UCSF

Rating form
completed by:

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Text in green is to be part of UCSF building database and may be part of UCOP database

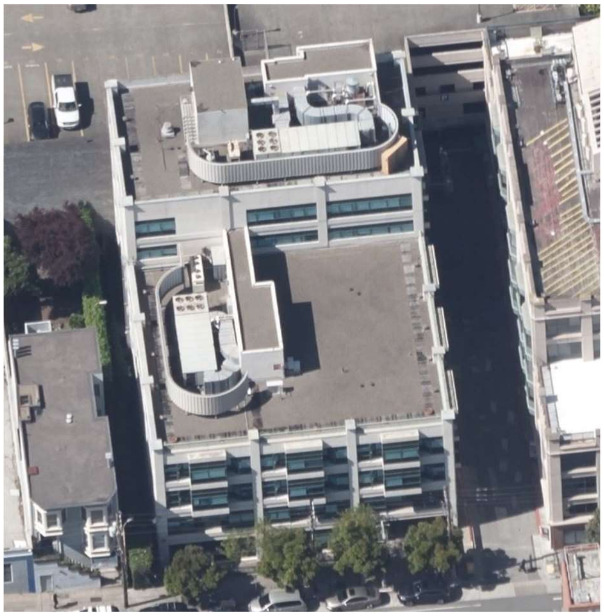
UCSF building seismic ratings

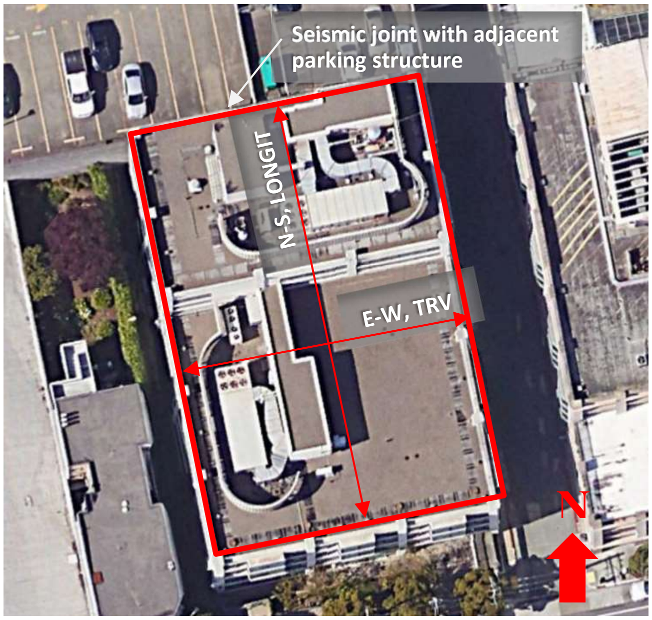
2330 Post Street, MOB 1

JOSEPH RICHARD MAFFEI
No. 3694

CAAN #2020
2330 Post Street, San Francisco, CA 94115
UCSF Campus: Mt. Zion

DATE: 2020-06-26





Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	V	Based on drawing review and Tier 1 evaluation[1]
Rating basis	Tier 1	ASCE 41-17
Date of rating	2019	
Recommended UCSF priority category for retrofit	Priority B	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application for modification
Ballpark total construction cost to retrofit to IV rating[2]	Very High (> $400/sf)	See recommendations on further evaluation and retrofit.
Is 2018-2019 rating required by UCOP?	Yes	Building previously rated IV but does not have a fully documented previous review
Further evaluation recommended?	Tier 2	Further evaluation of steel moment frame connections and strong-column/weak-beam is needed

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[1] The evaluations at UCSF translate the Tier 1 evaluation to a Seismic Performance Level rating using professional judgment discussed among the Seismic Review Committee. Non-compliant items in the Tier 1 evaluation do not automatically put a building into a particular rating category, but such items are evaluated along with the combination of building features and potential deficiencies, focused on the potential for collapse or serious damage to the gravity supporting structure that may threaten occupant safety.

[2] Per Section 3.A.4.i of the Seismic Program Guidebook, the cost includes all construction cost necessitated by the seismic retrofit, including restoration of finishes and any triggered work on utilities or accessibility. It does not include soft costs such as design fees or campus costs. The cost is in 2019 dollars.

Building information used in this evaluation

- Structural drawings by OLMM Structural Design, "Western Development Group Medical Building at 2330 Post Street," 1993-12-08 (16 sheets)
- Architectural drawing set by ESS Architecture and their consultants, "Western Development Group Medical Building at 2330 Post Street," 1993-12-13 (92 sheets)
- Structural steel erection drawings by Gayle Manufacturing, "Webcor Builders Inc. Post Street Medical Building," April 1994 (9 sheets)

Additional building information known to exist

- Architectural and M/E/FP drawings by ESS Architects and Ted Jacob Engineering Group, "UCSF Mount Zion Hospital and Medical Center 2330 Post Street Medical Office Building 1st, 2nd & 3rd Floor Tenant Improvements," 1995-02-14 (123 sheets)
- Structural steel fabrication drawings by Gayle Manufacturing (~65 sheets)

Scope for completing this form

We reviewed structural drawings for original construction and carried out an ASCE 41-17 Tier 1 evaluation. We walked through the building on 2019-11-05 to confirm that the building generally matches the original drawings and to check for non-structural life-safety issues.

Brief description of structure

The building has a floor area of approximately 50,500 square feet. It is approximately 85' x 125' in plan with an overall height of 72' to the top of the parapet. It is 6 stories with the lowest level approximately 4 to 7 feet below the Post Street sidewalk to the south. It was designed in 1993 by OLMM Structural Design and ESS Architecture. Construction of the shell was completed in 1994 and tenant improvements for floors 1, 2 and 3 were constructed in 1995. The building is separated by a seismic joint from the adjacent parking garage to the north.

Identification of levels: The lowest level is the 1st floor or ground floor. Floors 2 through 4 extend fully over the building's plan area. Floors 5 and 6 extend over the northern 40% of the building's footprint only.

Structural system for vertical (gravity) load: Above the 1st floor, the typical floor system is 3" composite metal deck with 2½" of normal weight concrete fill supported on W14 beams at 9'-9" on center spanning north-south to W24 girders at 25' on center spanning to W14 columns at 27'-6" on center. The roof framing system is similar. The 1st floor is a 5" thick concrete slab on grade.

Foundation system: The building's foundation is a grid of concrete strip footings. 4'-6" wide by 3'-3" deep, running in both the longitudinal and transverse directions and centered on each column line. The footings are reinforced with 10 - #11 bars top and bottom and #5 ties at 6" on center.

Structural system for lateral forces: There is a full-height welded steel moment frame at the perimeter on all 4 sides of the building. There is a 5th full-height moment frame oriented east-west at grid line 4, which is the south façade at the 5th and 6th floors. See figures 1 and 2.

Brief description of seismic deficiencies and expected seismic performance including mechanism of nonlinear response and structural behavior modes

Identified seismic deficiencies of the building include the following:

Structural feature or potential deficiency	Finding/notes
Welded moment connections	Full-penetration flange welds at steel frame moment connections are susceptible to early fracture because design and specification (and probably construction) of the welds predates the October 1994 UBC Emergency Provisions for steel moment-resisting frames (i.e. pre-Northridge.) The welds have the potential to fracture when subjected to earthquake deformation demand.

