UCSF

Rating form completed by:

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Date: 11/18/19

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

DATE: 2019-11-18

REGISTERED PROFESSIONAL ENGINEER BRET LIZUNDIA No. S3950 Exp. 12-31-20 STRUCTURAL STATE OF CALIFORNIA

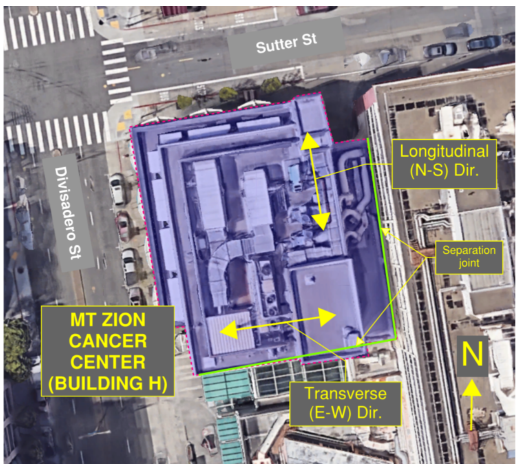
11-18-19

UCSF building seismic ratings

Mount Zion Cancer Center Building H

CAAN #3004
1600 Divisadero Street, San Francisco, CA 94115
UCSF Campus: Mount Zion

Plan



North elevation (looking south)

Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	IV	Findings based on drawing review and ASCE 41-17 Tier 1 evaluation[1]
Rating basis	Tier 1	ASCE 41-17
Date of rating	2019	
Recommended UCSF priority category for retrofit	N/A	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application for modification
Ballpark total project cost to retrofit to IV rating	N/A	See recommendations on further evaluation and retrofit
Is 2018-2019 rating required by UCOP?	Yes	Does not have a documented previous review
Further evaluation recommended?	No	

[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.

Building information used in this evaluation

- Structural drawings by Degenkolb Engineers, “UCSF Mount Zion Hospital Outpatient Cancer Center,” dated 30 July 1998, structural Sheets S100 to S103, S200 to S208, S401, S501 to S503, and S701 to S703.

Additional building information known to exist

- Architectural drawings by Stone, Marraccini & Patterson Architecture, Planning and Interior Architecture, “UCSF Mount Zion Hospital Outpatient Cancer Center,” dated 30 July 1998.

Scope for completing this form

The structural drawings for the original 1998 construction were reviewed, and these drawings are used as the basis for the completed ASCE 41-17 Tier 1 evaluation. A site visit was made on 23 September 2019 where the building exterior and portions of the interior were observed.

Brief description of structure

The Cancer Center, also known as Building H, is a seven-story structure located at the corner of Sutter Street and Divisadero Street in San Francisco, CA. It comprises one of several interconnected buildings that form the UCSF Mt. Zion Medical Campus. It is seismically separated from Building R located to the south and Building B located to the north. The structure contains two below grade stories and five above grade stories. It is rectangular in shape and measures approximately 129’-10” in the north-south direction by 104’-11” in the east-west direction.

The structure currently functions as a medical office building providing out-patient care to cancer patients. However, a number of spaces are currently vacant as their services have been relocated to the recently opened Precision Cancer Center located at the UCSF Mission Bay campus.

<u>Identification of levels</u>: The building levels are designated by the building occupants as the basement (EL. 108’-0”), the mezzanine (EL. 122’-6”), the first floor (EL. 133’-6”), the second floor (EL. 146’-6”), the third floor (EL. 159’-6”), the fourth floor (EL. 172’-6”), the fifth floor (EL. 185’-6”), the penthouse floor (198’-6”), and the roof (EL. 211’-6”). The story located between the penthouse floor and the roof contains a small footprint that serves as a small mechanical space. The exterior grade is located at the first floor. The Cancer Center is connected to Building B at the mezzanine floor and to Building R at the first and second floor.

<u>Foundation system</u>: The Cancer Center is supported by a 2’-10” thick mat foundation that is reinforced with #9 bars spaced at 12” o.c. in each direction at the top and bottom layers. Additional reinforcing is located on the north side of the structure. This region of the building contains thick lead lined concrete walls that serve as shielding for equipment that utilizes radiation. A portion of the mat slab located in the southwest corner of the structure is thickened to 5’-9”. This region of the mat is noted on the original construction drawings as “tower crane support.” This portion is also located a below an access hatch allows for the basement equipment to be serviced and replaced.

<u>Structural system for vertical (gravity) load:</u> The gravity load-carrying system consists of 3 ¼” lightweight concrete fill over 3”deep metal deck that spans to W14x22 steel beams and W16x31 steel girders. The beams are oriented in the east-west direction and are spaced 11’-4” apart. The typical bay size is 22’-8” x 22’-8”, and W14 steel columns support the floor framing.

The lower two stories of the Cancer Center are below grade. Reinforced concrete retaining walls are located around the building perimeter. These walls typically contain concrete pilasters located at the inside face of wall that supports the steel framing. However, there is one exception. The wall located along the south elevation is gravity load bearing as it directly supports steel girders.

<u>Structural system for lateral forces</u>: Above grade, the lateral load-carrying system is comprised of reinforced concrete diaphragms that span approximately 113 ft to steel moment-resisting frames. The moment frames are located either on or close to the building perimeter. There are two lines of frames in each direction at each story, except at between the fourth and fifth floor, where there are three lines of frames in each direction. At the fifth floor, one frame line in each direction offsets horizontally by one bay. Each frame line typically contains three bays of moment frames. The number of frames, size of the members, and the spans are symmetrical in each direction.

The frame beams are W24x94, W24x117, and W27x146, and the frame columns are W14x233, W14x311, and W14x342. The lateral system utilizes reduced beam section end connections. The beam flanges are welded to the column with complete penetration welds. At the top flange, the back-up bar remains and a 5/16" thick reinforcing fillet weld was added to the underside of the back-up bar. At the bottom flange, the back-up bar was removed, the weld was back gouged, and a 5/16" reinforcing fillet weld was added. Continuity plates are provided in the column web and are aligned with the beam top and bottom flanges. The plate thickness is equal to the thickness of the beam flange increased by ¼". The back-up bars at the continuity plates remain and a 5/16" thick reinforcing fillet weld was added at the underside of the back-up bars. The drawings specify that the notch toughness of weld filler material used for the complete penetration weld be not less than 20 ft-lbs at a temperature of -20 degrees Fahrenheit. Doubler plates are not provided in the column panel zone. The reduced beam section is braced laterally with W16x26 beams at the interior end of the protected zone. This project was designed in 1998 and references the 1994 Uniform Building Code. Although the 1994 UBC would not include post-Northridge modifications, it appears the project did incorporate a number of these recommendations.

Below grade, the lateral load-resisting system consists of reinforced concrete shear walls located around the building perimeter. The walls are 14", 16", 18", and 22" thick and contain a minimum horizontal reinforcing ratio of 0.0025 and a minimum vertical reinforcing ratio of 0.0052. The lowest story contains additional interior shear walls that are located to form shielding around the radiology equipment. The walls range in thickness from 4" to 66" and are typically lined with lead. The contain a minimum horizontal reinforcing ratio of 0.0025 and a minimum vertical reinforcing ratio of 0.002. The first-floor slab serves as a transfer diaphragm to deliver load from the moment frames and into the shear walls which are offset horizontally from the frames.

<u>Building condition</u>: Good. No on-going maintenance problems were noted by the building administrator. The roof and roof-top mechanical equipment are showing signs of age as some equipment housing, anchors, and skids are severely corroded.

<u>Building response in 1989 Loma Prieta Earthquake</u>: Not applicable. The Cancer Center was constructed after this seismic event.

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:

- The lateral force-resisting system offsets horizontally in two locations. At the fifth floor, the moment frames shift lines by one bay. At the first floor, the lateral system transitions from moment frames above to concrete shear walls below. The walls are offset horizontally from the moment frames by one bay.
- At the fifth floor, the two-bay moment frames located on Line 2 and Line F offset horizontally to Line 1 and Line G, respectively. As such, between the fourth to fifth floor there are 8 bays of frames in each direction. In the stories below the fourth floor, there are six bays of moment frames in each direction. The shear demand-to-capacity ratio between the third to fourth floor is 67% higher than between the fourth to fifth floor. This meets the ASCE 41 Tier 1 criteria for a potential weak story. However, the structure has sufficient shear capacity to resist the shear demands from BSE-C at all stories.
- The calculated interstory drift between the third and fourth floor is twice the interstory drift between the fourth and fifth floor. The reduction in story stiffness is more than the ASCE 41 limit of 70%; therefore, the building may have a soft story.
- The structure may contain inadequate seismic separation from adjacent buildings. The provided gap does not meet the ASCE 41-17 criteria of 1.5% times the story height. The provided gaps are 1", 2", 4", and 6", and the required gaps are 2", 4.14", 6.5", and 8.82" at the corresponding floor levels.
- The concrete shear wall on Line G is discontinuous below the mezzanine slab.
- The interstory drift ratio as calculated per ASCE 41-17 Section 4.4.3.1 is 0.036 and 0.04 between the second to third floor and the third to fourth floor, respectively. These exceed the Tier 1 limit of 0.03. When checked using

a limit of ASCE 7-10 with the forces prescribed by the BSE-1N seismic hazard level, the drift ratios are less than 0.02.

- The panel zones of the interior moment frame columns are slightly overstressed. They contain a demand-to-capacity ratio of 1.10.
- Slab openings are located adjacent to the moment frame located on Line B that comprise more than 25% of the total frame length. A slab opening is located adjacent to the wall on Line 1 that comprises more than 25% of the total wall length.

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)	Y	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 nonstructural life-safety concerns, including at exit routes. [2]

The egress stairs in the Cancer Center are constructed from steel plate stringers. No movement joints were observed at the stair landings or the floor levels. The construction documents indicate the interstory drift is 2”. Given this drift, forces that exceed the capacity of the stringers and their connections may develop.

UCOP nonstructural checklist item	Life safety hazard?	UCOP nonstructural 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	Bracing of the compressed gas storage is unknown.
Heavy masonry or stone veneer above exit ways and public access areas	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.	Gas is supplied to the structure. Bracing of the line is unknown.

Basis of Seismic Performance Level rating

The Cancer Center is a rectangular structure that contains a symmetrically located lateral load-resisting system. It utilizes special steel moment resisting frames above grade and reinforced concrete shear walls below grade. The

[2] 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 if and where nonstructural hazards may occur.

lateral elements are reasonably spaced apart and are located around the perimeter of the structure. The Cancer Center was designed to the 1994 UBC and thus does not qualify to be bench marked per the 3/26/19 UCOP Guidebook Version 1.3 policy. However, post-Northridge steel moment frame design detailing with reduced beam section beam-to-column connections were utilized. When checked for the demands from BSE-C, the maximum column axial stress is 8.2 ksi which is below the ASCE 41-17 limit of 15.0 ksi. The maximum column and beam flexural stresses are 13.9 and 20.2 ksi, respectively. These are also below the ASCE 41-17 limit of 50 ksi. The shear capacity of the moment frame columns is larger than the BSE-C story shear. In addition, the drift of the structure was checked using ASCE 7-10 with the BSE-1N seismic hazard level and was found to be below 0.02 at all stories. The average shear stresses in the reinforced concrete walls are low. The maximum stress is 47 psi which is well below the ASCE 41-17 Tier 1 limit of 126 psi.

At the fifth floor, the two-bay moment frames located on Line 2 and Line F offset horizontally to Line 1 and Line G, respectively. As such, between the fourth to fifth floor there are 8 bays of frames in each direction. In the stories below the fourth floor, there are six bays of moment frames in each direction. The reduction in the number of frames between the third to fourth floor as compared to the story above meets the ASCE 41 Tier 1 criteria for a potential weak and a potential soft story. Despite these deficiencies, it is expected that the building will perform in a ductile manner. The moment frames are well-detailed with strong-column weak-beam mechanisms, and it is likely that plastic hinges will form in the reduced section of the moment frame beams up the full height of the frame despite the additional frames from the fourth to fifth floors. When examined for the demands imposed by a plastic hinge forming in the beam, the column panel zones are slightly overstressed and have a demand-to-capacity ratio of 1.10. Finally, all of the moment frames meet the ASCE 41-17 provisions for strong column-weak beam.

The building is assigned a Seismic Performance Level rating of IV because the structure is expected to perform in a ductile manner in the nonlinear range. The assessment required to assign a Rating of III is beyond the scope of an ASCE 41-17 Tier 1 evaluation.

Note that the ASCE 41-17 Tier 1 demands do not include the increase that would result if the requirements of ASCE 7-16 Section 11.4.8-3 were applied. F_v would rise from 1.742 to 1.5 x 1.742 = 2.61; then S_{c1} would rise from 0.972 to 1.5 x 0.972 = 1.458; and T_s would become S_{c1}/S_{cs} = 1.458/1.433 = 1.02 seconds which exceeds T = 0.99 seconds. Thus, S_a would increase by a factor of 1.46 from 0.98g to the S_{cs} = 1.433g short period cap.

Recommendations for further evaluation or retrofit

No additional analysis is required.

