Buildings,"* 2013 (Reference 1). The evaluation was performed for Life Safety Performance objective under a Seismic Hazard Level of BSE-1E (20% in 50 years). The structural checklists from ASCE/SEI 41-13 for this building are attached in Appendix C. Since the building has what can be considered as truss moment frames in the transverse direction and braced frames in the longitudinal direction, checklists for both S1A building type corresponding to steel moment frames with flexible diaphragms, and S2A building type corresponding to steel braced frames with flexible diaphragms were completed.

3.0 SITE OBSERVATIONS

A site visit was performed on February 27th, 2017. The main purpose of the site visit was to visually review the readily accessible physical conditions of the building structure. Measurements, testing or explorations were not included in our scope.

In general, most structural steel columns, beams and trusses exhibit signs of corrosion. A limited number of braces were distorted or buckled (see Images 11 and 12, Appendix A). In several areas, a large portion of the corrugated metal panel forming the roof and exterior walls appear to be rusted through or completely missing (see Image 2, Appendix A). The concrete floor appears to be in good condition with only minor surface cracks at the slab's top surface. However, below deck inspection by COWI indicated several missing piles, areas of spalled concrete in the slab soffit, and signs of corroded reinforcement (Reference 8).

4.0 TIER 1 EVALUATION & KEY FINDINGS

Please refer to Appendix C for completed Tier 1 structural checklists 16.1, 16.4 and 16.5. Since drawings for the existing construction were not available, we made assumptions about member sizes and dimensions etc. to estimate building weight. Key observations from our evaluation are summarized below:

1. The building is located in an area of high seismicity and can be subjected to strong ground shaking in the future.
2. Based on the geotechnical report dated March 22, 2017 by Geotechnical Consultants, Inc. (GTC, Reference 9), potential for surface fault rupture at the site is negligible.

3. Based on the site specific geotechnical report by GTC, earthquake induced liquefaction and lateral spreading at Building 6 site is not anticipated to be pervasive. Moreover, since the building is supported on pile foundations, liquefaction should not have a major effect on the building's structural integrity. Missing timber piles, as identified by COWI, would require replacement, however.
4. As indicated in GTC's report, the building is situated in a low-lying area along the waterfront and may be susceptible to inundation due to sea level rise or earthquake induced tsunami. Evaluation of these impacts is beyond the scope of this evaluation.
5. The BSE-1E design ground shaking response spectra at the building site for the Tier-1 evaluation were obtained using USGS database, see Figure 4 in Appendix B. The building period was estimated to about 0.7 seconds. The spectral acceleration used in the evaluation corresponding to this building period is 0.9g.
6. Building 6 is rectangular in plan configuration without any discontinuous frames and does not appear to have any torsional or vertical irregularities. See Figure 5 in Appendix B for ground floor layout.
7. The connection of the roof corrugated metal diaphragm to the steel members appears to be through light gauge straps or wire pins. The metal siding also appears to be tied to building frame via straps or wires. This is not a compliant means of fastening to the building frame. See Images 8, 9 and 10, Appendix A.
8. The building's lateral force resisting system in the longitudinal (north-south) direction utilizes tension-only bracing. Based on our quick checks, and assuming L2x2x1/4 brace size, the braces appear to be adequate. However, since the rivet diameter is not known, the capacity of brace connections to develop tensile strength of the diagonal braces could not be confirmed. Column compactness could not be verified either because element thicknesses are not known.
9. The lateral system in the transverse (east-west) direction does not conform to systems defined in the current building codes. However, since the intent of the original design appears to have the columns and trusses perform as moment frames, the transverse direction behavior was reviewed using the S1A checklist 16.4 for steel moment frames with flexible diaphragms. Currently, using the quick checks as defined in ASCE 41-13, the building does not meet the drift and flexural capacity checks. In addition, since the member sizes are not available, it is unclear whether the columns are able to develop the strength of the truss braces connecting to it and whether the panel zone is adequate to resist seismic shear demands. A detailed Tier-3 evaluation of the structural system is recommended preceded by measurements and field investigations to identify section sizes and element dimensions and thicknesses.

5.0 RECOMMENDATIONS

Our evaluation was hampered by the fact that structural drawings for the existing construction are not available. We recommend field measurements of member sizes and thicknesses be surveyed so that a detailed quantitative evaluation can be performed. Notwithstanding the limited available information, the following recommendations are made on the basis of this evaluation:

1. Steel framing and metal decking shows signs of corrosion. Testing should be performed in selected areas to confirm that there is no significant loss of metal. Repair or replacement of those members with significant metal loss, if any, may need to be performed.
2. New metal decking should be installed on the roof in the areas where it is deteriorated or is missing. Metal decking should be positively fastened to steel members using approved fasteners.
3. Metal siding should be positively fastened to steel members using approved fasteners.
4. Based on Tier 1 screening checks it appears that the brace members in the longitudinal direction are adequate. However, their connections may not be adequate and further evaluation of the brace connections will be required. This evaluation would need to be preceded by field measurement of the members and rivet sizes. Detailed evaluations may indicate that strengthening of the column splices and brace-to-column connections is required. The strengthening may take the form of added field welds as shown in Figure 6, Appendix B.
5. The truss moment frames in building's transverse direction appear to be inadequate and will most likely need to be retrofitted. We recommend that a detailed seismic evaluation of the building be performed to identify the seismic deficiencies and to develop seismic retrofit measures. While the actual seismic retrofit measure would depend upon the deficiencies actually identified by detailed analyses and the future architectural and space requirements for the building, one potential seismic retrofit may be to create a steel moment frame by adding a steel beam at each bay as shown in Figure 7, Appendix B. This assumes that the built-up columns extending to the roof above the crane girder are adequate to resist the seismic forces by cantilever action
6. This report supersedes OLMM's 2008 report.
7. The recommendations outlined in COWI inspection report should be incorporated. In COWI's report, the option is given to provide new fiberglass sleeves at all locations where tops of timber piles are exposed or to institute a program of continued monitoring and address damage as it occurs. Both options should be priced. A continued monitoring program may be practical because even if new sleeves are installed, the remaining existing concrete sleeves will continue to deteriorate and there will need to be on-going inspections and maintenance regardless. An ongoing inspection and maintenance program can monitor the exposed piles for marin borer damage and repairs made when and if that becomes an issue. Issues such as slab soffit corrosion will also require continued monitoring.
8. The recommendations outlined in GTC Assessment letter should be incorporated. To install piles to replace missing piles, there maybe a need for added concrete grade beams. For pricing purposes,the addition of a grade beam with dimension of 30"x54" and 400 ft length with minimum reinforcement that is epoxied to the existing structure and slab soffit should account for added structural costs.

9. As noted in COWI's report, vehicular traffic or storage of any material should be barred from areas over, or within one bent of, missing piles until those piles are replaced.
10. The recommendations outlined in Carey & Co report dated May 2008 (reference 5) still appear to be relevant and should be incorporated.

6.0 LIMITATIONS AND DISCLAIMER

Our services have consisted of providing professional opinions, conclusions, and recommendations based on generally accepted structural engineering principles and practices existing at this time. This report includes a limited seismic assessment of the building. Items requiring action may exist that we have not been able to identify from this evaluation.

This report has been prepared for the exclusive use of the client, and is not for the benefit of, nor may be relied upon by, any other person or entity.

7.0 REFERENCES

1. ASCE (2013). Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers, ASCE/SEI 41-13, Reston VA.
2. "Rapid Structural Evaluation", 3/16/2007, Creegan & D'Angelo, San Francisco, CA.
3. "Structural Report", 12/20/2000, ABR Engineers, San Francisco, CA.
4. "Rapid Structural Assessment", 4/9/2003, Port of San Francisco, San Francisco, CA.
5. "Architectural Assessment Building 6 – Warehouse No 6", 5/2008, Carey & Co., San Francisco, CA.
6. "Pier 70, Building 6, Port of San Francisco Seismic Review", 5/2008, OLMM, Oakland, CA.
7. "National Registry Nomination documentation for the Union Iron Works Historic District", 2014, California State Office of Historic Preservation, Sacramento, CA.
8. "Pier 70 Building 6 Inspection Report R1", Feb. 2017, COWI Marine, Oakland, CA.
9. "Pier 70, Building 6 Geotechnical Assessment Letter Report", Geotechnical Consultants, Inc., March 22, 2017. San Francisco, CA.
10. Blakely, G.H. (1907). *Dimensions, weights and Properties of Special and Standard Structural Steel Shapes Manufactured by Bethlehem Steel Company - 1st Edition*. Philidelphia, PA: DANDO Printing and Publishing Co.

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APPENDIX A

IMAGES

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IMAGE 1: EXTERIOR VIEW OF BUILDING 6 LOOKING NORTH

IMAGE 2: Warehouse Interior Looking North

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IMAGE 3: Single Diagonal Brace at Every Bay on West Face

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IMAGE 4: "X"-Brace at Every Other Bay at East Face

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IMAGE 5: Slab Thickness Verification

IMAGE 6: Typical Brace to Column Connection

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IMAGE 7: Typical Column Splice Connection at Crane Track Support