Structural feature or potential deficiency	Finding/notes
Welded column splices	Tier 1 assumes no deficiency if both webs and flanges and column splices are connected in any manner. However, more detailed studies of moment-frame buildings have shown column splices to be a potential vulnerability, in particular those using partial penetration welds as is the case with this structure. For a typical case of welds sized to ½ of flange and web thickness, research has shown that the capacity of welds may be as low as 10% of the column section capacity. Each column in the building is spliced above the second floor, and each column that extends to the higher portion of the building is spliced above the 5th floor.
Moment frame stiffness	Quick Check inter-story drift ratios are 4% at the 3rd floor in the transverse direction, and 4% at the 1st and 2nd floors in the longitudinal direction. They are also between 3% and 4% in both directions at the 5th floor. This indicates that the building's moment frames may be too flexible to meet the required Seismic Performance Level of IV. Details show thick base plates and anchor bolts to heavily reinforced grade beams, so we assume base fixity at the ground floor in the quick check procedures.
Moment frame column strength	Quick Checks indicate that moment frame columns and panel zones have insufficient yield strength to protect against a story mechanism. **Strong column – weak beam checks:** The ratio of column to beam moment capacity ranged from 1.29 (best) to 0.11 (worst) for those that we checked, with the ratio exceeding 1.0 at only 2 locations. The low ratios are due to high axial loads in the columns, using ASCE 41 design forces without an M factor, as we interpret is appropriate for a Tier 1 analysis. This indicates the potential for hinging to occur in the columns, precipitating a story mechanism that can lead to collapse. The lowest ratios were at the 2nd floor level, ratios ranged from 0.11 to 0.70. We do not include the ratios for connections at the roof as these would not affect a story mechanism. **Panel zone capacity checks:** At all 32 of the moment connections that were checked, column panel zone shear capacity does not meet the Tier 1 requirement that it exceed 80% of the maximum demand from the adjoining beams. A factor in this is that the original moment frame design used A36 steel for beams and Grade 50 steel for columns For buildings 1990 and later ASCE 41-17 Tier 1 specifies a default yield strength of 49 ksi for A36, because it became common in the 1990s for A36 beams to be dual-certified and have higher yield strength, without engineers accounting for this in design. (The default yield strength for Tier 1 for A36 prior to 1990 is 37 ksi.) The actual potential for a story mechanism will be higher than indicated by ASCE-41 procedures, as described in the 1999 SEAOC Blue Book. Conversely, the presence of gravity columns and the out-of-plane action of moment-frame columns (with shear-tab beam connections out-of-plane) can reduce the potential for story mechanism. We have evaluated the benefit of gravity columns for similar structures and found that they are likely to prevent or mitigate a story mechanism, at least prior to widespread connection fractures occurring.

Structural feature or potential deficiency	Finding/notes
Site class D spectral shape	Per footnote 4, the earthquake demands are based on an F_v factor that does not include the requirements of Section 11.4.8-3 of ASCE 7-16. If such requirements were to be included, for this building with T=1.02 seconds (using ASCE 41-17 equation 4-4 based on building height), demands would increase by a factor of about 1.5. (See Figure 5.) The Quick Check of inter-story drift ratios would then be noncompliant for all stories and directions, with values up to 6% at the 5th floor. Also, the Quick Check for column flexural stress would be noncompliant in both the transverse and longitudinal directions at the 6th floor (38% over) and in the transverse direction at the 3rd floor (15% over).

Structural deficiency	Affects rating?	Structural deficiency	Affects rating?
Lateral system stress check (wall shear, column shear or flexure, or brace axial as applicable)	N	Openings at shear walls (concrete or masonry)	N
Load path	N	Liquefaction	N
Adjacent buildings	N	Slope failure	N
Weak story	N	Surface fault rupture	N
Soft story	N	Masonry or concrete wall anchorage at flexible diaphragm	N
Geometry (vertical irregularities)	N	URM wall height-to-thickness ratio	N
Torsion	N	URM parapets or cornices	N
Mass – vertical irregularity	N	URM chimney	N
Cripple walls	N	Heavy partitions braced by ceilings	N
Wood sills (bolting)	N	Appendages	N
Diaphragm continuity	N		

Summary of review of non-structural life-safety concerns, including at exit routes [3]

The anchorage of and fuel connections to 2 rooftop natural gas-fueled boilers should be reviewed and retrofitted if needed. There is an earthquake-activated automatic gas shutoff at the main gas service at ground level adjacent to Post Street. However, this rooftop equipment should be anchored. Note that rooftop seismic motions will be amplified.

UCOP non-structural checklist item	Life safety hazard?	UCOP non-structural checklist item	Life safety hazard?
Heavy ceilings, feature or ornamentation above large lecture halls, auditoriums, lobbies or other areas where large numbers of people congregate	None observed	Unrestrained hazardous materials storage	None observed
Heavy masonry or stone veneer above exit ways and public access areas [Or older or vulnerable precast concrete cladding]	None observed	Masonry chimneys	None observed
Unbraced masonry parapets, cornices or other ornamentation above exit ways and public access areas	None observed	Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.	Investigate rooftop boilers

Discussion of rating

We rate the building V primarily because the welded connections are vulnerable to fracture. The welding specifications for the moment frames that pre-date the 1994 UBC Emergency Provisions indicate that this building

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[3] For these Tier 1 evaluations, we do not visit all spaces of the building; we rely on campus staff to report to us their understanding of the type and location of potential non-structural hazards.

requires further study to confirm its rating and to identify retrofit steps if additional analysis shows that they are needed to improve the rating.

A second reason for the V rating is that there may be insufficient moment column yield strength if the ASTM A36 moment frame beams and girders have a higher yield strength than was assumed in the original design. This is a known issue with designs using A36 steel in this this time period. Insufficient column yield strength increases the likelihood of story mechanisms that can lead to total or partial building collapse.

Recommendations for further evaluation or retrofit

Further evaluation recommended?	Tier 3 NLRHA, could be done to see if there is a possibility the performance could be IV, even with the deficiencies, or could be done at time of planned retrofit to determine extent of measures
Likelihood of showing better rating	Unlikely [✓ Possible] Good chance
Likelihood of showing worse rating	[✓ Unlikely] Possible Good chance
Evaluation needed to clarify the necessary retrofit scope?	Yes, it could be used to determine how much connection retrofitting or other strengthening is needed to meet IV.
Discussion of priority assignment	We suggest Priority B because retrofit would be disruptive and best accomplished along with remodeling or other work.

Peer review comments on rating

The structural members of the UCSF Seismic Review Committee (Lizundia, Moore, Phipps, Thiel) reviewed the presentation of this evaluation on 18 November 2019, and they reviewed this report. The SRC agrees that a Seismic Performance Level Rating of V is appropriate.

Additional building data	Entry	Notes
Latitude	37.784567	
Longitude	- 122.440416	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	6	
Number of stories (basements) below lowest perimeter grade	0	1st floor elevation is 4 to 7 feet below Post Street sidewalk elevation.
Building occupiable area (OGSF)	50,491	From UCOP spreadsheet
Risk Category per 2016 CBC 1604.5	II	
Building structural height, h_n	68 ft	Structural height defined per ASCE 7-16 Section 11.2
Estimated fundamental period	1.02 sec	Estimated using ASCE 41-17 equation 4-4
Site data		
975 yr hazard parameters S_s, S_1	1.435, 0.559	
Site class	D	
Site class basis	Study	UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Site parameters F_a, F_v	1.0, 1.741[4]	
Ground motion parameters S_{cs}, S_{c1}	1.435, 0.973	
S_a at building period	0.951	
Site V_{s30}	308 m/s	

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[4] F_V factor used does not include the requirements of Section 11.4.8-3 of ASCE 7-16 that are applicable to Site Class D, and which per Exception 2 would result in an effective F_V factor 1.5 times larger. At the UCSF Mt. Zion campus this affects structures with $T > 0.68$ seconds.