Peer review comments on rating

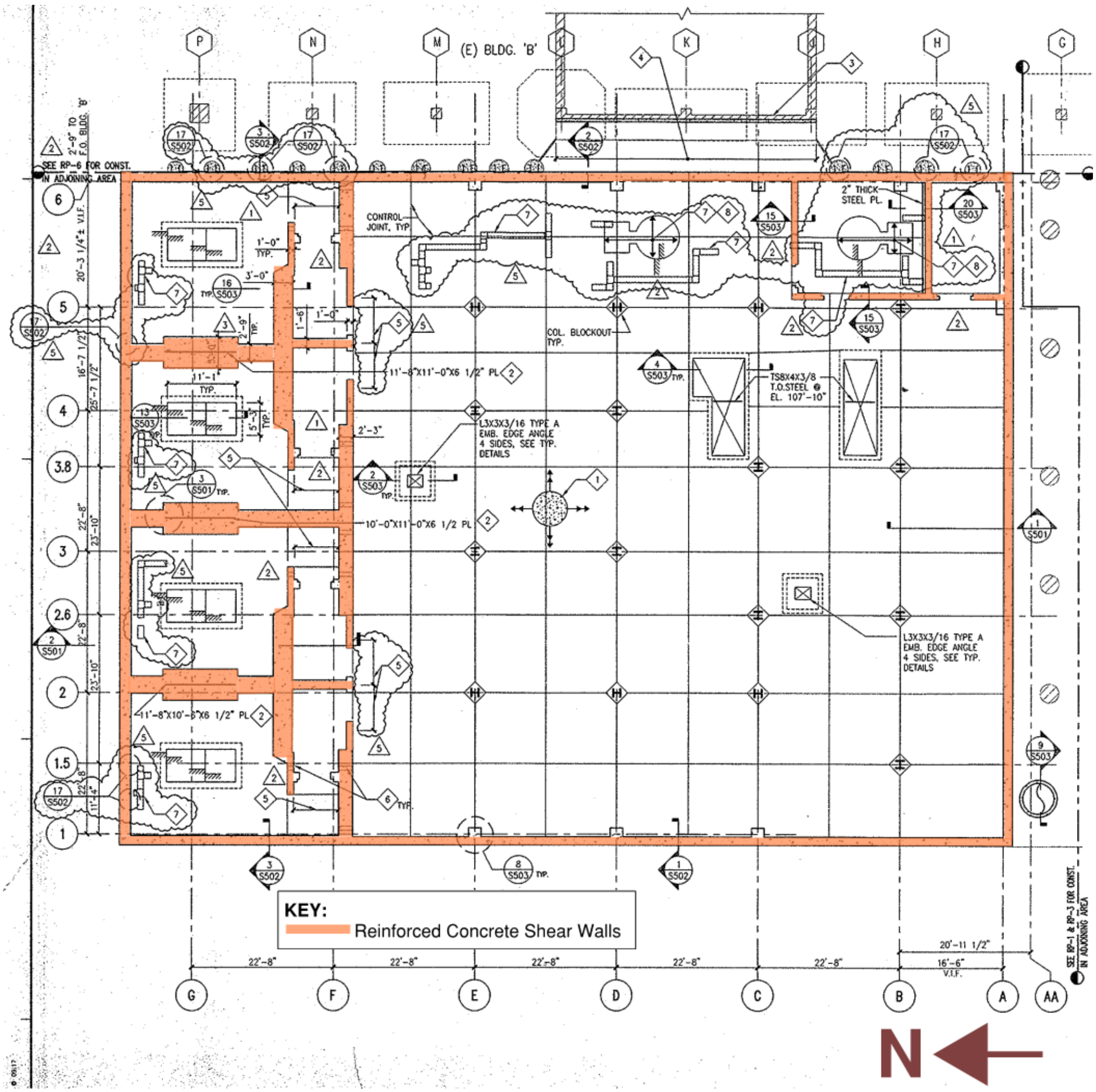
The structural members of the UCSF Seismic Review Committee (SRC) reviewed the evaluation on 18 November 2019 and were unanimous that the Seismic Performance Level Rating is Level IV. No additional analysis is required.

Additional building data	Entry	Notes
Latitude	37.78500	
Longitude	-122.43950	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	5	
Number of stories (basements) below lowest perimeter grade	2	
Building occupiable area (OGSF)	89,862	
Risk Category per 2016 CBC 1604.5	II	

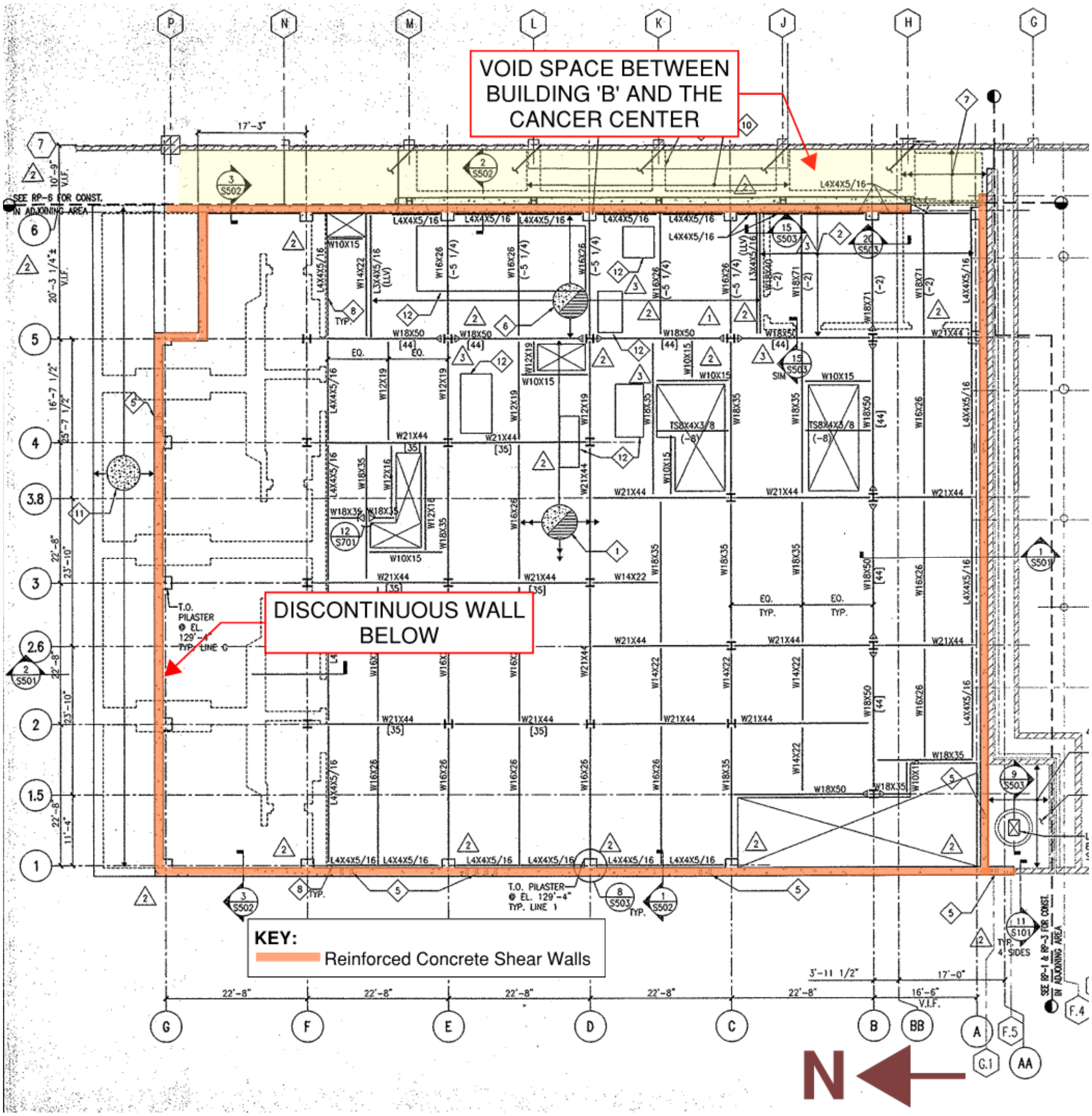
Building structural height, h_n	65.0 ft	Structural height defined per ASCE 7-16 Section 11.2
Coefficient for period, C_t	0.035	Estimated using ASCE 41-17 equation 4-4 and 7-18
Coefficient for period, β	0.8	Estimated using ASCE 41-17 equation 4-4 and 7-18
Estimated fundamental period	0.99 sec	Superstructure period is estimated using ASCE 41-17 equation 4-4 and 7-18
Site data		
975-year hazard parameters S_s, S_1	1.433g, 0.558g	UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019)
Site class	D	UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019)
Site class basis	Estimated	
Site parameters F_a, F_v	1.0, 1.742[13]	UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019)
Ground motion parameters S_{cs}, S_{c1}	1.433g, 0.972 g	UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019)
S_a at building period	0.98g	Superstructure: W = 6,584 kips, V base = 6,483 kips Substructure: W = 6,095 kips, V base = 6,997 kips (including V base from superstructure above)
Site V_{s30}	308 m/s	UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019)
V_{s30} basis	Estimated	
Liquefaction potential/basis	No	UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019)
Landslide potential/basis	No	UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019)
Active fault-rupture hazard identified at site?	No	UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019)
Site-specific ground motion study?	No	
Applicable code		
Applicable code or approx. date of original construction	Built: 1999 Code: 1994 UBC	
Applicable code for partial retrofit	None	No partial retrofit known
Applicable code for full retrofit	None	No full retrofit known

[3] The F_v factor used does not include the requirements of ASCE 7-16 Section 11.4.8-3 that are applicable to Site Class D and which per Exception 2 would result in an effective F_v of 2.61 (1.5 times larger than 1.742). At the Mt. Zion campus, this only affects structures with T > S_{c1}/S_{cs} = 0.972/1.433 = 0.68 seconds.

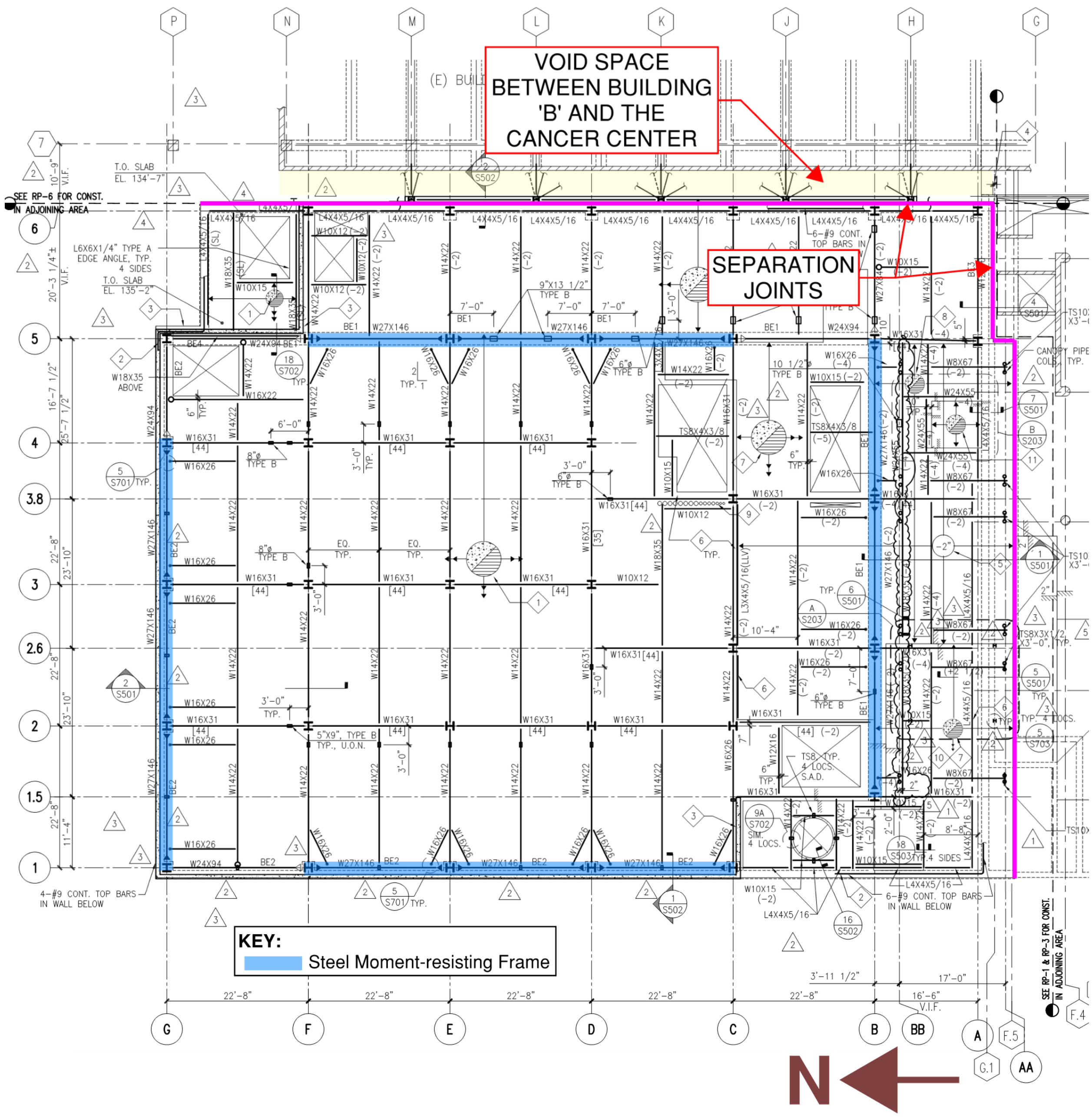
Model building data		
Model building type north-south	C2 Concrete Shear Walls S1 Steel Moment Frames	C2 for the stories below ground S1 for the stories above ground
Model building type east-west	C2 Concrete Shear Walls S1 Steel Moment Frames	C2 for the stories below ground S1 for the stories above ground
FEMA P-154 score	N/A	Not applicable as an ASCE 41 Tier 1 evaluation was performed
Previous ratings		
Most recent rating	III	
Date of most recent rating	2013	
2nd most recent rating	-	
Date of 2nd most recent rating	-	
3rd most recent rating	-	
Date of 3rd most recent rating	-	
Appendices		
ASCE 41 Tier 1 checklist included here?	Yes	Refer to attached checklist file