IMAGE 8: Exterior Wall Metal Panel Connection to Steel Frame Using Wire Pins

IMAGE 9: Roofing Connection at Loading Dock

IMAGE 10: Main Roof Connection to Steel Members

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IMAGE 11: Buckled Roof Tie

IMAGE 12: Buckled Tension-Brace

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APPENDIX B

FIGURES

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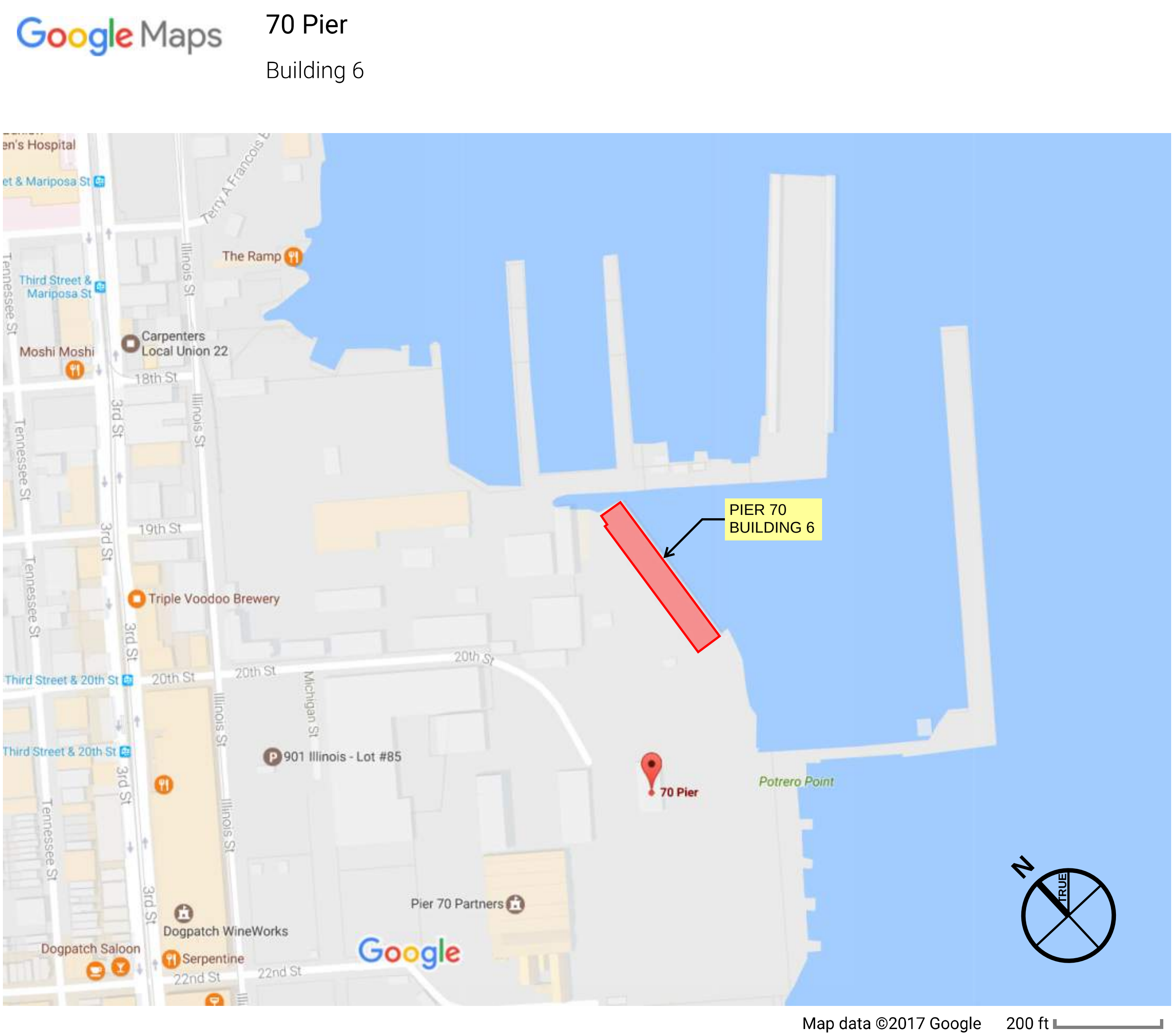