V_{s30} basis	Estimated	UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Liquefaction potential	No	
Liquefaction assessment basis	Study	UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Landslide potential	No	
Landslide assessment basis	Study	UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Active fault-rupture identified at site?	No	
Fault rupture assessment basis	Study	UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Site-specific ground motion study?	No	
Applicable code		
Applicable code or approx. date of original construction	Built: 1994 Code: 1992 SFBC / 1991 UBC	Code identified on Olmm sheet S1.1 & ESS sheet 0.0
Applicable code for partial retrofit	None	No partial retrofit known
Applicable code for full retrofit	None	No full retrofit known
Model building data		
Model building type	S1 Steel moment frame	
FEMA P-154 score	0.8	
Previous ratings		
Most recent rating	IV	2013 report
Date of most recent rating	2013-10-07	Basis: qualitative assessment based on document review
2nd most recent rating	Fair	In spreadsheet. Basis for rating is unknown
Date of 2nd most recent rating	-	Rating date is unknown
3rd most recent rating	-	
Date of 3rd most recent rating	-	
Appendices		
ASCE 41 Tier 1 checklist included here?	Yes	Refer to attached checklist file

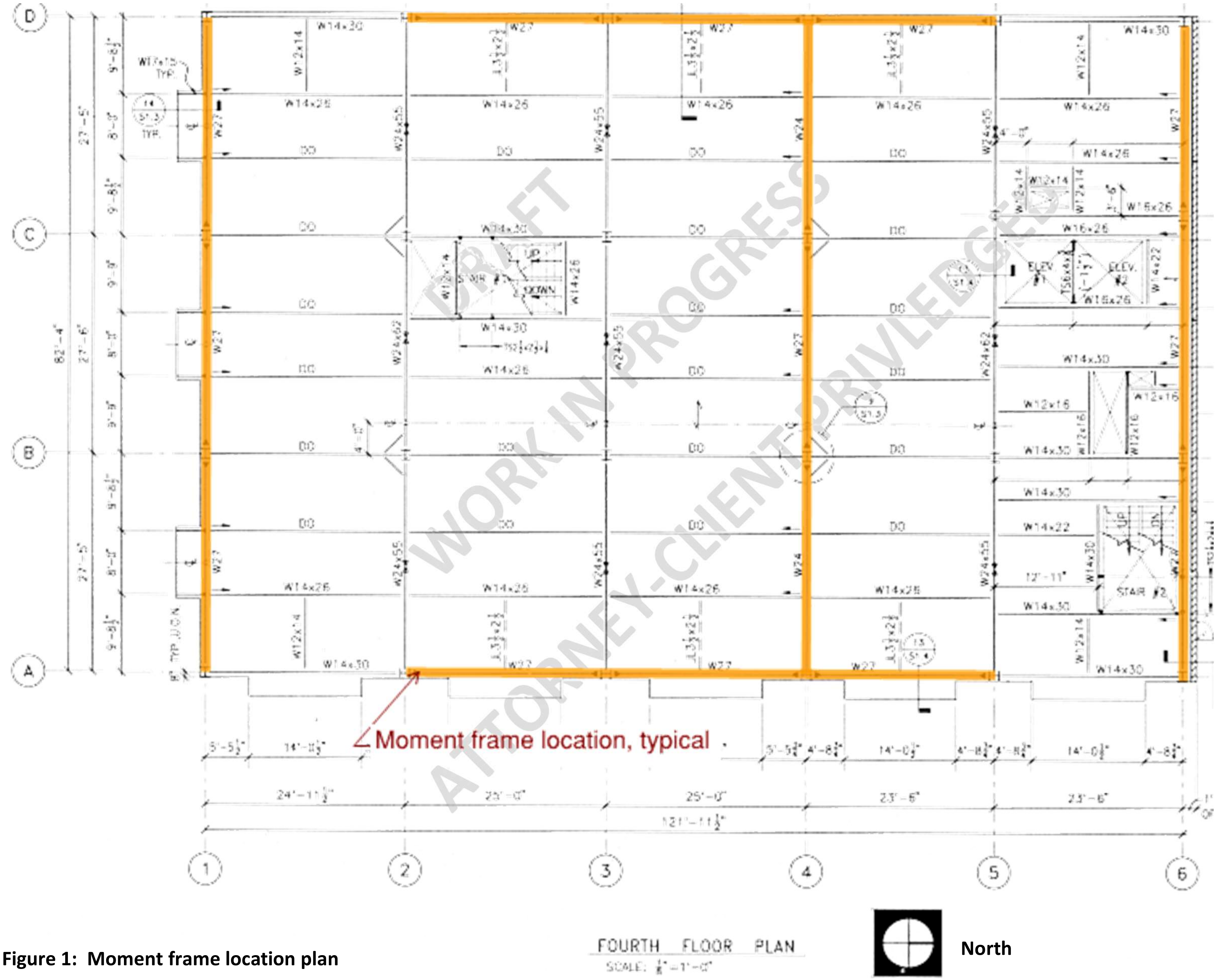


Figure 1: Moment frame location plan

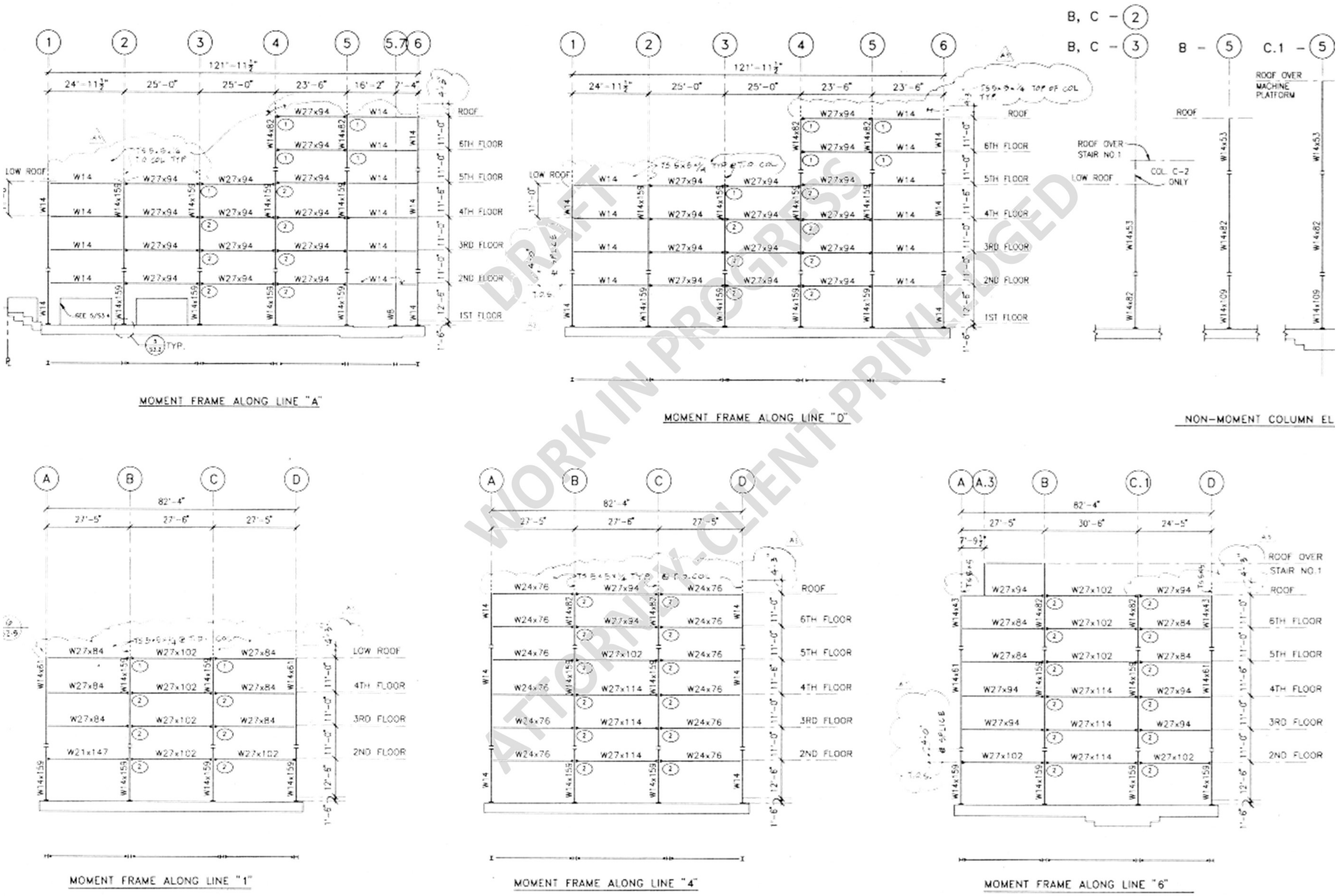


Figure 2: Moment frame elevations (ref. sheet S3.1)