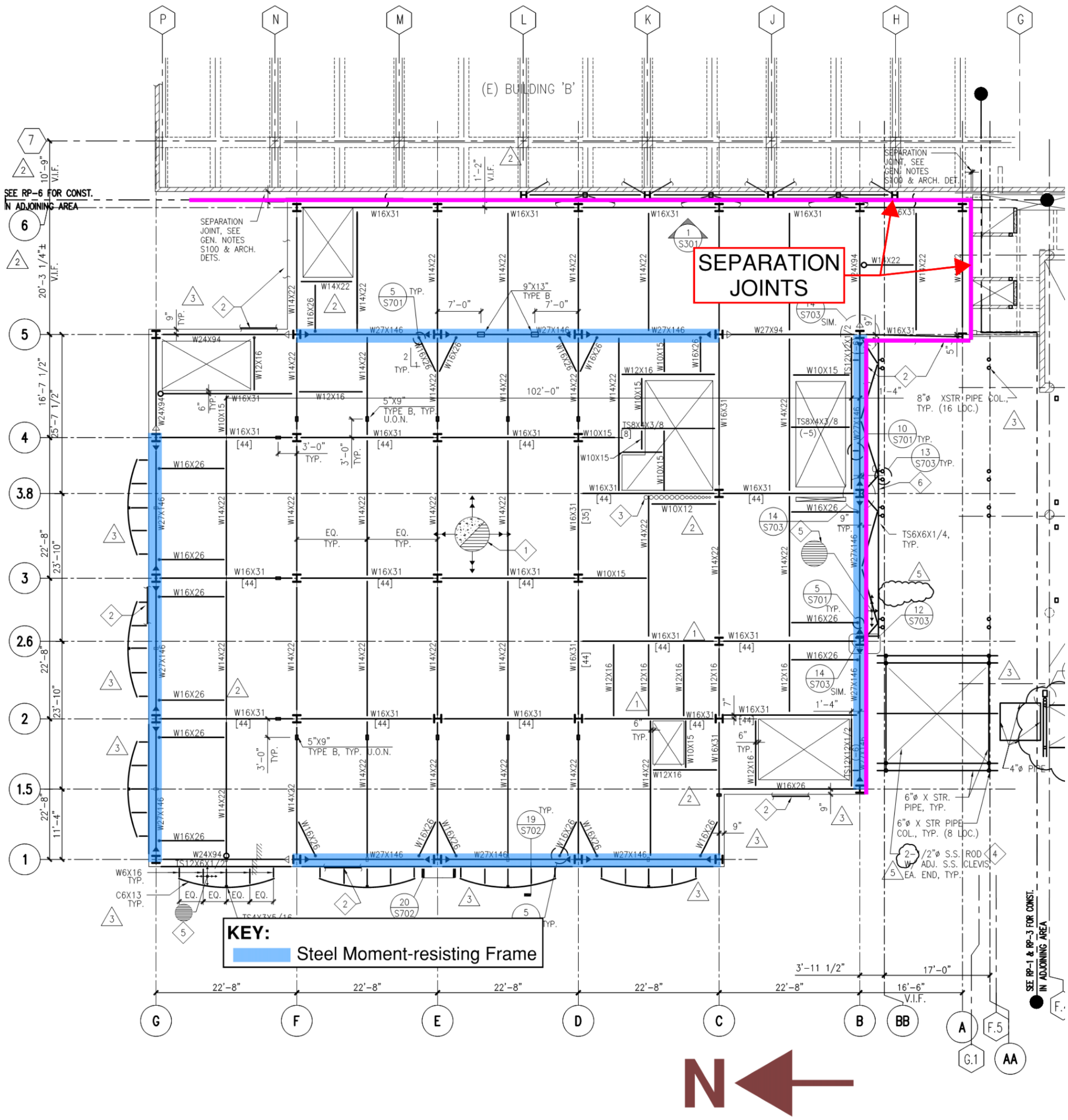
Lateral force-resisting system at the basement floor



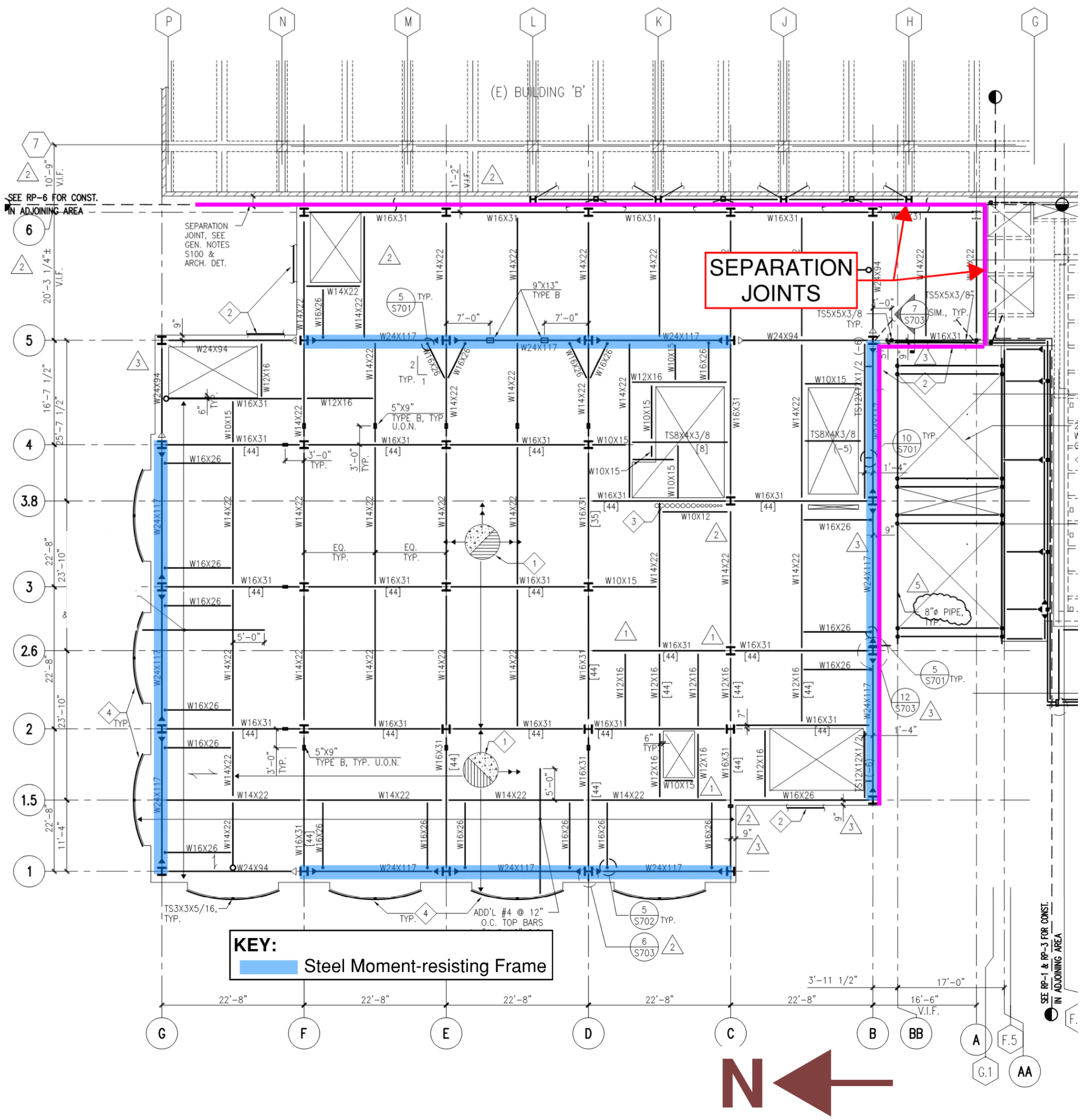
Lateral force-resisting system at the mezzanine floor



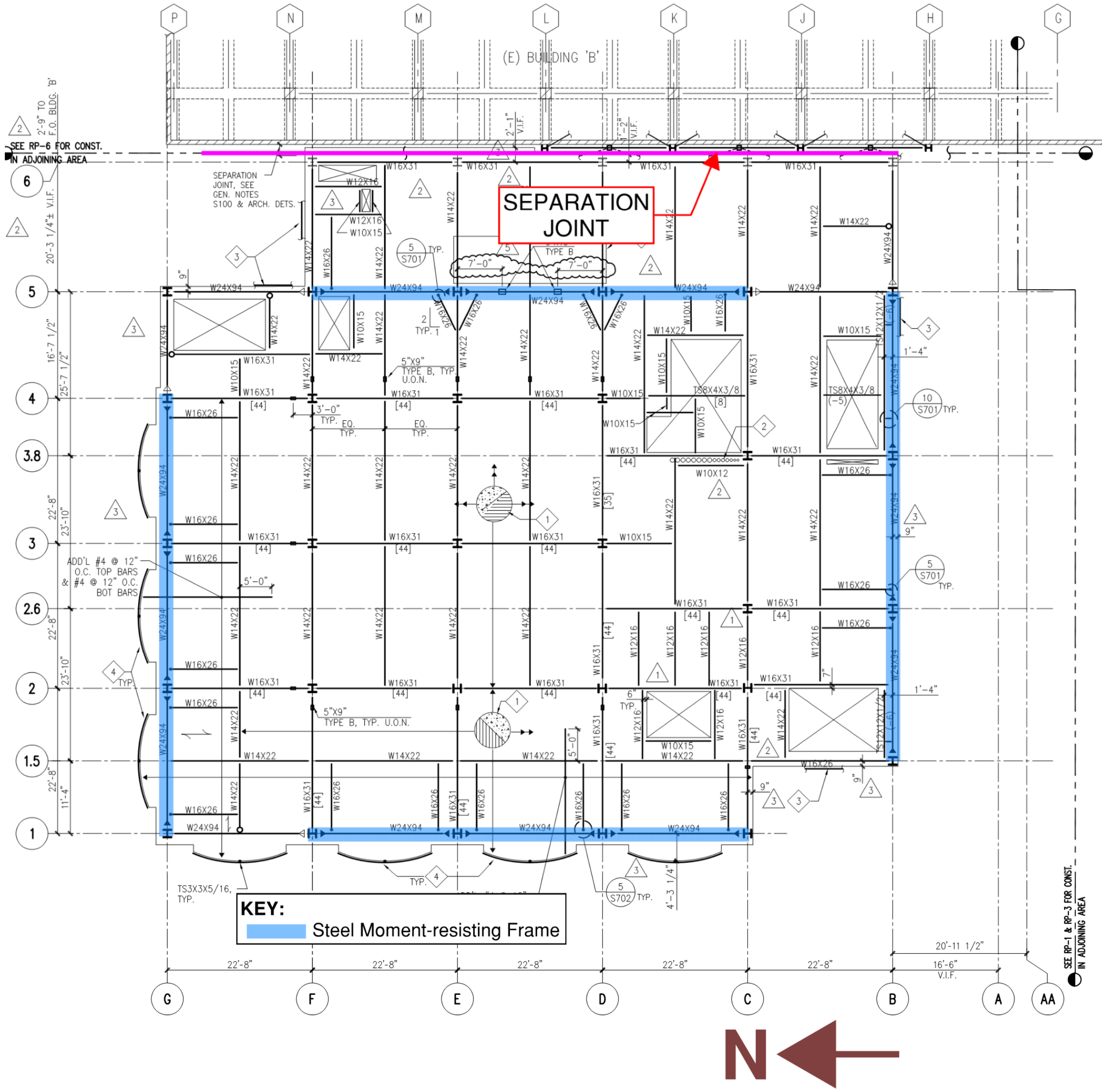
Lateral force-resisting system at the first floor



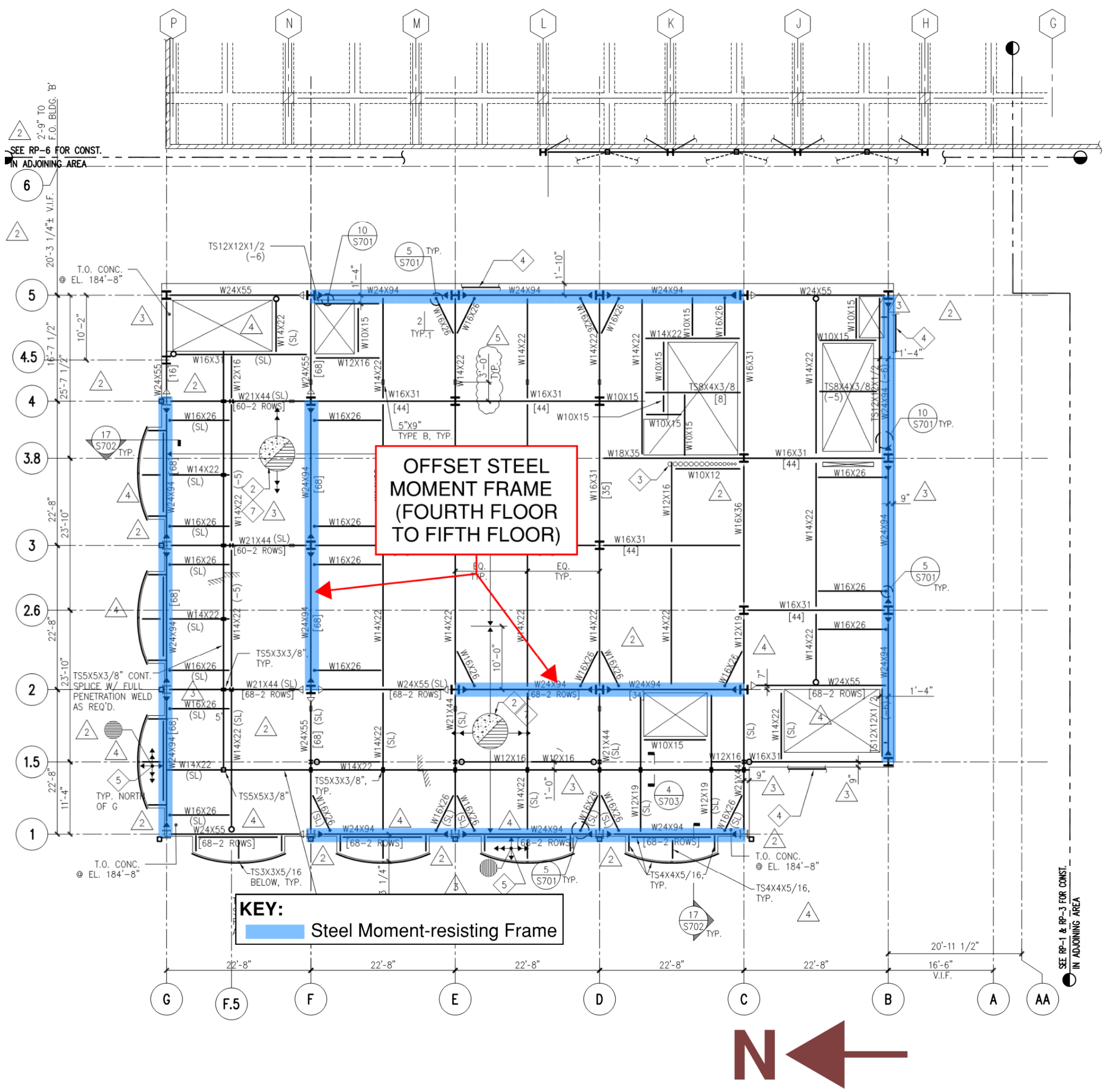
Lateral force-resisting system at the second floor