FIGURE 1: Building 6, Pier 70 Location on Map

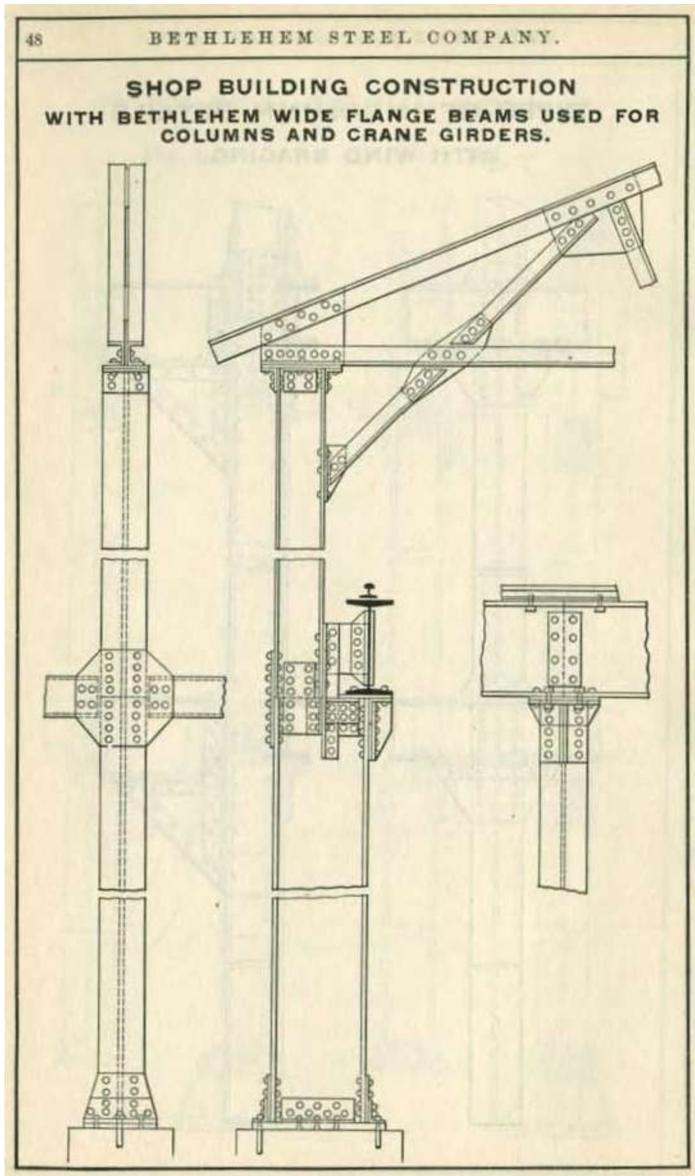


FIGURE 2: Construction Example Similar Detail to Building 6
Structural Steel Shapes, Bethlehem Steel Company
1st Edition, 1907

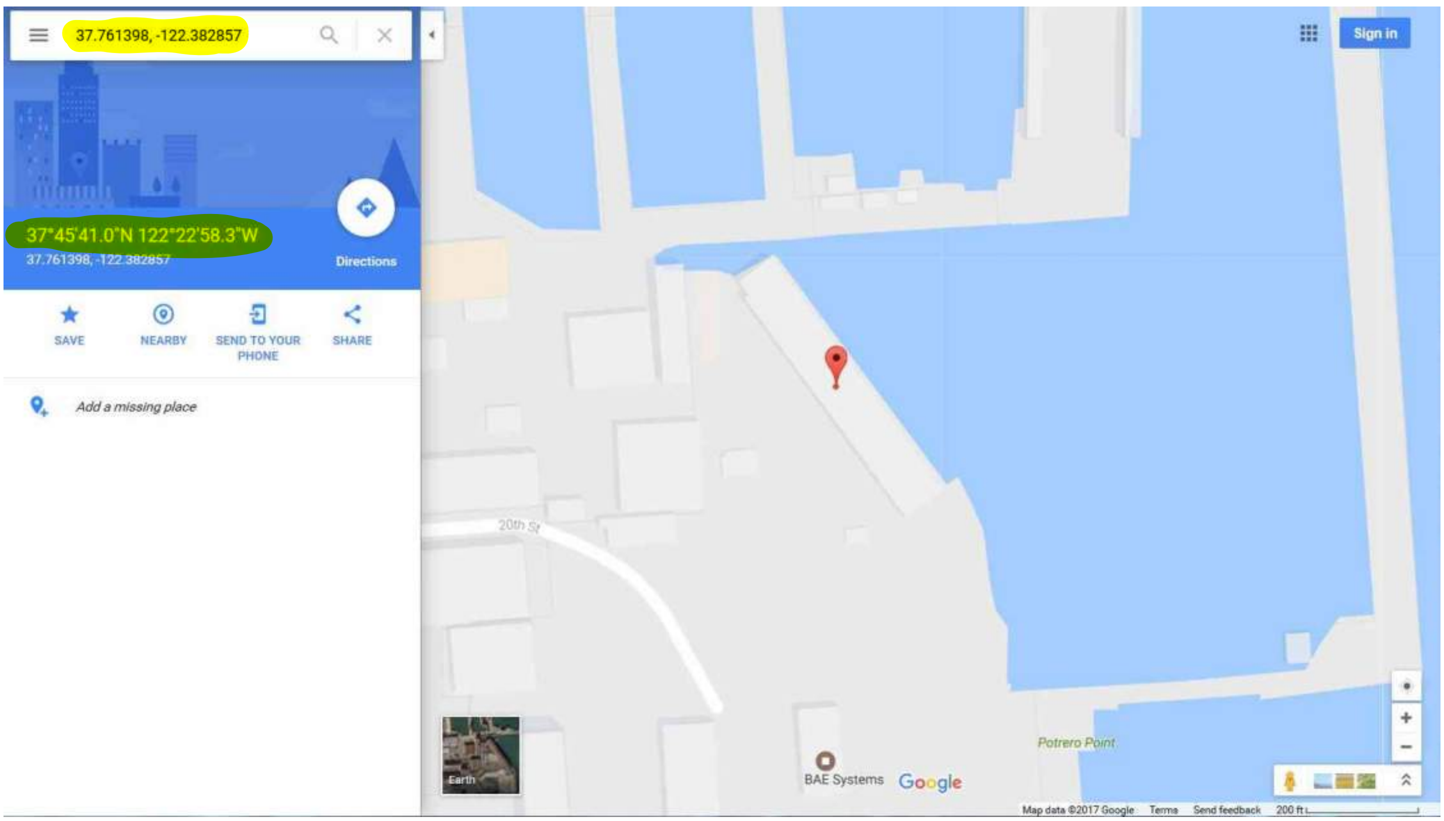
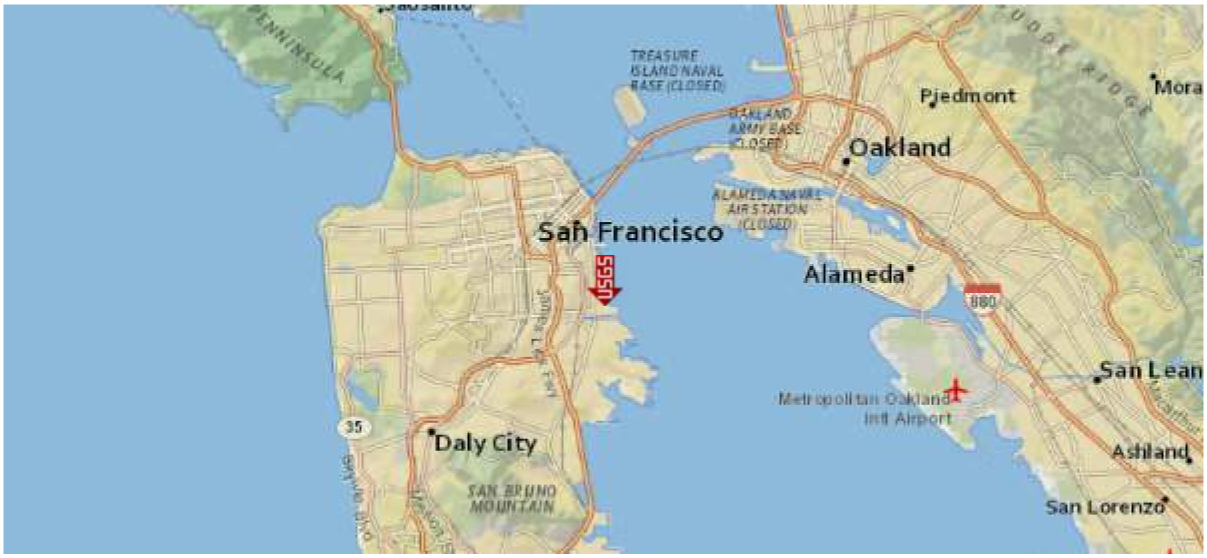