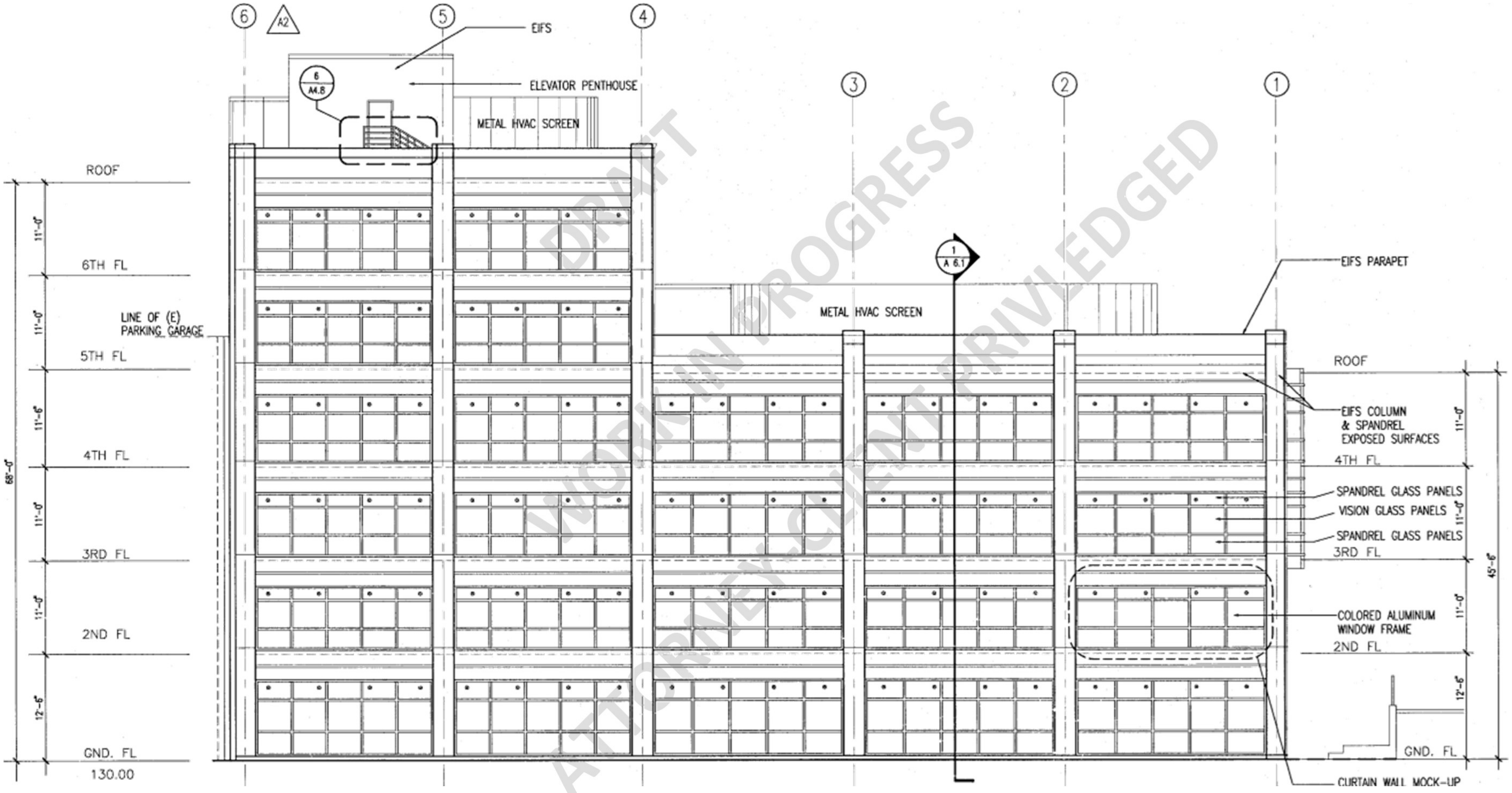
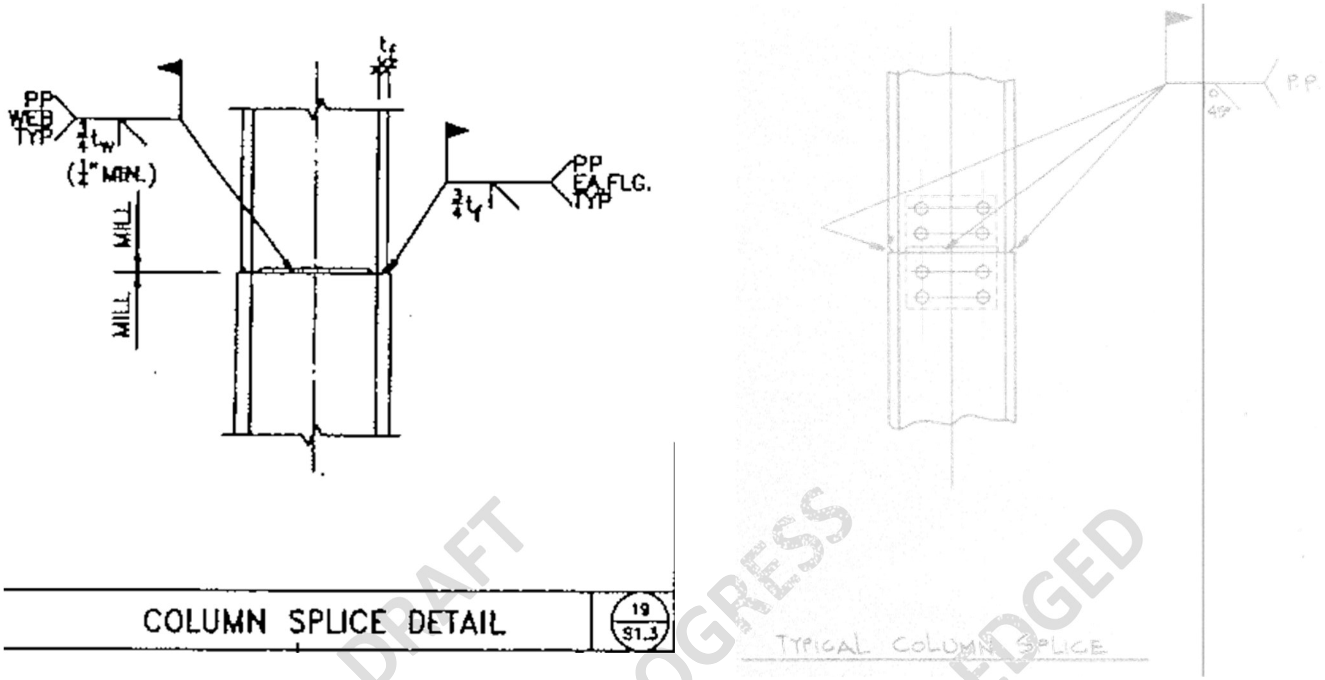


Figure 3: West elevation (ref. sheet A3.4)