Lateral force-resisting system at the third floor

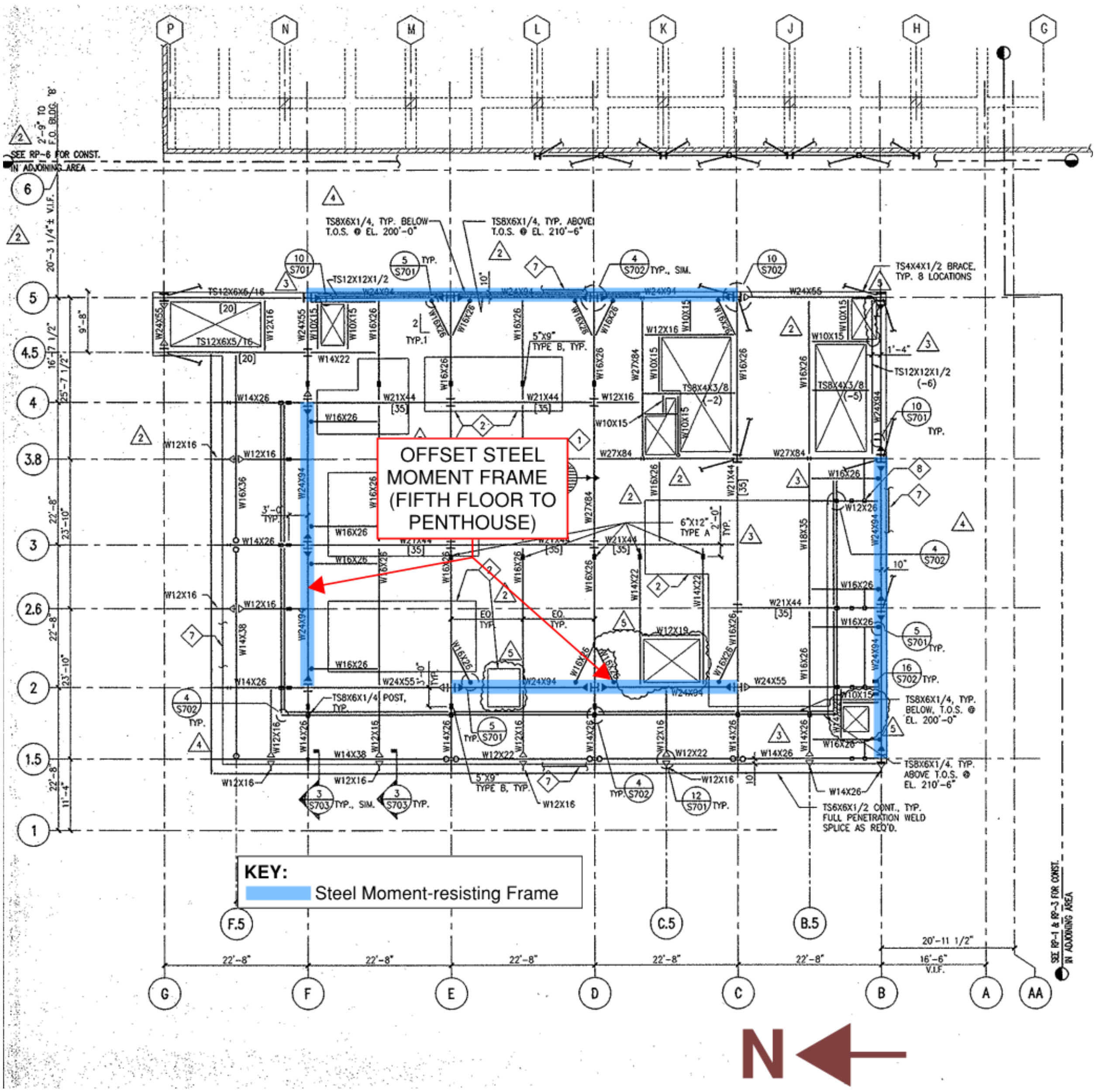


Lateral force-resisting system at the fourth floor

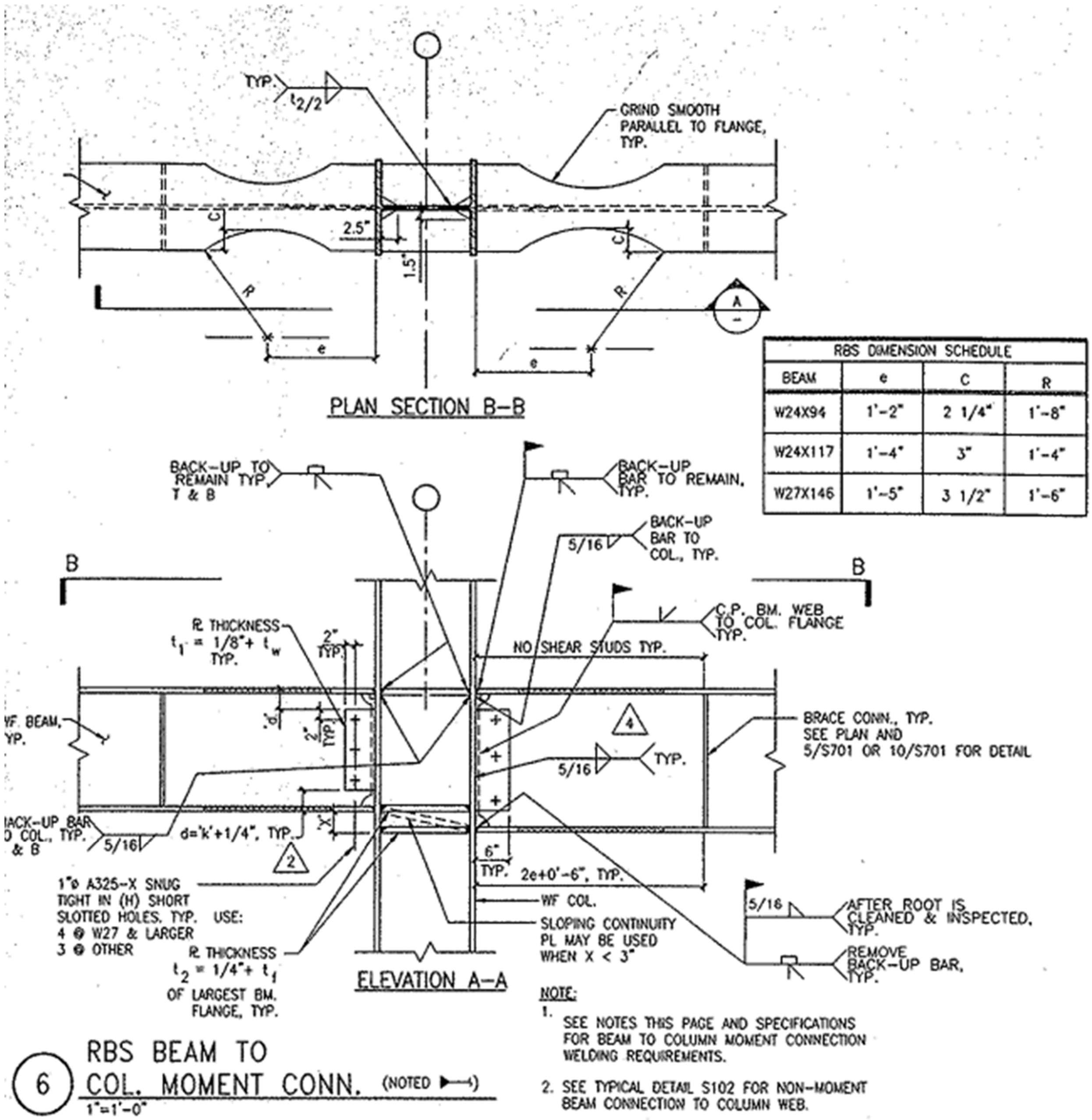


G

Lateral force-resisting system at the fifth floor



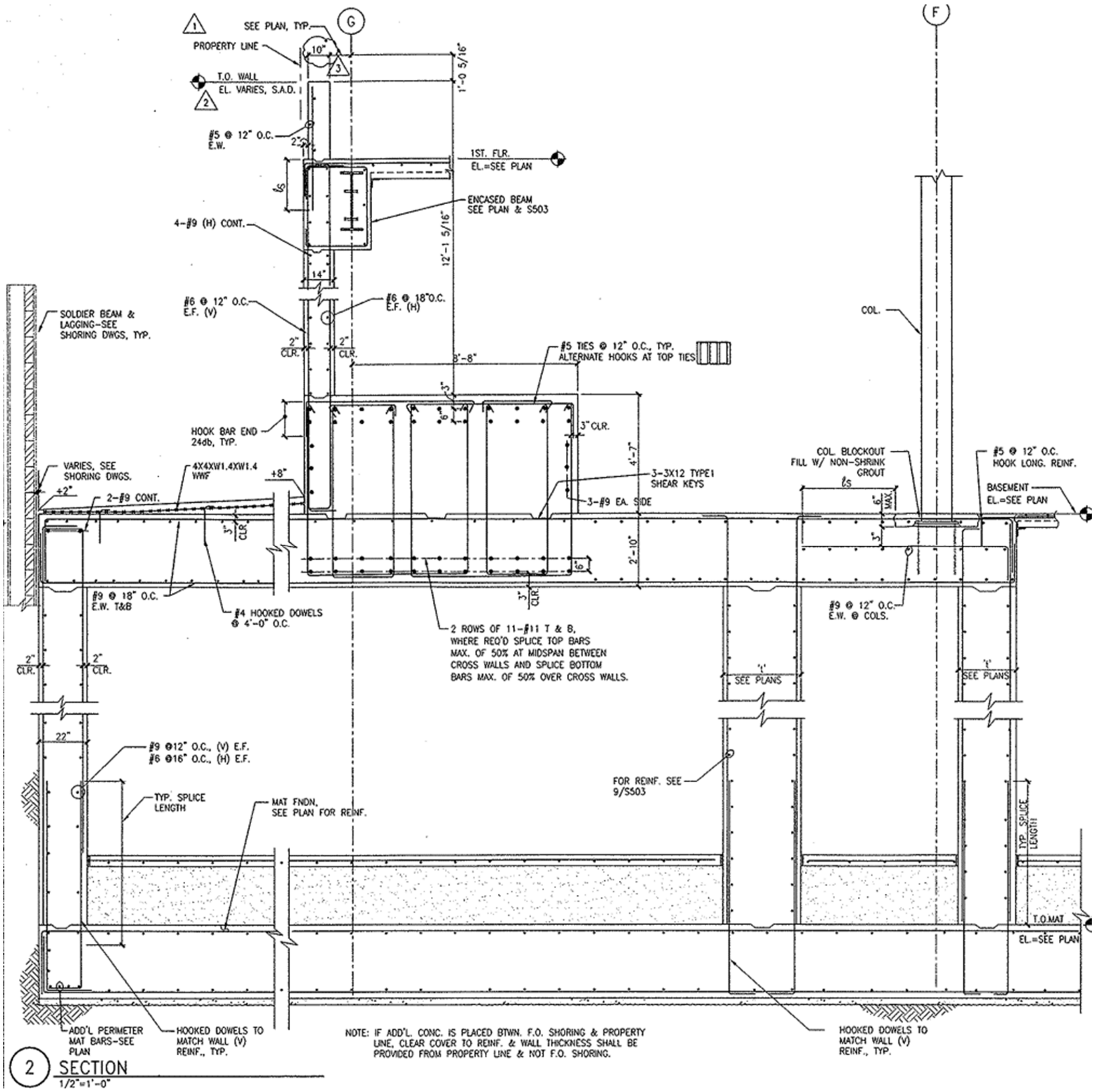
Lateral force-resisting system at the penthouse



RBS DIMENSION SCHEDULE

BEAM	e	C	R
W24X94	1'-2"	2 1/4"	1'-8"
W24X117	1'-4"	3"	1'-4"
W27X146	1'-5"	3 1/2"	1'-6"

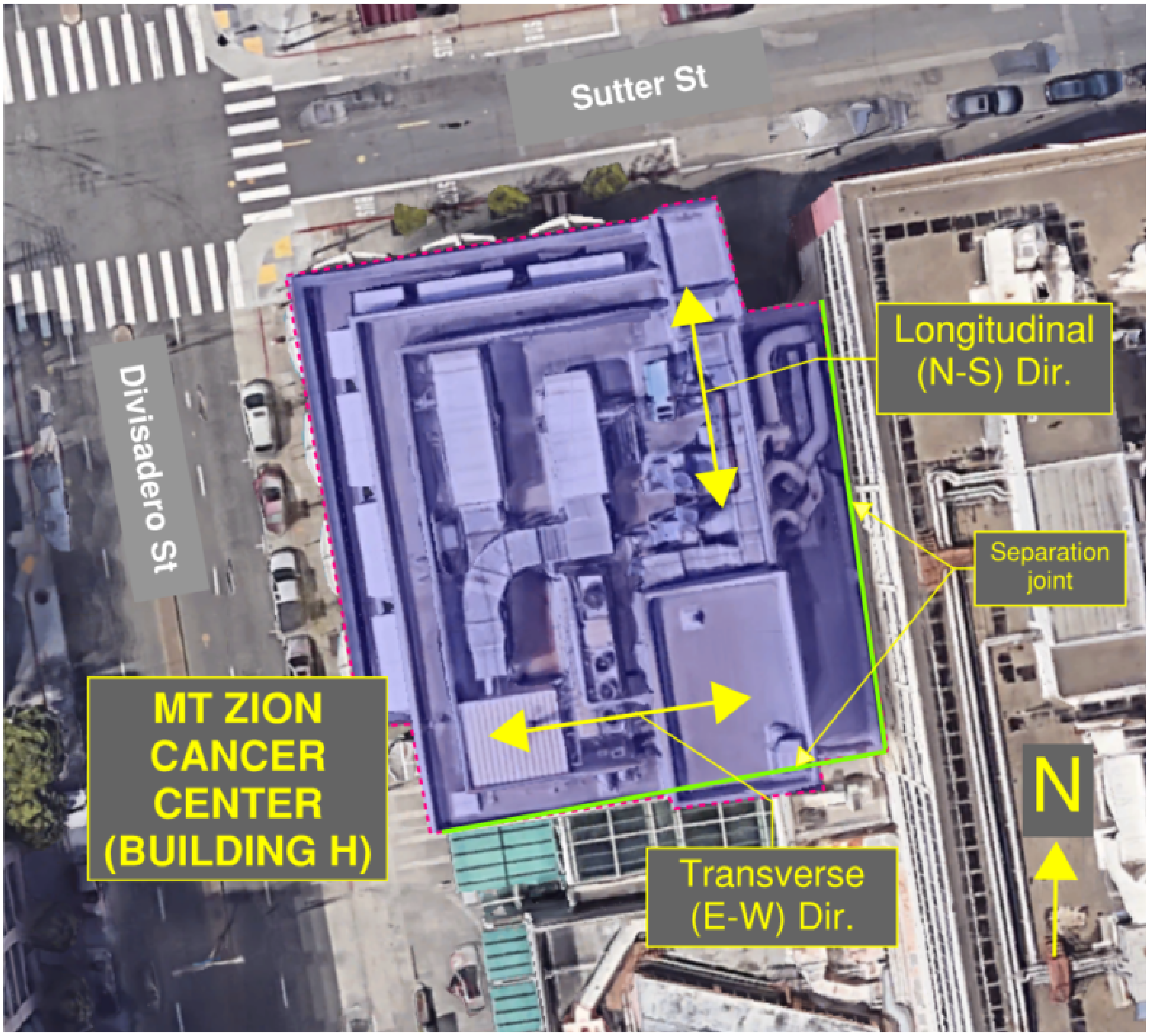
Typical reduced beam section (RBS) moment frame detail