FIGURE 3: Geo-Coordinates for Building 6, Pier 70

Design Maps Summary Report

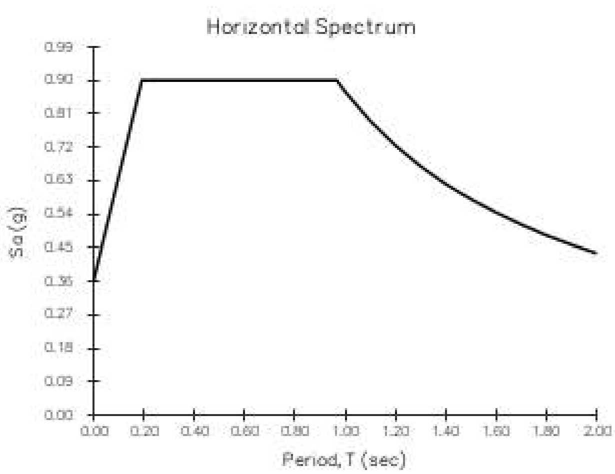
User–Specified Input

Report Title	20% in 50 years Wed February 22, 2017 17:29:00 UTC
Building Code Reference Document	ASCE 41-13 Retrofit Standard, BSE-1E (which utilizes USGS hazard data available in 2008)
Site Coordinates	37.7614°N, 122.38286°W
Site Soil Classification	Site Class E – "Soft Clay Soil"



USGS–Provided Output

$\mathbf{S_{S,20/50}}$	0.854 g	$\mathbf{S_{XS,BSE\text{-}1E}}$	0.900 g
$\mathbf{S_{1,20/50}}$	0.320 g	$\mathbf{S_{X1,BSE\text{-}1E}}$	0.870 g