a. Structural drawings **b. Steel erection drawings**

Figure 4: Column splice detail

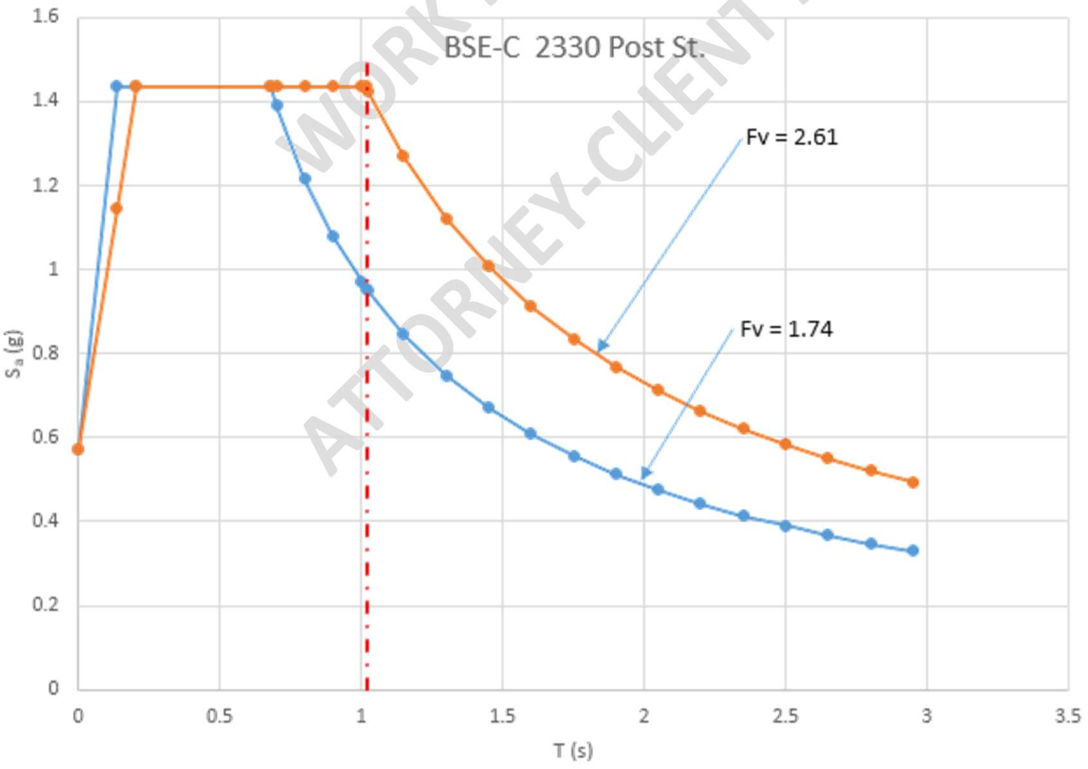


Figure 5: Response spectra

South and east façades

Post St pedestrian entrance

Retaining wall at west property line

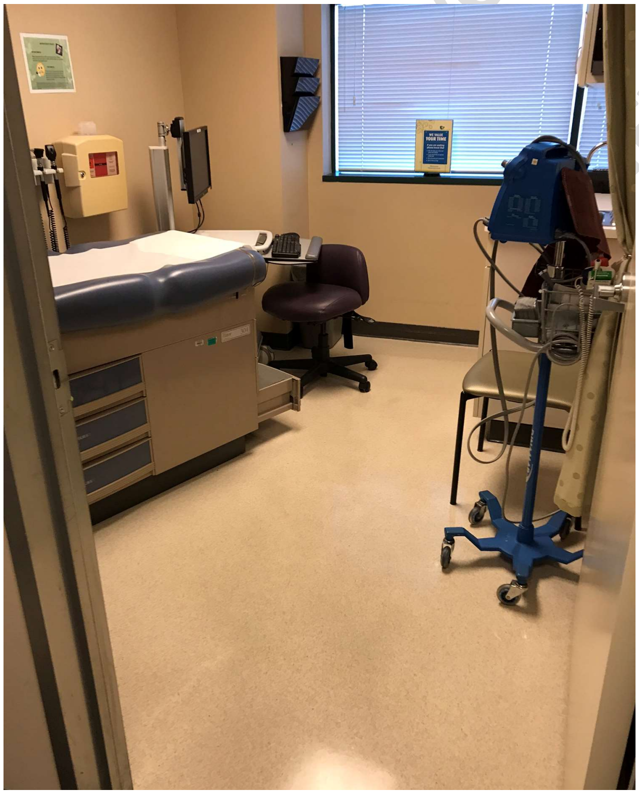
Lobby

Out-of-plane wall anchor at north CMU wall

Typical hallway

Threshold at entrance from parking garage
(seismic separation joint cover at left)

Rooftop boiler for domestic hot water



Typical examination room

Natural gas line to HVAC boiler

UC Campus:	San Francisco		Date:	11/06/2019	
Building CAAN:	2020	Auxiliary CAAN:	By Firm:	MSE	
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ASCE 41-17
Collapse Prevention Basic Configuration Checklist

LOW SEISMICITY

BUILDING SYSTEMS - *GENERAL*

	Description
C NC N/A U ● ○ ○ ○	LOAD PATH: The structure contains 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) **Comments:**
C NC N/A U ● ○ ○ ○	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2) **Comments:** Parking structure height is ~ 46 ft per north elevation on sht. A3.3. Seismic gap is approximately 10” based on field observation. _10” > .015·46·12” = 8.3”_
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) **Comments:**

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

	Description
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) **Comments:**
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) **Comments:**

Note: **C =** Compliant **NC =** Noncompliant **N/A =** Not Applicable **U =** Unknown

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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) **Comments:**
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) **Comments:** There is approximately a 67% reduction in the N-S moment frame horizontal dimension at the 5th floor because of the building setback
C NC N/A U ○ ● ○	MASS: There is no change in effective mass of 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) **Comments:** The change is > 50% at the 5th floor because of the building setback
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) **Comments:** In E-W direction at the lower stories the CR is at ~ (75'+122')/3 = 65' from Line 1. CM is ~ 61' from Line 1. 4'/122' = 3.3% < 20%

MODERATE SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN ADDITION TO THE ITEMS FOR LOW SEISMICITY)

GEOLOGIC SITE HAZARD

	Description
C NC N/A U ● ○ ○ ○	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2m) under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1) **Comments:** Per Egan report
C NC N/A U ● ○ ○ ○	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is 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) **Comments:** Per Egan report

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Collapse Prevention Basic Configuration Checklist

MODERATE SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN ADDITION TO THE ITEMS FOR LOW SEISMICITY)

GEOLOGIC SITE HAZARD

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) **Comments:** per Egan report

HIGH SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN ADDITION TO THE ITEMS FOR MODERATE SEISMICITY)

FOUNDATION CONFIGURATION

	Description
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) **Comments:** 82.3’/67.0’ = 1.23 > 0.6·0.951 = 0.571
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) **Comments:**

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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type S1-S1A

LOW SEISMICITY

SEISMIC-FORCE-RESISTING SYSTEM

	Description
C NC N/A U ● ○ ○ ○	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1) **Comments:** There are 2 moment frame lines in the N-S direction and 3 in the E-W direction.
C NC N/A U ○ ● ○ ○	DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.4.3.1, is less than 0.030. (Commentary: Sec. A.3.1.3.1. Tier 2: Sec. 5.5.2.1.2) **Comments:** Drift ratio is 0.04 at 3rd floor in the transverse direction and at the 1st and 2nd floors in the longitudinal direction.
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 4.4.3.6, is less than 0.30F_y. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3) **Comments:** Checked using the Section 4.4.3.6 Quick Check procedure.
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 4.4.3.9, 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) **Comments:** Checked using the Section 4.4.3.9 Quick Check procedure.

CONNECTIONS

	Description
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) **Comments:** See OLMM dwg. S1.4 ¾” Nelson studs @ 24” o.c.
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) **Comments:** See OLMM det. 5/S3.2

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ASCE 41-17

Collapse Prevention Structural Checklist For Building Type S1-S1A

MODERATE SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN ADDITION TO THE ITEMS FOR LOW SEISMICITY)

SEISMIC-FORCE-RESISTING SYSTEM

	Description
C NC N/A U ○ ● ○ ○	REDUNDANCY: 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) **Comments:** N-S moment frames are non-compliant on Lines A & D at the 5th & 6th floors, where there moment frames have 1 bay.
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) **Comments:** CMU at north façade adjacent to parking structure is outside of Line 6 moment frame. See OLMM detail 19/S3.2
C NC N/A U ○ ● ○ ○	MOMENT-RESISTING CONNECTIONS: All moment connections can 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). **Comments:** Full-penetration flange welds considered non-compliant at Tier 1 per A3.1.3.4

HIGH SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN ADDITION TO THE ITEMS FOR LOW AND MODERATE SEISMICITY)

SEISMIC-FORCE-RESISTING SYSTEM

	Description
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 in accordance with AISC 341, Section A3.2. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1) **Comments:** Full-penetration flange welds considered non-compliant at Tier 1 per A3.1.3.4
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) **Comments:** Noncompliant at 30 of the 32 joints that were checked. See calculations.

Note: **C** = Compliant **NC** = Noncompliant **N/A** = Not Applicable **U** = Unknown

UC Campus:	**San Francisco**			Date:	**01/07/2020**		
Building CAAN:	**2020**	Auxiliary CAAN:		By Firm:	**MSE**		
Building Name:	**Mt. Zion 2330 Post Street, MOB 1**			Initials:	***RBW***	Checked:	***JM***
Building Address:	**2330 Post Street, San Francisco**			Page:	**3**	of	**4**

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type S1-S1A

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) **Comments:** OLMM Det. 19/S1.3 and erection drawings.
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) **Comments:** Noncompliant at 30 of the 32 joints that were checked. See calculations.
C NC N/A U ● ○ ○ ○	COMPACT MEMBERS: All frame elements meet section requirements in accordance with AISC 341, Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.4) **Comments:** See calculations.