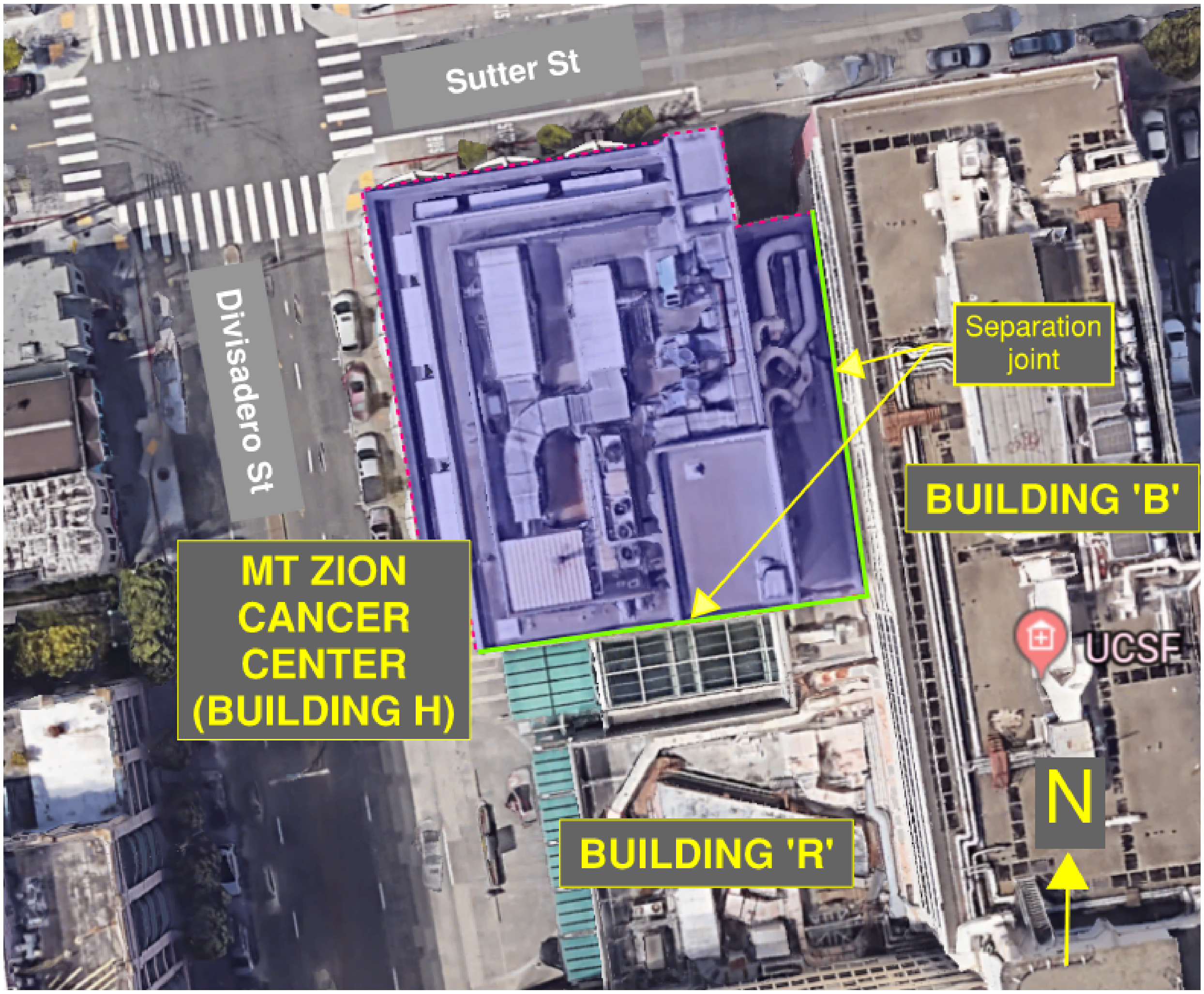
Section of the radiation room at the basement floor

APPENDIX A

Additional Images


Sutter St
Divisadero St
Longitudinal (N-S) Dir.
Separation joint
MT ZION CANCER CENTER (BUILDING H)
Transverse (E-W) Dir.
N

Plan

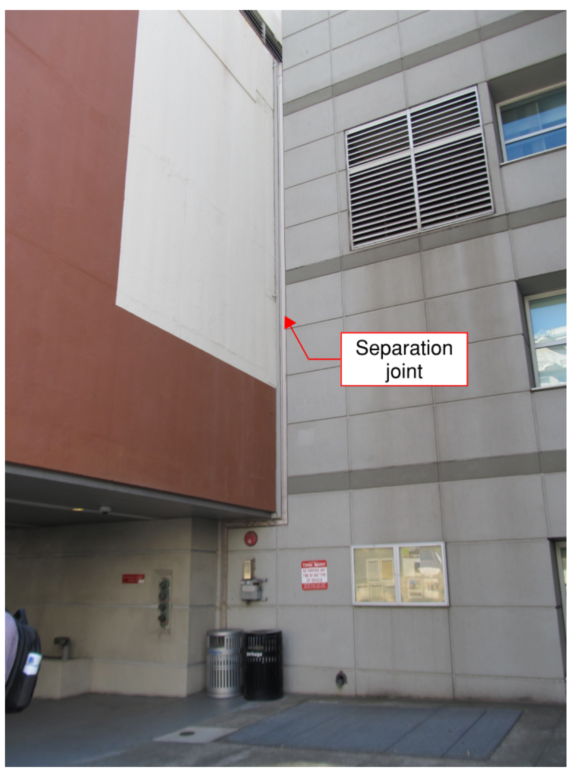


Adjacent buildings to the Mt. Zion Cancer Center

North elevation (looking south)

West elevation (looking east)

West and south elevation (looking northeast)



Separation joint between the Cancer Center and Building 'B' (looking southeast)

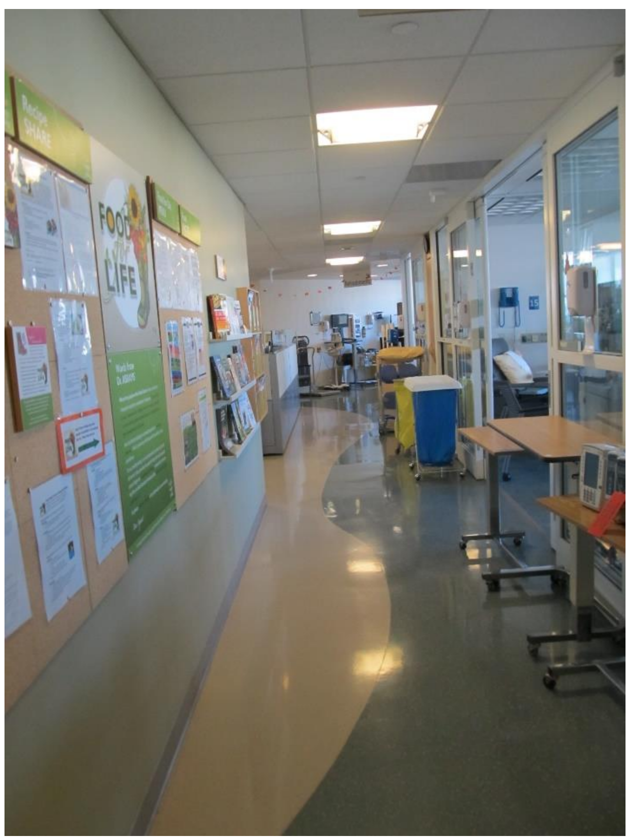
Separation joint between the Cancer Center and Building 'R' (looking east)

Equipment at the roof (looking south)

Extensive corrosion at the base of the roof equipment

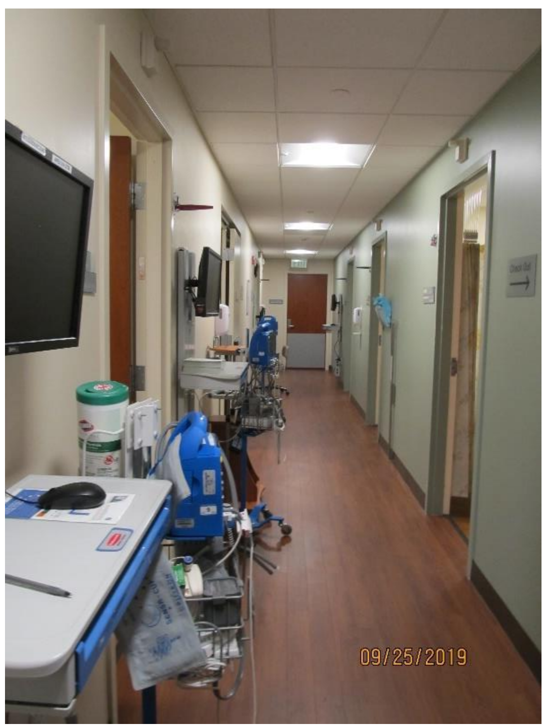
Corroding mechanical equipment at the roof

Balcony at the fifth floor (looking west)



Infusion center at the fifth floor (looking west)

Typical patient room

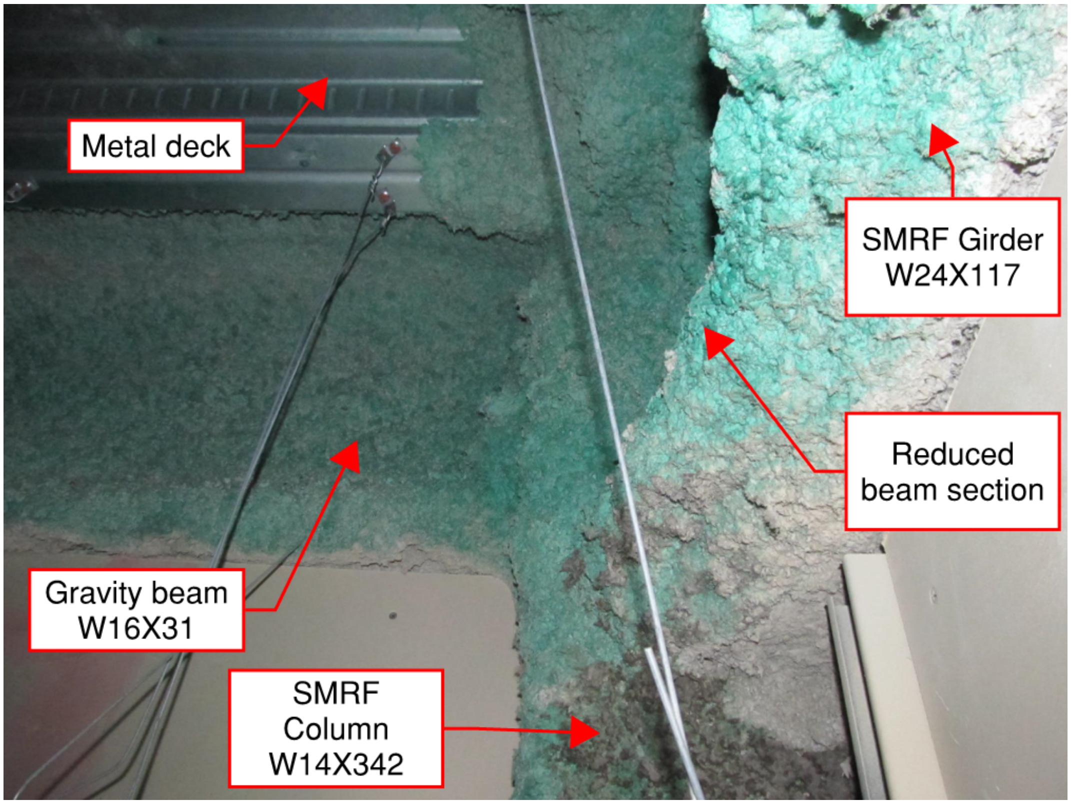


Typical interior corridor with patient rooms located on both sides (looking north)

Radiation treatment room in the basement (looking northeast)

Second floor patient waiting room that overlooks the atrium at the main entrance to Building B (looking east)

Office space at the first floor (looking north)



Reduced beam section moment frame connection with fireproofing located on the underside of the third floor (looking southwest with the RBS in the foreground)

Mechanical room at the mezzanine floor (looking northeast)

Electrical room at the mezzanine floor (looking northwest)

Diagonal steel framing encased in fireproofing provided for lateral bracing of the moment frame beam on the underside of the first floor (looking north)

Steel bracing at the mezzanine level
Building 'B' on the left and Cancer Center on the right (looking south)

Concrete spalling in void space between Building 'B' and the Cancer Center

Underside of steel plate stair stringers at the intermediate landing. No slip joint was observed at the floor level or intermediate landing.

APPENDIX B

ASCE 41-17 Tier 1 Checklists (Structural)

UC Campus:	San Francisco			Date:	11/18/2019		
Building CAAN:	3004	Auxiliary CAAN:		By Firm:	RUTHERFORD + CHEKENE		
Building Name:	UCSF Mt. Zion Cancer Center Building "H"			Initials:	EGM	Checked:	BL
Building Address:	1600 Divisadero, San Francisco, CA 94115			Page:	1	of	4

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:** Composite concrete over metal deck functions as floor diaphragms and delivers load to the steel moment-resisting frames. The lateral force-resisting system transitions from steel moment frames above the first floor to reinforced concrete shear walls below the first floor. The first floor slab acts as a transfer diaphragm.
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:** Buildings "B" and "R" are located to the east and south of the Cancer Center, respectively. The clear separations listed on Sheet S100 General Notes are reproduced below.

Floor	Adjacent Structures	
	Building "B"	Building "R"
Roof – Penthouse	N/A	N/A
Fifth floor	N/A	N/A
Fourth floor	6"	N/A
Third floor	4"	4"
Second floor	2"	2"
First floor	1"	1"

This above Table is related to the current floor naming convention and interpreted as follows:

Floor	Adjacent Structures		Required gap	Acceptance criteria
	Building "B"	Building "R"		
Fifth floor to Penthouse to Roof	N/A	N/A	N/A	N/A
Fourth to Fifth floor	N/A	N/A	N/A	N/A
Third to Fourth floor	6"	N/A	8.82	NG
Second to Third floor	4"	4"	6.48"	NG
First to Second floor	2"	2"	4.14	NG
Mezzanine to First Floor (see note)	1"	1"	1.98"	NG

Note: The basement slab of the adjacent structures aligns with the Mezzanine slab of the Cancer Center. The Cancer Center contains an additional story below the basement of Building R and B. The gap required is based upon an 11 ft story height from the First floor to the Mezzanine. The basement story is not considered.