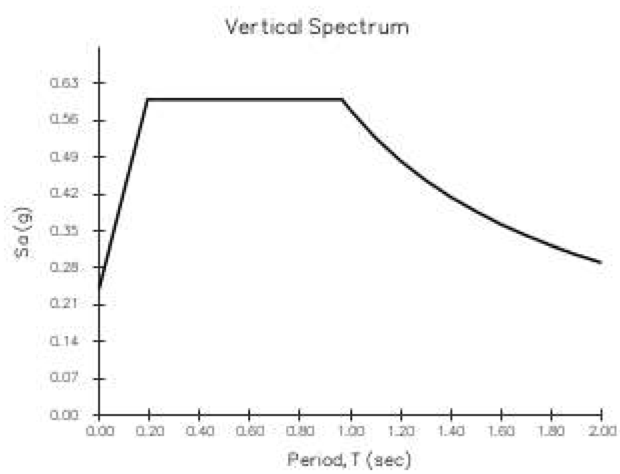
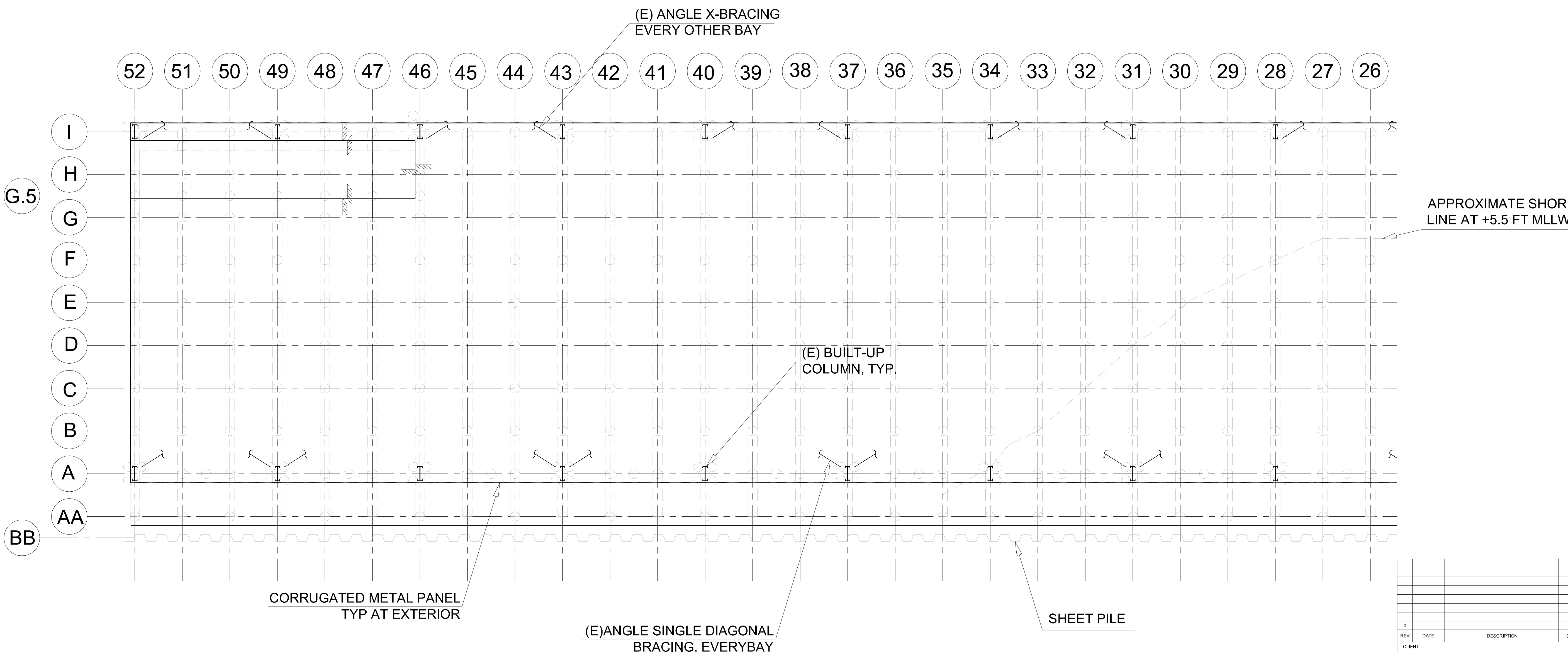


FIGURE 4: Seismic Coefficients for BSE-1E at Bldg 6, Pier 70



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SK-1
GENERAL PLAN 26-52
SCALE: NTS

FIGURE 5: GROUND FLOOR PLAN

REV	DATE	DESCRIPTION	BY	CHK
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OLMM CONSULTING ENGINEERS
701 Sutter St. 4th Floor
San Francisco CA, 94109
Phone: 415.882.9449

COWI Marine North America
A Business Unit of COWI North America, Inc.
1300 Clay Street, 7th Floor Oakland, CA 94612
Tel.: 510.839.8972 Fax: 510.839.9715
Website: www.cowi-na.com