DIAPHRAGMS (STIFF OR FLEXIBLE)

	Description
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) **Comments:** The greatest number of diaphragm openings occurs near Line 6. Only Stair #2 opening is immediately adjacent, and it is 18% of the frame length.

FLEXIBLE DIAPHRAGMS

	Description
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) **Comments:**
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) **Comments:**
C NC N/A U ○ ○ ● ○	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) **Comments:**

Note: **C** = Compliant **NC** = Noncompliant **N/A** = Not Applicable **U** = Unknown

UC Campus:	San Francisco		Date:	01/07/2020		
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Collapse Prevention Structural Checklist For Building Type S1-S1A

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 (12.2 m) and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) **Comments:**
C NC N/A U ○ ○ ● ○	OTHER DIAPHRAGMS: Diaphragms do 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) **Comments:**

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UCOP SEISMIC SAFETY POLICY
Falling Hazard Assessment Summary

	Description
P ☐ **N/A** ☒	**Heavy ceilings, features or ornamentation above large lecture halls, auditoriums, lobbies, or other areas where large numbers of people congregate (50 ppl or more)** **Comments:** none observed
P ☐ **N/A** ☒	**Heavy masonry or stone veneer above exit ways or public access areas** **Comments:** none observed
P ☐ **N/A** ☒	**Unbraced masonry parapets, cornices, or other ornamentation above exit ways or public access areas** **Comments:** none observed
P ☐ **N/A** ☒	**Unrestrained hazardous material storage** **Comments:** none observed
P ☐ **N/A** ☒	**Masonry chimneys** **Comments:** none observed
P ☒ **N/A** ☐	**Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.** **Comments:** There are 2 gas-fired boilers on the high roof (domestic hot water and HVAC re-heat) that are mounted to an unrestrained steel skid resting on an equipment pad. Gas lines into the boilers are rigid. Although these are exterior units and there is automatic seismic shut-off at the gas service connection, restraint of and connections to these units should be evaluated and retrofitted as needed.
P ☐ **N/A** ☐	**Other:** **Comments:**

Falling Hazards Risk: Low

Note: P= Present, N/A = Not Applicable; Falling Hazards Risk: Low, Moderate, or High

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SEISMIC EVALUATION OF EXISTING BUILDINGS - TIER 1 SCREENING

ASCE 41-17 Chapter 4

General			(parentheses indicate ASCE 41-17 reference)
Building	Mt. Zion MOB 1, 2330 Post Street		
Architect	ESS Architecture		
Structural Engineer	Olmm Consulting Engineers		
Location	2330 Post St., San Francisco, CA 94115		
Design date	1993		
Latitude	37.7846		Google Earth
Longitude	-122.4404		
Stories above grade	6		

Seismic parameters			
Risk Category	II		CBC 2016 Table 1604.5
Site Class	D		Egan report
Liquefaction hazard	Very Low		Egan report
S_{cS}	1.435	g	Egan report
S_{c1}	0.973	g	Egan report

Scope		
Performance level	CP	(4.1.1, Table 2-1)
Seismic hazard level	BSE-C	(4.1.2, Table 2-1)
Level of seismicity	High	(4.1.3, Table 2-5)
Building type	S1: Steel moment frames with stiff diaphragms	(4.2.2, Table 3-1)

Material properties				Notes	
Steel	F_y	50	ksi	SMRF cols A572 Gr 50	(Table 4-5)
Steel	F_y	49	ksi	SMRF bms A36	(Table 4-5)
Steel	E	29000	ksi		(4.2.3)

Checklists		
Benchmark building	No	UCOP Seismic Program Guidebook v. 1.3 Table 1
Checklist(s) req'd	ASCE 41-17 Collapse Prevention Structural Checklist for Building Type S1 ASCE 41-17 Collapse Prevention Basic Configuration UCOP SEISMIC SAFETY POLICY Falling Hazard Assessment Summary	

3

Seismic forces					
V	4771	kip	$V = CS_a W$	= 0.95W	(Eq. 4-1)
W	5018	kip	building weight		(4.5.2.1)
C	1.0				(Table 4-7)
S_a	0.951	g	$S_a = S_{x1}/T \leq S_{XS}$		(Eq. 4-3)
T	1.02	sec	$T = C_t h_n^{\beta}$		(Eq. 4-4)
C_t	0.035				(4.4.2.4)
β	0.80				(4.4.2.4)
h_n	68.0	ft	building height		

Story forces

$F_{story} = V(wh^k)/(\Sigma wh^k)$ (Eq. 4-2a)

$V_{story} = \Sigma_{above} F_{story}$ (Eq. 4-2b)

k 1.26 k = 1.0 for T < 0.5, 2.0 for T > 2.5, linear interpolation between (4.4.2.2)

						(4-2a)	(4-2b)	5 - 6	1 - 4
Level	w	story ht	h	wh^k	F_{story}	F_{story}	V_{story}	M_{OT}	M_{OT}
	kip	ft	ft			kip	kip	kip·ft	kip·ft
Roof	477		68.0	97878	0.21	999			
6	444	11.00	57.0	72949	0.16	745	999	10989	
5/ low roof	1009	11.00	46.0	126443	0.27	1291	1744	30169	
4	1032	11.50	34.5	89947	0.19	918	3034		34893
3	1032	11.00	23.5	55410	0.12	566	3952		78369
2	1024	11.00	12.5	24796	0.05	253	4518		128065
1		12.50	0.0				4771		187702
totals	5018			467423	1.0	4771			

Drift check (4.4.3.1)

$$D_r = \left(\frac{k_b + k_c}{k_b k_c}\right)\left(\frac{h}{12E}\right)V_c \qquad (4\text{-}6)$$

D_r = drift ratio for stories with continuous columns above and below

direction	story	column section	I_c in^4	h in	k_c	beam section	I_b in^4	L in	k_b	V_{story} kip	n_col	V_c kip	D_r
E-W	5	W14X82	881	132	6.7	W27X94	3270	330	9.9	1744	4	436	0.041
	5	W14X159	1900	132	14.4	W27X102	3620	330	11.0	1744	4	436	0.027
	3	W14X159	1900	132	14.4	W27X114	4080	330	12.4	3952	6	659	0.038
	2	W14X159	1900	132	14.4	W27X114	4080	330	12.4	4518	10	452	0.026
	1	W14X159	1900	150	12.7	W27X114	4080	330	12.4	4771	10	477	0.033
N-S	5	W14X82	881	132	6.7	W27x94	3270	300	10.9	1744	4	436	0.040
	5	W14X159	1900	132	14.4	W27x94	3270	300	10.9	1744	4	436	0.027
	3	W14X159	1900	132	14.4	W27x94	3270	300	10.9	3952	8	494	0.030
	2	W14X159	1900	141	13.5	W27x94	3270	300	10.9	4518	8	565	0.038
	1	W14X159	1900	150	12.7	W27x94	3270	300	10.9	4771	8	596	0.044

Column axial stress check (4.4.3.6)

$$p_{ot} = \frac{1}{M_s}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right)\left(\frac{1}{A_{col}}\right) \qquad \text{(Eq. 4-11)}$$

direction	**E/W**	**N/S**		
V	4771	4771	kip	
n_f	3	2		total no.of frames in the direction of loading
h_n	68.0	68.0	ft	
L	82.3	73.5	ft	
col_sec	W14X159	W14X159		end column section
A_{col}	46.70	46.70	in^2	
M_s	2.5	2.5		CP
p_{ot}	7.50	12.60	ksi	
F_y	50	50	ksi	(Table 4-5)
$P_{ot} < 0.3F_y$	YES	YES		

Strong column - weak beam check AISC 341-16 Sect. E.4.a

check 2 representative SMRFs

Line 4 (transverse direction)