It is also noted that stiff concrete shear walls comprise the lateral load-carrying system below the first floor. It is unlikely that the 1.5% drift predicted by this Tier 1 checklist would be required. It is unknown if the floor levels of the adjacent structures align.

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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:** There are no mezzanines present in the structure. It is noted that the occupants refer to one of the floors as the mezzanine level. However, this is a naming convention that was adopted after construction. The design drawing reference this level as the "basement," and the extent of the floor area is the same as the typical floors above.

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:** At the fifth floor, the two-bay moment frames located on Line 2 and Line F offset horizontally to Line 1 and Line G, respectively. As such, between the fourth to fifth floor there are 8 bays of frames in each direction. In the stories below the fourth floor, there are six bays of moment frames in each direction. The shear demand-to-capacity ratio between the third to fourth floor is 67% higher than the demand-to-capacity ratio between the fourth and fifth floor.
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:** At the fifth floor, the two-bay moment frames located on Line 2 and Line F offset horizontally to Line 1 and Line G, respectively. As such, between the fourth to fifth floor, there are 8 bays of frames in each direction. In the stories below the fourth floor, there are six bays of moment frames in each direction. The interstory drift between the third and fourth floor is twice the interstory drift between the fourth and fifth floor which indicates the story between the third to fourth floor is half as stiff as the story above.
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:** At the fifth floor, the two-bay moment frames located on Line 2 and Line F offset horizontally to Line 1 and Line G, respectively. At the first floor, the lateral force-resisting system transitions from steel moment-resisting frames to reinforced concrete shear walls. At this level, the lateral system offsets horizontally from Line B and Line 5 to Line A and Line 6, respectively. Finally, the shear wall located on Line G is discontinuous below the mezzanine slab.
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:** No horizontal offsets of more than 30% are present in the structure.

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C NC N/A U	
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 mass of adjacent stories changes by less than 20%.
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:** The building floor plan is approximately rectangular, and the steel moment frames and concrete shear walls are located around the perimeter of the structure.

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 "Table 1 - UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards" by Egan (2019), the liquefaction potential is very low.
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 "Table 1 - UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards" by Egan (2019), the site contains less than a 1-degree slope and is not susceptible to slope failure.
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 "Table 1 - UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards" by Egan (2019), the site is not susceptible to surface fault rupture.

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

FOUNDATION CONFIGURATION

	Description
C NC N/A U (●) C	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:** The building width is B = 104'-11" from Grid 1 to 6. The building height from the basement to the penthouse is H = 90'-6", B/H = 1.16 Sa = 0.98g for at BSE-2E 0.6x Sa = 0.59 B/H > 0.6 Sa.
C NC N/A U (●) C	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:** The soil is classified as Site Class D. However, the foundation consists of a 2'-10" thick concrete mat slab.

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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:** At the story between the fourth and fifth floor, there are 3 lines of moment frames in each direction. At the other stories, there are 2 lines of moment frames in each 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:** The ASCE 41 limit of 0.03 using the Quick Check procedure is exceeded in stories between second to third floor and third to fourth floor. In these stories, the drift ratios are 0.037 and 0.041, respectively. The drift ratios are compliant when checked per ASCE 7-16.
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:** The maximum axial stress due to overturning forces using the Quick Check is 8.2 ksi and takes place at the story between the first and second floor. The stress is less than the limit of $0.3F_y$ = 15 ksi.
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:** The highest average flexural stress in columns is 13.9 ksi at story between the first and second floor. The highest average flexural stress in beams is 20.2 ksi at story between the third and fourth floor. These values do not exceed the ASCE 41 limit of F_y = 50 ksi.

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:** Per Detail 2, 3, & 4 on Sheet S103, shear is transferred from the composite deck to the beams with welded shear studs located at 12" o.c. along the beam top flange. Per Detail 6 & * on Sheet/S701, collector beams are provided along the moment frame lines. They contain complete penetration welds at the top and bottom flanges.

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

LOW SEISMICITY

SEISMIC-FORCE-RESISTING SYSTEM

C NC N/A U	
☒ 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:** Per Detail 6/S702, the moment frame columns contain 20"x20" steel base plates that are anchored to the mat foundation with 4 -1.5" diameter rods with a 2'-0" embedment.

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:** At every story, there are typically 3 bays of moment frames per line in the E-W direction, and 3 bays of moment frames per line in the N-S direction.
☐ 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:** There are no concrete and masonry infill walls present.
☒ 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:**. The reduced beam section (RBS) moment connection specified on Det. 6/S701 is expected to develop the strength of the adjoining members based on the plastic capacity of the reduced section.

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Collapse Prevention Structural Checklist For Building Type S1-S1A

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	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:** The reduced beam section (RBS) moment connection specified on Det. 6/S701 is expected to develop the strength of the adjoining members based on the plastic capacity of the reduced section.
☐ 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:** Panel zones at interior joints in the moment-resisting frames are slightly overstressed and have maximum demand-to-capacity ratios = 1.10. The panel zones at the ends of moment-resisting frames are compliant and have maximum demand-to-capacity ratios = 0.55.
☒ 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:** As shown on Detail 3/S702, the column flanges are joined using complete penetration welds, and the webs are joined using partial penetration welds.
☒ 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:** All of the joints in structure are strong column-weak beam.
☒ 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:** All the frame elements conforming the seismic force-resisting system are at least moderately ductile members.

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Collapse Prevention Structural Checklist For Building Type S1-S1A

DIAPHRAGMS (STIFF OR FLEXIBLE)

C NC N/A U	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:** A stair and elevator opening are located adjacent to the moment frame on Line B. The combined length of these openings is approximately 36% of the frame length.

FLEXIBLE DIAPHRAGMS

C NC N/A U	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:** The building has rigid diaphragms.
☐ 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:** The building has rigid diaphragms.
☐ 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:** The building has rigid diaphragms.
☐ 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:** The building has rigid diaphragms.
☐ 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:** The building has rigid diaphragms.

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Collapse Prevention Structural Checklist For Building Type C2-C2A

Low And Moderate Seismicity

Seismic-Force-Resisting System

	Description
C ☐ NC ☒ N/A ☐ U ☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5.2.5.1) **Comments:** The wall located along Line A is gravity-load bearing and supports steel girders. Otherwise, the walls are not gravity load bearing as they contain concrete pilasters located at the inside face of the wall.
C ☒ NC ☐ N/A ☐ U ☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1) **Comments:** Shear walls are located around the perimeter of the building. Between the mezzanine and the first floor, there are two lines of the wall in each direction. Between the basement and the mezzanine, there are six lines of wall in each direction. The four interior walls added in this story serve as shielding for the equipment utilizing radiation that is located in this story.
C ☒ NC ☐ N/A ☐ U ☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1) **Comments:** The calculated wall stresses do not exceed the ASCE 41 limit of 126 psi for f'c = 4,000 psi at any story. The average shear stresses in the north-south direction are 27 psi (basement floor to the mezzanine floor) and 40 psi (mezzanine floor to the first floor). The average shear stresses in the east-west direction are 21 psi (basement floor to the mezzanine floor) and 47 psi (mezzanine floor to the first floor).
C ☒ NC ☐ N/A ☐ U ☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3) **Comments:** Section 1 & 2/S502 show the following typical reinforcing for the exterior concrete walls: - For 14" thick walls: #6 at 18" o.c. e.f. horizontal (ρ = 0.0035), #6 at 12" o.c. e.f. vertical (ρ = 0.0052). - 18" thick walls: #6 at 18" o.c. e.f. horizontal (ρ = 0.0027), and one layer of #8 and one layer of #9 at 12" o.c. vertical (ρ = 0.008). - For 22" thick walls: #6 at 16" o.c. e.f. horizontal (ρ = 0.0025), and #9 at 12" o.c. e.f. vertical (ρ = 0.0075). The linear accelerator vault wall reinforcement is specified on Det. 16/S503 as shown below:

Wall Thickness	Maximum Thickness	Vert. Reinf.	Minimum ρ_{vert}	Horiz. Reinf.	Minimum ρ_{horiz}
4" - 8"	8"	#6 at 12"	0.0046	#6 at 16"	0.0034
9" - 14"	14"	#4 at 12" E.F.	0.0024	#5 at 16" E.F.	0.0028
15" to 18"	18"	#5 at 12" E.F.	0.0030	#6 at 18" E.F.	0.0027
19" to 26"	26"	#6 at 16" E.F.	0.0021	#7 at 18" E.F.	0.0026
27" to 34"	34"	#7 at 18" E.F.	0.0020	#8 at 18" E.F.	0.0026
35" to 44"	44"	#8 at 18" E.F.	0.0020	#9 at 18" E.F.	0.0025
45" to 66"	66"	#8 at 12" E.F.	0.0020	#9 at 12" E.F.	0.0025

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Collapse Prevention Structural Checklist For Building Type C2-C2A

Connections

	Description
C NC N/A U	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms 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 have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1) **Comments:** The building has rigid diaphragms.
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2) **Comments:** The wall sections on Sheets S501 and S502 show the longitudinal slab bars are typically hooked at the back of the concrete walls.
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing directly above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4) **Comments:** The wall sections on Sheets S501 and S502 show hooked dowels embedded into the mat foundation that splice with the wall vertical reinforcing. The dowel size and spacing is to match wall vertical reinforcement.

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	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2) **Comments:** The building columns are steel wide flange sections with compact flanges and webs, per AISC 341-16. The following column sections comprise the lateral force-resisting system:

Column Section	$b_f/2t_f$	h/t_w
W14X233	4.62	10.7
W14X311	3.59	8.09
W14X342	3.31	7.41

The above values are less than the limit for b/2t and h/t. The limit for b/t is 9.2, and the limit for h/t varies with the column axial between the values of 49.2 to 85.2.

	Description
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3) **Comments:** The building does not contain flat slabs.

C / NC / N/A / U	Description
N/A	COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1) **Comments:** The concrete shear walls do contain coupling beams.

Diaphragms (Stiff Or Flexible)

C / NC / N/A / U	Description
C	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1) **Comments:** There are no split-level diaphragms within the structure.
NC	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3) **Comments:** A slab opening is located adjacent to the wall on Line 1 at the mezzanine level. It measures 39'-2" long and comprises approximately 30% of the total wall length.

Flexible Diaphragms

C / NC / N/A / U	Description
N/A	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) **Comments:** The building has rigid diaphragms.
N/A	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:** The building has rigid diaphragms.
N/A	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:** The building has rigid diaphragms.
N/A	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:** The building has rigid diaphragms.

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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:** The building has rigid diaphragms.

Connections

	Description
C NC N/A U ☐ ☐ ☒ ☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5) **Comments:** The building is supported on a mat foundation.

APPENDIX C

UCOP Seismic Safety Policy Falling Hazards Assessment Summary

UC Campus:	San Francisco		Date:	11/18/2019		
Building CAAN:	3004	Auxiliary CAAN:	By Firm:	Rutherford+Chekene		
Building Name:	UCSF Mt. Zion Cancer Center Building "H"		Initials:	*EGM*	Checked:	*BL*
Building Address:	1600 Divisadero St, San Francisco, CA 94115		Page:	1	of	1

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:** No areas of congregation of over 50 people are located within the building.
P ☒ N/A ☐	**Heavy masonry or stone veneer above exit ways or public access areas** **Comments:** Brick veneer is located on the exterior of the structure on the north and west elevation.
P ☐ N/A ☒	**Unbraced masonry parapets, cornices, or other ornamentation above exit ways or public access areas** **Comments:** There are no masonry parapets, cornices, or other ornamentation.
P ☒ N/A ☐	**Unrestrained hazardous material storage** **Comments:** Compressed gas storage is located within the structure. It is unknown if these items are braced.
P ☐ N/A ☒	**Masonry chimneys** **Comments:** No masonry chimneys are in the building.
P ☒ N/A ☐	**Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.** **Comments:** The structure is supplied natural gas from the adjacent Building A. Gas is piped to the roof to supply the boiler room. Emergency shut off valves were observed. Bracing of the supply line is unknown.
P ☒ N/A ☐	**Other:** **Comments:** The egress stairs are constructed with steel plate stringers. No movement joints were observed in at the floor levels or intermediate landings. The General Notes, Sheet S100, indicate the interstory drift is 2". With this drift, it is likely that forces will be induced in the stair stringers and their connections that exceed their capacity.
P ☐ N/A ☐	**Other:** **Comments:**
P ☐ N/A ☐	**Other:** **Comments:**

Falling Hazards Risk: Low

Note: P= Present, N/A = Not Applicable

APPENDIX D

Quick Check Calculations

Flat Load Tables

STAIR & ELEVATOR ROOM ROOF	Seismic Weight psf	Dead Load psf	Remarks
Roofing, waterproofing, and insulation	10	10	Asphalt built-up roofing (BUR)
Metal deck	2	2	1 1/2" 18GA metal roof deck
Beams/girders	7	7	Steel beams below metal deck
MEP	3	3	MEP hung from underside of roof slab
Lighting, and misc.	2	2	Lighting and misc. hung from underside of roof slab
Columns	8	0	Wide flange steel columns
Partitions	5	0	
Total	36	24	

1 - The flat load is a metal deck assembly that takes place above the stairs and elevator at the roof between Grids B-C.5/3.8-5 and F-G/4.5-5.

2 - The stair roof on the northeast corner slopes down toward the North.

3 - This flat load includes weight of (9) steel columns below floor in a 1,400 ft^2 area. Column trib. height is 6'-6".

ELEVATOR SHEAVE FLOOR	Seismic Weight psf	Dead Load psf	Remarks
Elevator equipment	10	10	Sheaves, counterweights, and elevator car
Composite deck	60	60	4 1/2" LWC fill over 3" 18GA metal deck
Beams/girders	13	13	Steel beams below metal deck
MEP	3	3	MEP hung from underside of floor slab
Lighting, and misc.	2	2	Lighting and misc. hung from underside of floor slab
Columns	0	0	Wide flange steel columns
Partitions	0	0	
Total	87	87	

1 - The flat load is a composite slab assembly that takes place above the elevator between Grids B-C.5/3.8-5. It is situated between the penthouse and the roof.

2 - The column weight is distributed between the penthouse and the roof flat loads.

3 - LW concrete unit weight of 115 psf is assumed.