PROJECT TITLE
PIER 70 BUILDING 6 INSPECTION

CONFORMED SET

DRAWING TITLE
OVERALL SITE PLAN

SCALE	DESIGNED	APPROVED
AS SHOWN	SRNM	-
DRAWN	CHECKED	DATE
SRNM	-	2-14-2017

JOB NO.	DRAWING NO.	REV.
A077120	SK-1	0

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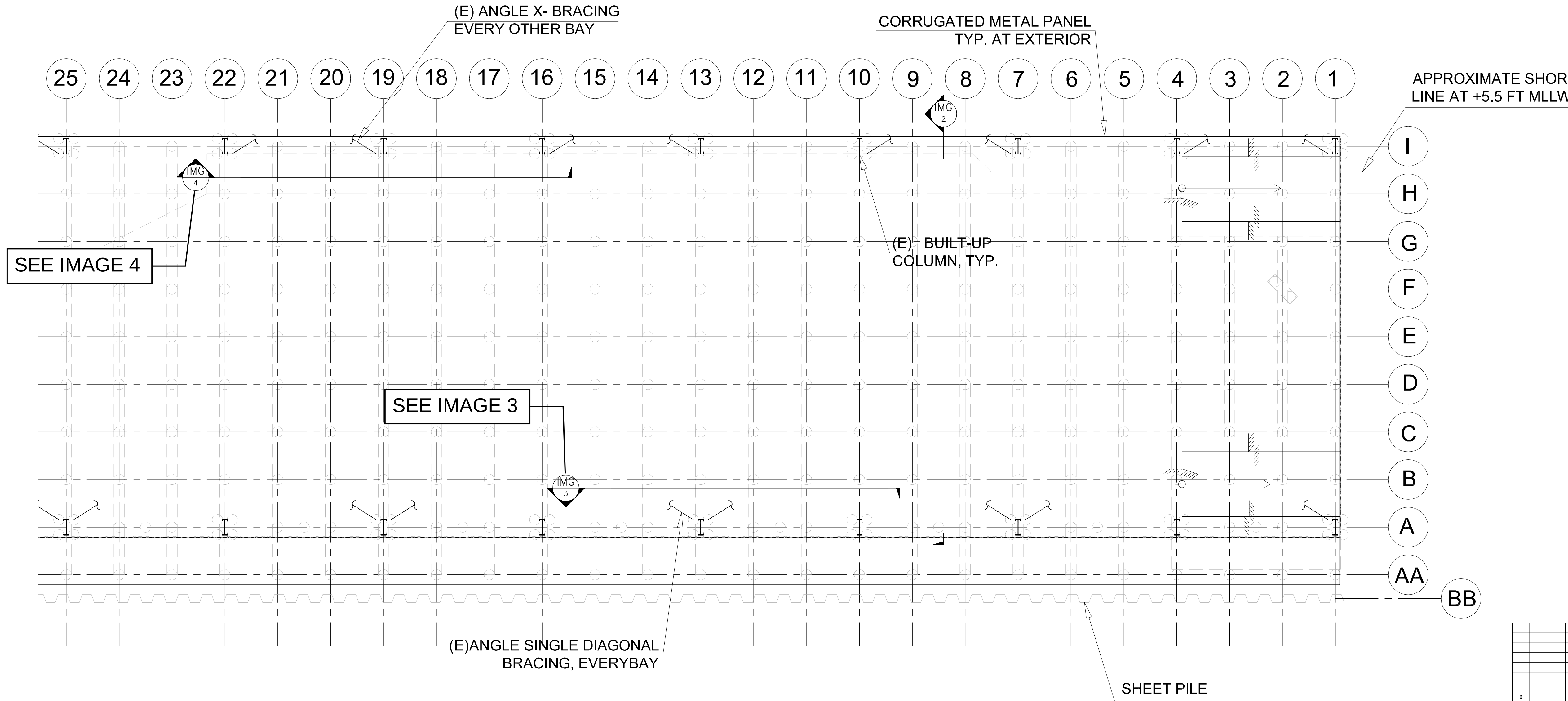

1 SK-2 GENERAL PLAN 1-25

SCALE: NTS

TRUE NORTH

REFERENCE NORTH

FIGURE 5: GROUND FLOOR PLAN... CONTINUE

REV	DATE	DESCRIPTION	BY	CHK
0				

CLIENT

OLMM CONSULTING ENGINEERS
701 Sutter St. 4th Floor
San Francisco CA, 94109
Phone: 415.882.9449

COWI Marine North America
A Business Unit of COWI North America, Inc.
1300 Clay Street, 7th Floor Oakland, CA 94612
Tel.: 510.839.8972 Fax: 510.839.9715
Website: www.cowi-na.com

PROJECT TITLE
PIER 70 BUILDING 6 INSPECTION

CONFORMED SET

DRAWING TITLE
OVERALL SITE PLAN

SCALE	DESIGNED	APPROVED
AS SHOWN	SRNM	-
DRAWN	CHECKED	DATE
SRNM	-	2-14-2017
JOB NO.	DRAWING NO.	REV.
A077120	SK-2	0

FIGURE 6: Strengthening of Brace Connection in Longitudinal Direction

FIGURE 7: Retrofit Solution in Transverse Direction

APPENDIX C

STRUCTURAL CHECKLISTS

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San Francisco, CA 94109

1305 Franklin Street, Suite 312
Oakland, CA 94612

Project: BUILDING 6 Location: PIER 70, SAN FRANCISCO

Completed by: HLK Date: 3/17/2017

TIER 1 CHECKLISTS

16.1 BASIC CHECKLIST

Very Low Seismicity

Structural Components

C NC N/A U LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

C NC N/A U WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)

Project: BUILDING 6 Location: PIER 70, SAN FRANCISCO

Completed by: HLK Date: 3/17/2017

16.1.2LS LIFE SAFETY BASIC CONFIGURATION CHECKLIST

Low Seismicity

Building System

General

C **NC** N/A U LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

C NC N/A U ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1a, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)

C NC **N/A** U MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

Building Configuration

C NC **N/A** U WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)

C NC **N/A** U SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)

C NC N/A U VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)

C NC **N/A** U GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)

C NC **N/A** U MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)

C NC N/A U TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.