Line D (east façade, longitudinal direction)

$$\frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} > 1.0$$

AISC 341-16 Eq E3-1

$$\Sigma M^*_{pc} = \Sigma Z_c (F_{yc} - \alpha_s P_r / A_g)$$

AISC 341-16 Eq E3-1

SMRF columns:	**W14X132**	**W14X82**	
Z_c	234	139	in^3
A_g	38.8	24	in^2
d_c	14.7	14.3	in
t_w	0.645	0.510	in
α_s	1.0		(factored loads)
P_r	$P_{grav} + P_{eq}$		

P_{grav} 1.1(D+0.25·L)·TA kips TA = trib area, L = floor & roof live loads, unreduced (Eq. 7-1)

Frame	4		D			
Col	C	B	2	3	4	5
6	36.8	36.8			19.3	37.4
5	74.6	74.6			39.1	76.0
4	146.4	146.4	35.8	35.8	76.7	115.2
3	225.4	225.4	78.3	78.4	118.0	155.2
2	304.3	304.3	120.9	121.0	159.3	195.3
1	382.4	382.4	163.0	163.2	200.2	234.9

P_{eq} $(M_{OT} \cdot \overline{x})\cdot A_g/(n_{frame}\cdot I_{frame})$ kips

Frame	4		D			
Col	C	B	2	3	4	5
$\overline{x}$ *(ft)*	13.75	13.75	37.13	12.13	12.88	36.38
6	20.1	20.1			233.82	233.82
5	55.1	55.1			641.9	641.8945
4	97.5	97.5	214.9	70.2	716.4	852.44
3	150.4	150.4	482.6	157.6	809.3	1114.8
2	210.8	210.8	788.7	257.6	915.4	1414.6
1	283.4	283.4	1155.9	377.5	1042.8	1774.5

P_r $P_{grav} + P_{eq}$ kips

Frame	4		D			
Col	C	B	2	3	4	5
6	56.8	56.8			253.1	271.3
5	129.6	129.6			680.9	717.8
4	243.9	243.9	250.7	106.0	793.1	967.6
3	375.8	375.8	561.0	236.0	927.3	1270.0
2	515.2	515.2	909.6	378.6	1074.7	1609.9
1	665.8	665.8	1319.0	540.7	1243.0	2009.4

M_{pc} $Z_c(F_{yc} - \alpha_s P_r/A_g)$ kip·in

Frame	4		D			
Col	C	B	2	3	4	5
6	6621	6621			5484	5379
5	10918	10918			7593	7371
4	10229	10229	10188	11061	6917	5864
3	9434	9434	8317	10276	6108	4041
2	8593	8593	6214	9417	5218	1991
1	7684	7684	3745	8439	4204	-418

$\sum M^{*}_{pb} = \sum(M_{pr} + M_v)$

$M_{pr} = F_{yb} \cdot Z_{bx}$ L = distance between column centerlines

$V_p = 2 \cdot M_{pr} / L`$    $L` = L - 2e - d_c$

$M_v = V_p(d_c/2+e)$ e = 0

beam sections

Frame	4			D			
level / bay	D - C	C - B	B - A	2 - 3	3 - 4	4 - 5	
L (ft)	27.42	27.50	27.42	25.00	25.00	23.50	
L` (ft)	26.23	26.31	26.23	23.81	23.81	22.31	W14X82
L` (ft)	26.19	26.28	26.19	23.78	23.78	22.28	W14X132
high roof	W24X76	W27X94	W24X76			W27X94	
6	W24X76	W27X94	W24X76			W27X94	
5	W24X76	W27X102	W24X76	W27X94	W27X94	W27X94	
4	W24X76	W27X114	W24X76	W27X94	W27X94	W27X94	
3	W24X76	W27X114	W24X76	W27X94	W27X94	W27X94	
2	W24X76	W27X114	W24X76	W27X94	W27X94	W27X94	

			W14X132	W14X82
beam	Z_{bx}	e	$d_c/2+e$	$d_c/2+e$
	in^3	in	in	in
W24X76	200	0.00	7.35	7.15
W27X94	278	0.00	7.35	7.15
W27X102	305	0.00	7.35	7.15
W27X114	343	0.00	7.35	7.15

M_{pr}   kip·in

Frame	4			D		
level / bay	D - C	C - B	B - A	2 - 3	3 - 4	4 - 5
high roof	9800	13622	9800			13622
6	9800	13622	9800			13622
5	9800	14945	9800	13622	13622	13622
4	9800	16807	9800	13622	13622	13622
3	9800	16807	9800	13622	13622	13622
2	9800	16807	9800	13622	13622	13622

`

V_p   kip

Frame	4			D		
level / bay	D - C	C - B	B - A	2 - 3	3 - 4	4 - 5
high roof	62.3	86.3	62.3			101.8
6	62.3	86.3	62.3			101.8
5	62.4	94.8	62.4	95.5	95.5	101.9
4	62.4	106.6	62.4	95.5	95.5	101.9
3	62.4	106.6	62.4	95.5	95.5	101.9
2	62.4	106.6	62.4	95.5	95.5	101.9

M_v   kip·in

Frame	4			D		
level / bay	D - C	C - B	B - A	2 - 3	3 - 4	4 - 5
high roof	445	635	458			749
6	445	635	458			749
5	458	697	458	702	702	749
4	458	784	458	702	702	749
3	458	784	458	702	702	749
2	458	784	458	702	702	749

M_{pb} kip·in

Frame	4			D		
level / bay	D - C	C - B	B - A	2 - 3	3 - 4	4 - 5
high roof	10245	14257	10258			14371
6	10245	14257	10258			14371
5	10258	15642	10258	14324	14324	14371
4	10258	17591	10258	14324	14324	14371
3	10258	17591	10258	14324	14324	14371
2	10258	17591	10258	14324	14324	14371

$\sum M^*_{pc}$ kip·in

Frame		4			D			
level/col		C	B		2	3	4	5
high roof		6621	6621				5484	5379
6		17539	17539				13078	12750
5		21147	21147		10188	11061	14510	13235
4		19663	19663		18505	21337	6108	9905
3		18027	18027		14531	19693	11326	6031
2		16278	16278		9960	17856	9422	1572

$\sum M^*_{pb}$

Frame		4			D			
level/col		C	B		2	3	4	5
high roof		24502	24515				14371	14371
6		24502	24515				14371	14371
5		25900	25900		14324	28648	28695	14371
4		27849	27849		14324	28648	28695	14371
3		27849	27849		14324	28648	28695	14371
2		27849	27849		14324	28648	28695	14371

$\sum M^*_{pc}/\sum M^*_{pb}$

Frame		4			D			
level/col		C	B		2	3	4	5
high roof		0.27	0.27				0.38	0.37
6		0.72	0.72				0.91	0.89
5		0.82	0.82		0.71	0.39	0.51	0.92
4		0.71	0.71		1.29	0.74	0.21	0.69
3		0.65	0.65		1.01	0.69	0.39	0.42
2		0.58	0.58		0.70	0.62	0.33	0.11

Flexural stress check -beams (4.4.3.9)

$$f_j^{avg} = V_j \frac{1}{M_s} \left(\frac{n_c}{n_c - n_f} \right) \left(\frac{h}{2} \right) \frac{1}{Z} \qquad (4\text{-}14)$$

n_{mc} = no. of beam ends with moment connection to column

M_s 9.0 CP

direction	level	h in	V_j kip	n_f	n_c	beam sections	Z_b in^3	n_{mc}	$Z_b \cdot n_{mc}$ in^3	Z in^3	f_j^{avg} ksi
E-W	high roof	132	999	2	4	W24X76	200	1	200	2122	6.90
						W27X94	278	2	556		
						W24X76	200	1	200		
						W27X94	278	1	278		
						W27X102	305	2	610		
						W27X94	278	1	278		
	6	132	1744	2	4	W24X76	200	1	200	2054	12.45
						W27X94	278	2	556		
						W24X76	200	1	200		
						W27X84	244	1	244		
						W27X102	305	2	610		
						W27X84	244	1	244		
	5/LR	138	3034	3	6	W27X84	244	1	244	3206	14.51
						W27X102	305	2	610		
						W27X84	244	1	244		
						W24X76	200	1	200		
						W27X102	305	2	610		
						W24X76	200	1	200		
						W27X84	244	1	244		
						W27X102	305	2	610		
						W27X84	244	1	244		
	4	132	3952	3	6	W27X84	244	1	244	3426	16.92
						W27X102	305	2	610		
						W27X84	244	1	244		
						W24X76	200	1	200		
						W27x114	343	2	686		
						W24X76	200	1	200		
						W27X94	278	1	278		
						W27x114	343	2	686		
						W27X94	278	1	278		
	3	132	4518	3	6	W27X84	244	1	244	3426	19.34
						W27X102	305	2	610		
						W27X84	244	1	244		
						W24X76	200	1	200		
						W27x114	343	2	686		
						W24X76	200	1	200		
						W27X94	278	1	278		
						W27x114	343	2	686		
						W27X94	278	1	278		
	2	150	4771	3	10	W21X147	373	2	746	4958	11.46
						W27X102	305	2	610		
						W27X102	305	2	610		
						W24X76	200	1	200		
						W27x114	343	2	686		
						W24X76	200	1	200		
						W27X102	305	2	610		
						W27x114	343	2	686		
						W27X102	305	2	610		