PENTHOUSE	Seismic Weight psf	Dead Load psf	Remarks
Mechanical equipment	25	50	Estimated equipment weight
Concrete pads	8	8	4" thick LWC pads below heavy mechanical equipment
Roofing, waterproofing, and insulation	10	10	Asphalt built-up roofing (BUR)
Composite deck	48	48	3 1/4" LWC fill over 3" 18GA metal deck
Beams/girders	8	8	Steel beams below metal deck
MEP	7	7	MEP hung from underside of slab
Ceiling, lighting, and misc.	5	5	Lay-in ceiling, lighting, and misc. hung from underside of slab
Columns	5	0	Wide flange steel columns
Partitions	5	0	
Total	121	135	

1 - The flat load is a composite slab assembly that takes place at entire roof of the structure (named the penthouse on the 1998 drawings) between Grids B-G/1.5-5.

2 - The equipment where it is located is assumed to weigh 50 psf. The equipment is located on approximately 1/2 of the room area and therefore, 25 psf is assumed for seismic mass.

3 - 4" thick lightweight concrete pads are assumed below heavy mechanical equipment and takes place in 20% the penthouse plan area.

4 - This flat load includes weight of (29) steel columns below and (9) steel columns above floor in a 8,227 ft^2 area. Column trib. height is 13'-0".

5 - LW concrete unit weight of 115 psf is assumed.

BALCONY	Seismic Weight psf	Dead Load psf	Remarks
Planters	40	40	Planters with saturated soil
Concrete pavers	50	50	4" thick NWC pavers
Waterproofing and insulation	5	5	
Composite deck	48	48	3 1/4" LWC fill over 3" 18GA metal deck
Beams/girders	17	17	Steel beams below metal deck
MEP	7	7	MEP hung from underside of slab
Ceiling, lighting, and misc.	5	5	Lay-in ceiling, lighting, and misc. hung from underside of slab
Columns	11	0	Wide flange and tube shape steel columns
Partitions	5	0	
Total	188	172	

1 - The flat load is a composite slab assembly that takes place at the fifth floor between Grids C-G/1-1.5 and F.5-G/1-4.5.

2 - Twelve 36" wide x 60" long x 30" tall and 2.5" thick concrete boxes containing 20" of saturated soil are smeared over the balcony area. A satuarted soil weight of 125 pcf is used.

3 - This flat load includes weight of (2) steel columns below floor in a 227 ft^2 area. Column trib. height is 6'-6".

4 - LW concrete unit weight of 115 psf is assumed.

	Seismic Weight	Dead Load	
TYPICAL FLOOR	**psf**	**psf**	**Remarks**
Flooring	5	5	Carpet and vinyl composition tiles
Composite deck	48	48	3 1/4" LWC fill over 3" 18GA metal deck
Beams/girders	7	7	Steel beams below metal deck
MEP	7	7	MEP hung from underside of floor slab
Ceiling, lighting, and misc.	5	5	Lay-in ceiling, lighting, and misc. hung from underside of floor slab
Columns	9	9	Wide flange and tube shape steel columns
Partitions	10	10	
Total	90	90	

1 - The flat load is a composite slab assembly that takes place at the mezzanine between Grids B-G/1-5, from second to fourth floor between Grids A-G/1-6 and at the fifth floor between Grids B-G/1.5-5.

2 - This flat load includes weight of (45) steel columns below and (38) steel columns above floor in a 13,346 ft^2 area. Column trib. height is 13'-0".

3 - LW concrete unit weight of 115 psf is assumed.

4 - The steel girders conforming the SMRF on the underside of the first floor are encased in concrete, per Det. 11/S503. However, this condition is not typical in other floors.

	Seismic Weight	Dead Load	
THICKENED COMPOSITE DECK	**psf**	**psf**	**Remarks**
Flooring	5	5	Carpet and vinyl composition tiles
Composite deck	67	67	5 1/4" LWC fill over 3" 18GA metal deck
Beams/girders	5	5	Steel beams below metal deck
MEP	7	7	MEP hung from underside of floor slab
Ceiling, lighting, and misc.	5	5	Lay-in ceiling, lighting, and misc. hung from underside of floor slab
Columns	8	8	Wide flange and tube shape steel columns
Partitions	10	10	
Total	107	107	

1 - The flat load is a composite slab assembly that takes place at the first floor between Grid AA-BB/1-5 and A-F/5-6.

2 - This flat load includes weight of (20) steel columns and (16) embedded concrete pilasters below and (45) steel columns above floor in a 14,603 ft^2 area. Column trib. height is 12'-0".

3 - LW concrete unit weight of 115 psf is assumed.

4 - The thickness of the concrete fill for the composite deck varies from 3 1/4" to 7' 1/4"; however, a thickness of 5 1/4" is considered the most representative of this area.

	Seismic Weight	Dead Load	
RADIATION ONCOLOGY SLAB	**psf**	**psf**	**Remarks**
Topping slab, and flooring	24	24	NWC topping slab, and vinyl composition tile flooring
Slab	425	425	2'-10" NWC slab
Beams/girders	0	0	
MEP	7	7	MEP hung from underside of floor slab
Ceiling, lighting, and misc.	5	5	Lay-in ceiling, lighting, and misc. hung from underside of floor slab
Columns	7	7	
Partitions	10	10	
Total	478	478	

1 - The flat load is a reinforced concrete slab assembly that takes place at the mezzanine between Grids F-G.5/1-6.

2 - The slab thickness is shown on Det. 2/S501 and Det. 3/S502 in the 1998 strurctural drawings.

3 - This flat load includes weight of (7) embedded concrete pilasters below and above floor in a 3,984 ft^2 area. Column trib. height is 12'-9".

4 - One-third of the the area has a topping slab with varying thickness, as specified on Sheet S202 and Det. 2/S501; the remaining part consists of VCT flooring.

5 - The concrete slab is directly supported by concrete walls and embedded pilasters.

Story Weight

Structure above ground

$\gamma_{cladding}$ = 35 psf

Floor Levels	Floor Area (ft²)[1,2] STAIR & ELEVATOR ROOM ROOF	ELEVATOR SHEAVE FLOOR	PENTHOUSE	BALCONY	TYPICAL FLOOR	Floor Weight (psf) STAIR & ELEVATOR ROOM ROOF	ELEVATOR SHEAVE FLOOR	PENTHOUSE	BALCONY	TYPICAL FLOOR	Height Elevation (ft)	Height below floor level (ft)	Exterior Wall and Glass Weight [3] Length below floor level (ft)	Ext Wall & Glass Seismic Weight (kips)	Additional Weight (kips) [4]	Total Seismic Weight (kips)
													207.5			
Penthouse Floor & Roof	1,400	1,081	8,227	0	0	36	87	121	188	90	198.50	13.00	408.1	93	36	1,268
Fifth Floor	0	0	0	1,581	8,138	36	87	121	188	90	185.50	13.00	371.8	177		1,207
Fourth Floor	0	0	0	0	13,346	36	87	121	188	90	172.50	13.00	371.8	169		1,371
Third Floor	0	0	0	0	13,346	36	87	121	188	90	159.50	13.00	371.8	169		1,371
Second Floor	0	0	0	0	13,346	36	87	121	188	90	146.50	13.00	363.3	167		1,369
First Floor											133.50					

6,584 kips

Structure below ground

wconcrete = 150 pcf

Floor Levels	Floor Area (ft²)[1] TYPICAL FLOOR	THICKENED COMPOSITE DECK	RADIATION ONCOLOGY SLAB	Floor Weight (psf) TYPICAL FLOOR	THICKENED COMPOSITE DECK	RADIATION ONCOLOGY SLAB	Height Elevation (ft)	Height below floor level (ft)	Wall Weight [5,6] Wall height tributary to each floor level (ft)	Wall Area below (ft²)	Wall Weight below (kips)	Wall Seismic Weight (kips)	Total Seismic Weight (kips)
First Floor	10,561	4,042	0	90	107	478	133.50	11.00	5.50	540	892	446	1,922
Mezzanine	10,775	0	3,984	90	107	478	122.50	14.50	12.75	1,299	2,826	1,859	4,173
Basement							108.00		7.25				

6,095 kips

Notes:
1 - The seismic base is set at the first floor.
2 - The penthouse, elevator sheave floor, and roof are lumped together for seismic weight calculation. Roof areas only take place above stairs and elevator.
3 - The exterior cladding is comprised of the following assemblies:

Exterior Walls
40 psf (120 pcf) - brick
4 psf - dens glass
3 psf - metal studs
3 psf - 5/8" gypboard
Σ = 50 psf

Windows
10 psf - glass
Σ = 10 psf

In a typical floor, the exterior walls constitute approximately 62% of the exterior area, and the remaining 38% consists of glass windows. Thus, 35 psf is a representative weight for the exterior cladding of the building.
4 - The additional weight considers the screen wall on the penthouse covering the mecanical equipment. Assumptions include 187 linear feet for a 13-ft high walls considering 15 psf.

5 - The wall weight includes area of exterior and interior concrete walls below ground.
6 - A sample calculation for the wall seismic weight at the mezzanine is provided below:

Wall ID	Thickness (in)	Length (ft)	Concrete/Total area *	Area (ft^2)
LB - 1X	18	141.0	1.00	211.5
LB - 2X	33	5.0	1.00	13.8
LB - 3X	60	11.8	1.00	58.8
LB - 4X	33	9.0	1.00	24.8
LB - 5X	18	9.8	1.00	14.6
LB - 6X	33	5.0	1.00	13.8
LB - 7X	60	10.5	1.00	52.5
LB - 8X	33	16.3	1.00	44.7
LB - 9X	33	5.0	1.00	13.8
LB - 10X	60	11.8	1.00	58.8
LB - 11X	33	9.0	1.00	24.8
LB - 12X	18	9.8	1.00	14.6
LB - 13X	12	5.0	1.00	5.0
LB - 14X	12	14.0	1.00	14.0
LB - 15X	12	5.0	1.00	5.0
LB - 1Y	22	106.3	1.00	194.8
LB - 2Y	12	5.0	1.00	5.0
LB - 3Y	36	23.5	1.00	70.5
LB - 4Y	12	6.3	1.00	6.3
LB - 5Y	12	6.0	1.00	6.0
LB - 6Y	36	26.0	1.00	78.0
LB - 7Y	12	7.0	1.00	7.0
LB - 8Y	27	13.667	1.00	30.8
LB - 9Y	12	4.75	1.00	4.8
LB - 10Y	12	5.4167	1.00	5.4
LB - 11Y	27	30.75	1.00	69.2
LB - 12Y	12	7.25	1.00	7.3
LB - 13Y	12	5.8333	1.00	5.8
LB - 14Y	27	13.75	1.00	30.9
LB - 15Y	12	13.5	1.00	13.5
LB - 16Y	12	18	1.00	18.0
LB - 17Y	18	84.25	1.00	126.4
LB - 18Y	27	22	1.00	49.5
			Σ =	1299.2

Wall ID	Thickness (in)	Length (ft)	Concrete/Total area *	Area (ft^2)
LM - 1X	14	131.75	1.00	153.7
LM - 2X	14	7.25	1.00	8.5
LM - 3X	14	113	1.00	131.8
LM - 1Y	14	85	1.00	99.2
LM - 2Y	14	20.25	1.00	23.6
LM - 3Y	14	106	1.00	123.7
			Σ =	540.5

*Solid / Total area factor accounts for percentage of wall that is solid compared to the total area including openings.

Wall height above = 11.00 ft
Wall height below = 14.50 ft

Wall area above = 540.5 ft^2
Wall area below = 1299.2 ft^2

$w_{concrete}$ = 0.15 kcf

$$Wall\ seismic\ weight\ = w_{concrete} \times \left(Area_{belox} \times \frac{Height_{belox}}{2} + Area_{above} \times \frac{Height_{above}}{2} \right)$$

Wall seismic weight = 1859 kips

6 - The floors have been renamed as follows:

Elevation	1998 Drawings	Current Name
198'-6"	Penthouse	Penthouse
185-6"	Fifth floor	Fifth floor
172'-6"	Fourth floor	Fourth floor
159'-6"	Third floor	Third floor
146'-6"	Second floor	Second floor
133'-6"	First floor	First floor
122'-6"	Basement floor	Mezzanine
108'-0"	Sub-basement floor	Basement floor

Period of the Superstructure

C_t=	0.035
h_n (ft)=	65.00
B=	0.8

T=	0.99	sec

Notes:

1- The period is calculated per ASCE 41-17 Equation 4-4.

$$T = C_t \cdot h_n^{B}$$

2- Ct and B are for "moment-resisting frame systems of steel" per ASCE 41-17 Section 4.4.2.4.

3- The building height is taken from the first floor to the penthouse floor.

where

T = Fundamental period (s) in the direction under consideration;

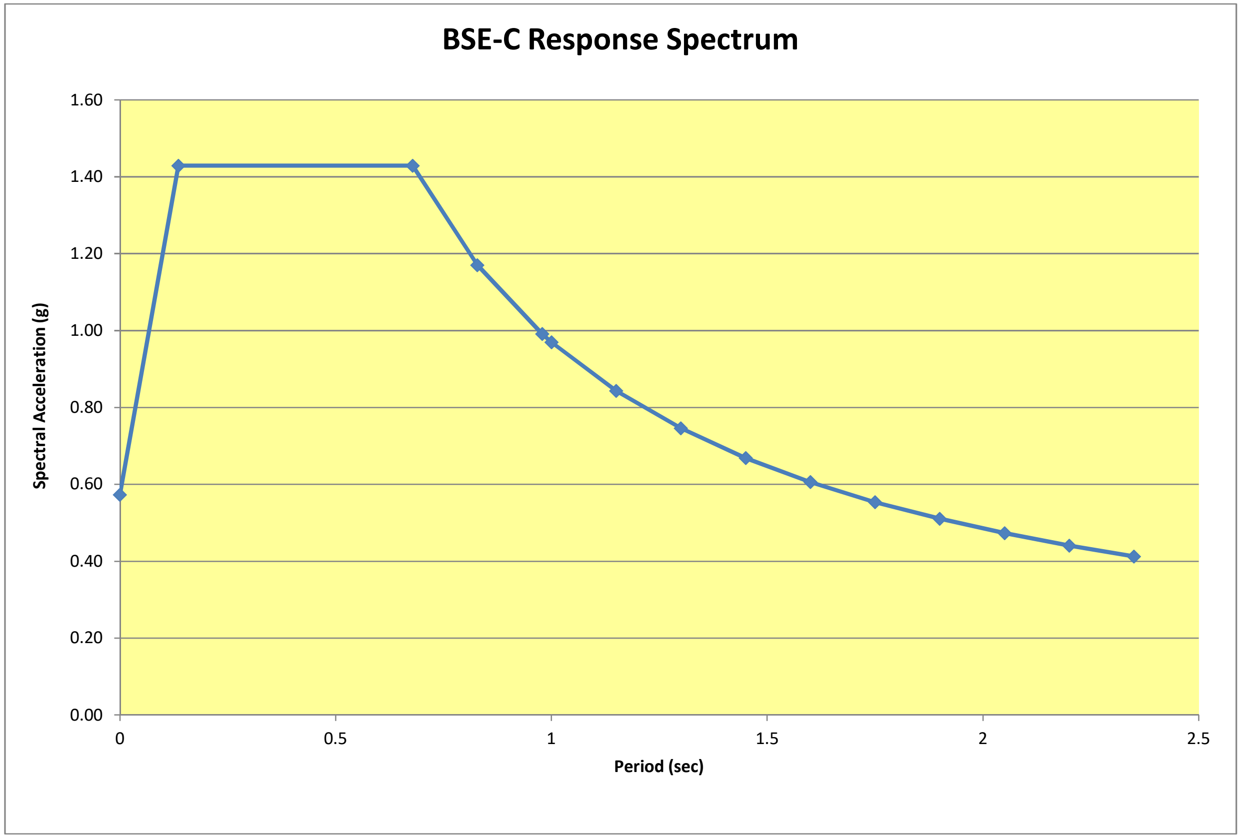
C_t = 0.035 for moment-resisting frame systems of steel (Building Types S1 and S1a);
= 0.018 for moment-resisting frames of reinforced concrete (Building Type C1);
= 0.030 for eccentrically braced steel frames (Building Types S2 and S2a);
= 0.020 for all other framing systems;

h_n = Height (ft) above the base to the roof level;

β = 0.80 for moment-resisting frame systems of steel (Building Types S1 and S1a);
= 0.90 for moment-resisting frame systems of reinforced concrete (Building Type C1); and
= 0.75 for all other framing systems.