Geologic Site Hazards

C NC N/A U LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)

C NC **N/A** U SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)

C NC N/A U SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

Foundation Configuration

C NC N/A U OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

C NC N/A U TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

Project: BUILDING 6 Location: PIER 70, SAN FRANCISCO

Completed by: HLK Date: 3/17/2017

16.4LS LIFE SAFETY STRUCTURAL CHECKLIST FOR BUILDING TYPES S1: STEEL MOMENT FRAMES WITH STIFF DIAPHRAGMS AND S1A: STEEL MOMENT FRAMES WITH FLEXIBLE DIAPHRAGMS

Low Seismicity

Seismic-Force-Resisting System

C NC N/A U DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.5.3.1, is less than 0.025. (Commentary: Sec. A.3.1.3.1. Tier 2: Sec. 5.5.2.1.2)

C NC N/A U COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3)

C NC N/A U FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.5.3.9, is less than F_y. Columns need not be checked if the strong column–weak beam checklist item is compliant. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.2)

Connections

C NC N/A U TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2)

C NC N/A U STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1)

Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.

Seismic-Force-Resisting System

C NC N/A U REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1)

C NC N/A U INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1)

C NC N/A U MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members based on the specified minimum yield stress of steel. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1). Note: more restrictive requirements for High Seismicity.

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

Seismic-Force-Resisting System

C NC N/A U MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel per AISC 341, Section A3.2. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1)

C NC N/A U PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column. (Commentary: Sec. A.3.1.3.5. Tier 2: Sec. 5.5.2.2.2)

C NC N/A U COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web. (Commentary: Sec. A.3.1.3.6. Tier 2: Sec. 5.5.2.2.3)

C NC N/A U STRONG COLUMN—WEAK BEAM: The percentage of strong column–weak beam joints in each story of each line of moment frames is greater than 50%. (Commentary: Sec. A.3.1.3.7. Tier 2: Sec. 5.5.2.1.5)

C NC N/A U COMPACT MEMBERS: All frame elements meet section requirements set forth by AISC 341 Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.4)

Diaphragms (Stiff or Flexible)

C NC N/A U OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3)

Flexible Diaphragms

C NC N/A U CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)

C NC N/A U STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)

C NC N/A U SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)

C NC N/A U DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)

C NC N/A U OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)

Project: BUILDING 6 Location: PIER 70, SAN FRANCISCO

Completed by: HLK Date: 3/17/2017

16.5LS LIFE SAFETY STRUCTURAL CHECKLIST FOR BUILDING TYPES S2: STEEL BRACED FRAMES WITH STIFF DIAPHRAGMS AND S2A: STEEL BRACED FRAMES WITH FLEXIBLE DIAPHRAGMS

Low Seismicity

Seismic-Force-Resisting System

C NC N/A U COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3)

C NC N/A U BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.5.3.4, is less than $0.50F_y$. (Commentary: Sec. A.3.3.1.2. Tier 2: Sec. 5.5.4.1)

Connections

C NC N/A U TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2)

C NC N/A U STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1)

Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.

Seismic-Force-Resisting System

C NC N/A U REDUNDANCY: The number of lines of braced frames in each principal direction is greater than or equal to 2. The number of braced bays in each line is greater than 2. (Commentary: Sec. A.3.3.1.1. Tier 2: Sec. 5.5.1.1)

C NC N/A U CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4)

C NC N/A U COMPACT MEMBERS: All brace elements meet compact section requirements set forth by AISC 360, Table B4.1. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4)

C NC N/A U K-BRACING: The bracing system does not include K-braced bays. (Commentary: Sec. A.3.3.2.1. Tier 2: Sec. 5.5.4.6)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

Seismic-Force-Resisting System

C NC N/A U COLUMN SPLICES: All column splice details located in braced frames develop 50% of the tensile strength of the column. (Commentary: Sec. A.3.3.1.3. Tier 2: Sec. 5.5.4.2)

C NC N/A U SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have Kl/r ratios less than 200. (Commentary: Sec. A.3.3.1.4. Tier 2: Sec. 5.5.4.3)

C NC N/A U CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4)

C NC N/A U COMPACT MEMBERS: All brace elements meet section requirements set forth by AISC 341, Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4)

C NC N/A U CHEVRON BRACING: Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs. (Commentary: Sec. A.3.3.2.3. Tier 2: Sec. 5.5.4.6)

C NC N/A U CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces shall frame into the beam–column joints concentrically. (Commentary: Sec. A.3.3.2.4. Tier 2: Sec. 5.5.4.8)

Diaphragms (Stiff or Flexible)

C NC N/A U OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 25% of the frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3)

Flexible Diaphragms

C NC N/A U CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)

C NC N/A U STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)

C NC N/A U SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)

C NC N/A U DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)

C NC N/A U OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)