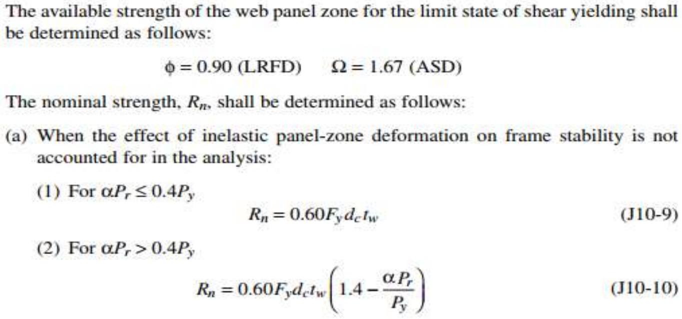
N-S	high roof	132	999	2	4	W27X94	278	2	556	1112	13.2
						W27X94	278	2	556		
	6	132	1744	2	4	W27X94	278	2	556	1112	23.0
						W27X94	278	2	556		
	5/LR	138	3034	2	8	W27X94	278	2	556	3336	9.30
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
	4	132	3952	2	8	W27X94	278	2	556	3336	11.58
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
	3	132	4518	2	8	W27X94	278	2	556	3336	13.24
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
	2	150	4771	2	8	W27X94	278	2	556	3336	15.89
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		
						W27X94	278	2	556		

Flexural stress check - columns (4.4.3.9)

M_s 9.0 CP

direction	level	h in	V_j kip	n_f	n_c	column sections	Z_c in^3	Z in^3	f_j^{avg} ksi
E-W	high roof	132	999	2	4	W14X82	139	556	26.35
	6	132	1744	2	4	W14X82	139	556	45.99
	5/LR	138	3034	3	6	W14X159	287	1722	27.02
	4	132	3952	3	6	W14X159	287	1722	33.66
	3	132	4518	3	6	W14X159	287	1722	38.48
	2	150	4771	3	10	W14X159	287	2870	19.79
N-S	high roof	132	999	2	4	W14X82	139	556	26.35
	6	132	1744	2	4	W14X82	139	556	45.99
	5/LR	138	3034	2	8	W14X159	287	2296	13.51
	4	132	3952	2	8	W14X159	287	2296	16.83
	3	132	4518	2	8	W14X159	287	2296	19.24
	2	150	4771	2	8	W14X159	287	2296	23.09

Panel zone capacity check



The available strength of the web panel zone for the limit state of shear yielding shall be determined as follows:

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

The nominal strength, R_n, shall be determined as follows:

(a) When the effect of inelastic panel-zone deformation on frame stability is not accounted for in the analysis:

(1) For $\alpha P_r \le 0.4P_y$

$$R_n = 0.60F_y d_c t_w \qquad \text{(J10-9)}$$

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(2) For $\alpha P_r > 0.4P_y$

$$R_n = 0.60F_y d_c t_w \left(1.4 - \frac{\alpha P_r}{P_y}\right) \qquad \text{(J10-10)}$$

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beam	*d*	*tf*
	in	in
W27X94	26.9	0.745
W27X102	27.1	0.83
W24X76	23.9	0.68
W27x114	27.3	0.93

	W14X132	W14X82	
$0.6{\cdot}F_y{\cdot}d_c{\cdot}t_w$ =	284	219	kips
$P_y = F_y{\cdot}A_g$ =	1940	1200	kips

P_r/P_y

Frame	4		D			
Col	C	B	2	3	4	5
6	0.11	0.11			0.57	0.60
5	0.13	0.13	0.13	0.05	0.41	0.50
4	0.19	0.19	0.29	0.12	0.48	0.65
3	0.27	0.27	0.47	0.20	0.55	0.83
2	0.34	0.34	0.68	0.28	0.64	1.04

$(1.4 - P_r/P_y) < 1.0$

Frame	4		D			
Col	C	B	2	3	4	5
6	1	1			0.83	0.80
5	1	1	1	1	0.99	0.90
4	1	1	1	1	0.92	0.75
3	1	1	0.93	1	0.85	0.57
2	1	1	0.72	1	0.76	0.36

Qty 1/2" Doubler Plates S3.1

Frame	4		D			
Col	C	B	2	3	4	5
6	2	2			1	1
5	2	2	0	2	2	0
4	2	2	0	2	2	0
3	2	2	0	2	2	0
2	2	2	0	2	2	0

R_n kip

Frame	4		D			
Col	C	B	2	3	4	5
6	648	648			361	347
5	725	725	284	725	719	256
4	725	725	284	725	669	212
3	725	725	265	725	614	162
2	725	725	205	725	551	104

$\sum M^{*}_{pb}$ kip·in

Frame	4		D			
level/col	C	B	2	3	4	5
6	24502	24515			14371	14371
5	25900	25900	14324	28648	28695	14371
4	27849	27849	14324	28648	28695	14371
3	27849	27849	14324	28648	28695	14371
2	27849	27849	14324	28648	28695	14371

d_b in

Frame	4		D			
level/col	C	B	2	3	4	5
6	26.9	26.9			26.9	26.9
5	27.1	27.1	26.9	26.9	26.9	26.9
4	27.3	27.3	26.9	26.9	26.9	26.9
3	27.3	27.3	26.9	26.9	26.9	26.9
2	27.3	27.3	26.9	26.9	26.9	26.9

$0.8\ V_b$ kip

Frame	4		D			
level/col	C	B	2	3	4	5
6	728.70	729.08			427.39	427.39
5	764.58	764.58	425.99	851.98	853.38	427.39
4	816.09	816.09	425.99	851.98	853.38	427.39
3	816.09	816.09	425.99	851.98	853.38	427.39
2	816.09	816.09	425.99	851.98	853.38	427.39

$0.8V_b/R_n$

Frame	4		D			
level/col	C	B	2	3	4	5
6	1.12	1.13			1.18	1.23
5	1.05	1.05	1.50	1.17	1.19	1.67
4	1.12	1.12	1.50	1.17	1.28	2.02
3	1.12	1.12	1.61	1.17	1.39	2.64
2	1.12	1.12	2.08	1.17	1.55	4.13

Compact member check

E	29000	ksi	elastic modulus	
F_{y36}	37	ksi	specified min yield stress	
F_{y50}	50	ksi	specified min yield stress	
R_{y36}	1.5		expected/min yield stress ratio	AISC 341 Table A3.1
R_{y50}	1.1		expected/min yield stress ratio	AISC 341 Table A3.1
ϕ_c	0.9		resistance factor for compression	AISC 360 H1.1
C_a	0.1		(assumed value)	AISC 341 Table D1.1

Check moment frame members using Table D1.1

Section	F_y	R_y	b/t	λ_{md}	Check	h/t_w	λ_{md}	Check
W14X82	50	1.1	5.92	9.2	C	22.4	63	C
W14X159	50	1.1	6.54	9.2	C	15.3	63	C
W27X94	37	1.5	6.70	9.1	C	49.5	63	C
W27X102	37	1.5	6.03	9.1	C	47.1	63	C
W27X114	37	1.5	5.41	9.1	C	42.5	63	C
W24x76	37	1.5	6.61	9.1	C	49	63	C
W27X84	37	1.5	7.78	9.1	C	52.7	63	C