Site Parameters

Period (s)	Sa (g)
0	0.57
0.14	1.43
0.68	1.43
0.83	1.17
0.98	0.99
1.00	0.97
1.15	0.84
1.30	0.75
1.45	0.67
1.60	0.61
1.75	0.55
1.90	0.51
2.05	0.47
2.20	0.44
2.35	0.41



BSE-C		
β =	0.05	
B_1 =	1.00	
S_S =	1.433	g
S_1 =	0.558	g
F_a =	1.000	g
F_v =	1.742	g
Site Class =	D	
S_{CS} =	1.433	g
S_{C1} =	0.972	g
T_0 =	0.14	s
T_s =	0.68	s
T =	0.99	s
S_a =	0.98	g (See Note 2)
Tier 1 S_a =	**0.98**	g (See Note 3)

Notes:

1- Spectral accelerations based upon site class provided in "Table 1- UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards". The procedure as specified in ASCE 41-17, Section 2.4.1.7 is used to develop General Response Spectrum shown above.

2 - Per Section 2.4.1.7 of ASCE 41-17, use of spectral response acceleration in the extreme short-period range ($T < T_0$) shall only be permitted in dynamic analysis procedures and only for modes other than the fundamental mode.

3- Per Section 4.4.2.3 for Tier 1 screening in ASCE 41-17, the spectral acceleration, Sa, is computed as the least value of S_{X1}/T, and S_{XS}.

Seismic Force Distribution

Horizontal Response Spectrum Seismic Parameters		
Hazard Level	BSE-C	
Site Class	D	
S_{CS}=	1.43 g	(See Note 2)
S_{C1}=	0.97 g	(See Note 2)

T=	0.99	s	
Sa=	0.98	g	(See Note 3)
W=	6,584	kips	
C=	1.0	Per ASCE 41-17 Table 4-7	

V=	6,483	kips

k= 1.24 Per ASCE 41-17 Section 4.4.2.2, K = 1.0 for periods less than 0.5 sec and K = 2.0 for T >2.5 sec. It varies linearly in between 0.5 sec and 2.5 sec period.

Structure above ground

Floor Levels	Story Height	Total Height, H	Weight, W	W x H^k	coeff	Fx	Story Shear, V
	(ft)	(ft)	(kips)			(kips)	(kips)
Penthouse Floor & Roof	13.00	65.00	1,268	227,822	0.36	2,327	2,327
Fifth Floor	13.00	52.00	1,207	164,282	0.26	1,678	4,006
Fourth Floor	13.00	39.00	1,371	130,485	0.21	1,333	5,339
Third Floor	13.00	26.00	1,371	78,809	0.12	805	6,144
Second Floor	13.00	13.00	1,369	33,236	0.05	340	6,483
First Floor							
Σ =	65.0		6,584	634,633	1	6,483	

Structure below ground

Floor Levels	Weight, W	PGA	Fx, Substructure	Fx, Superstructure	Story Shear, V
	(kips)	(g)	(kips)	(kips)	(kips)
First Floor	1922	0.57	1,102	6,483	7,585
Mezzanine	4173	0.57	2,392	-	9,977
Basement					

Notes:

1- The superstructure is taken to be from the first floor to the penthouse. A linear distribution is assumed in the superstructure per ASCE 41-17, Section 4.4.2.2.

2- The substructure is taken to be from the first floor to the basement. A uniform force distribution is assumed below grade. At each floor level, the mass is multiplied by the peak ground acceleration. The base shear from the superstructure is added to the substructure at the first floor.

3- S_{XS} and S_{X1} refer to the spectral response at 0.2s and 1.0s, respectively, after applying site amplification factors Fa and Fv. These values match S_{CS} and S_{C1} for the building, per the table "UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards".

4- Per Section 4.4.2.3 in ASCE 41-17, the spectral acceleration, Sa, is computed as the least value of S_{X1}/T, and S_{XS}.

5- Modification Factor, C, per ASCE 41-17, Table 4-7.

Table 4-7. Modification Factor, *C*

Building Type[a]	Number of Stories 1	2	3	≥4
Wood and cold-formed steel shear wall (W1, W1a, W2, CFS1) Moment frame (S1, S3, C1, PC2a)	1.3	1.1	1.0	1.0
Shear wall (S4, S5, C2, C3, PC1a, PC2, RM2, URMa) Braced frame (S2) Cold-formed steel strap-brace wall (CFS2)	1.4	1.2	1.1	1.0
Unreinforced masonry (URM) Flexible diaphragms (S1a, S2a, S5a, C2a, C3a, PC1, RM1)	1.0	1.0	1.0	1.0

[a] Defined in Table 3-1.

Average Wall Stress Check

Average Stresses

Ms = 4.5
f'c = 4000 psi (see Note 3)

Longitudinal (N-S direction)					
Story	**Story Shear**	**Wall Area**	**Average Shear Stress Demand**	**Tier 1 Shear Stress Limit**	**Wall OK?**
	(kips)	(in^2)	(psi)	(psi)	
First Floor - Mezzanine	7,585	42,336	40	126	OK
Mezzanine - Basement	9,977	82,107	27	126	OK

Transverse (E-W direction)					
Story	**Story Shear**	**Wall Area**	**Average Shear Stress Demand**	**Tier 1 Shear Stress Limit**	**Wall OK?**
	(kips)	(in^2)	(psi)	(psi)	
First Floor - Mezzanine	7,585	35,490	47	126	OK
Mezzanine - Basement	9,977	104,982	21	126	OK

Notes:

1 - Shear stress check is performed following the ASCE 41-17 Tier 1 screening criteria, and the BSE-C site modified spectral response parameters.

2 - The concrete shear walls are located below ground in this structure.

3 - Ms factor per ASCE 41-17 Table 4-8.

Table 4-8. M_s Factors for Shear Walls

	Level of Performance		
Wall Type	**CP[a]**	**LS[a]**	**IO[a]**
Reinforced concrete, precast concrete, wood, reinforced masonry, and cold-formed steel	4.5	3.0	1.5
Unreinforced masonry	1.75	1.25	1.0

[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.

4 - Per the General Note on Sheet S100 in the 1998 drawings, the basement walls are specified with a compressive strength of 4,000 psi.

5 - The Tier 1 shear stress limit for concrete shear walls is defined as the greater of 100 psi or 2√(f'c).

Column Shear Capacity in N-S and E-W Direction

Elastic modulus: 29000 ksi

Story	Column Section	Beam Section	BSE-C Story Shear (kips)	F_y (ksi)	d (in)	t_w (in)	A_w (in^2)	Single Column V_n (kips)	Cols of this Section per Floor	Σ V_n (kips)	Σ V_n per floor (kips)	DCR
Penthouse Floor & Roof - Fifth Floor	W14X233	W24X94	2327	50	16.0	1.07	17.1	514	3	1,541	4,434	0.52
	W14X311	W24X94	2327	50	17.1	1.41	24.1	723	4	2,893		
Fifth Floor - Fourth Floor	W14X233	W24X94	4006	50	16.0	1.07	17.1	514	3	1,541	7,327	0.55
	W14X311	W24X94	4006	50	17.1	1.41	24.1	723	8	5,787		
Fourth Floor - Third Floor	W14X311	W24X94	5339	50	17.1	1.41	24.1	723	8	5,787	5,787	0.92
Third Floor - Second Floor	W14X342	W24X117	6144	50	17.5	1.54	27.0	809	8	6,468	6,468	0.95
Second Floor - First Floor	W14X342	W27X146	6483	50	17.5	1.54	27.0	809	8	6,468	6,468	1.00

Notes:

1 - The number of columns correspond to the wide flange steel columns in the seismic-force resisting frame.

2 - Each direction of loading has the same number of MF bays, size of MF members, and spans. Therefore, the calculation above ia applicable in both directions.

3 - Shear capacity is calculated using Eq. G2-1 / AISC 360. The factor Cv = 1.0.

$$V_n = 0.6 F_y A_w C_v$$

Story Drift for Moment Frames in N-S and E-W Direction for BSE-C

Per Section 4.4.3.1 in ASCE 41-17:

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

where

D_r = Drift ratio: interstory displacement divided by story height;
k_b = I/L for the representative beam;
k_c = I/h for the representative column;
h = Story height (in.);
I = Moment of inertia (in.^{4});
L = Beam length from center-to-center of adjacent columns (in.);
E = Modulus of elasticity (kip/in.^{2}); and
V_c = Shear in the column (kip).

Elastic modulus: 29000 ksi

Story	Column Section	Beam Section	BSE-C Story Shear (kips)	Columns		Column Geometry			Beam Geometry			k_c (in^3)	k_b (in^3)	D_r	D_{limit}	Acceptance Criteria
				Total No. Cols per Floor	V_c (kips)	I_c (in^4)	h_c (ft)	h (in)	I_b (in^4)	L_b (ft)	L (in)					
Penthouse Floor & Roof - Fifth Floor	W14X233	W24X94	2,327	7	332	3010	13.00	156.0	2700	22.67	272.0	19.3	9.9	0.023	0.03	OK
	W14X311	W24X94	2,327	7	332	4330	13.00	156.0	2700	22.67	272.0	27.8	9.9	0.020	0.03	OK
Fifth Floor - Fourth Floor	W14X233	W24X94	4,006	11	364	3010	13.00	156.0	2700	22.67	272.0	19.3	9.9	0.025	0.03	OK
	W14X311	W24X94	4,006	11	364	4330	13.00	156.0	2700	22.67	272.0	27.8	9.9	0.022	0.03	OK
Fourth Floor - Third Floor	W14X311	W24X94	5,339	8	667	4330	13.00	156.0	2700	22.67	272.0	27.8	9.9	0.041	0.03	NG
Third Floor - Second Floor	W14X342	W24X117	6,144	8	768	4900	13.00	156.0	3540	22.67	272.0	31.4	13.0	0.037	0.03	NG
Second Floor - First Floor	W14X342	W27X146	6,483	8	810	4900	13.00	156.0	5660	22.67	272.0	31.4	20.8	0.029	0.03	OK

Notes:

1 - The number of columns correspond to the wide flange steel columns in the seismic-force resisting frame.

2 - Each direction of loading has the same number of MF bays, size of MF members, and spans. Therefore, the calculation above ia applicable in both directions.

3 - This check computes story drift under the BSE-C story shear.

Column Axial Stress Check Caused by Overturning

Per Section 4.4.3.6 in ASCE 41-17:

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

where

n_f = Total number of frames in the direction of loading;
V = Pseudo seismic force;
h_n = Height (ft) above the base to the roof level;
L = Total length of the frame (ft);
M_s = System modification factor taken as equal to 2.5 for buildings being evaluated to the Collapse Prevention Performance Level, equal to 1.5 for buildings being evaluated to the Life Safety Performance Level, and equal to 1.0 for buildings being evaluated to the Immediate Occupancy Performance Level; and
A_{col} = Area of the end column of the frame.

Story	Column Section	Story Shear (kips)	F_y (ksi)	M_s	n_f	h_n (ft)	L (ft)	A_{col} (in^2)	p_{ot} (ksi)	0.3F_y (ksi)	Acceptance criteria
Penthouse Floor & Roof - Fifth Floor	W14X233	2,327	50	2.5	2	65.00	45.33	68.5	6.50	15	OK
	W14X311	2,327	50	2.5	2	65.00	68.00	91.4	3.25	15	OK
Fifth Floor - Fourth Floor	W14X233	4,006	50	2.5	3	65.00	45.33	68.5	7.45	15	OK
	W14X311	4,006	50	2.5	3	65.00	68.00	91.4	3.72	15	OK
Fourth Floor - Third Floor	W14X311	5,339	50	2.5	2	65.00	68.00	91.4	7.44	15	OK
Third Floor - Second Floor	W14X342	6,144	50	2.5	2	65.00	68.00	101	7.75	15	OK
Second Floor - First Floor	W14X342	6,483	50	2.5	2	65.00	68.00	101	8.18	15	OK

Notes:

1 - Per General Notes on S100, wide flange rolled shapes conform the specification ASTM A572 Gr. 50 (Fy = 50 ksi).

2 - The height above the base to the roof level, h_n, is set from the first floor up to the penthouse.

3 - Both perpendicular directions have the same number of moment frame lines and number of bays per line. Thus, the table is applicable for the E-W and N-S directions.

4 - Under similar conditions, the mlongest frame was was taken for the calculations as it entails a higher axial stress due to overturning.

Flexural Stress in Columns and Beams of Steel Moment Frames

Per Section 4.4.3.9 in ASCE 41-17:

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

where

n_c = Total number of frame columns at the level, j, under consideration.

n_f = Total number of frames in the direction of loading at the level, j, under consideration.

V_j = Story shear computed in accordance with Section 4.4.2.2.

h = Story height (in.).

Z = For columns, the sum of the plastic section moduli of all the frame columns at the level under consideration. For beams, it is the sum of the plastic section moduli of all the frame beams with moment-resisting connections. If a beam has moment-resisting connections at both ends, then the contribution of that beam to the sum is twice the plastic section modulus of that beam (in^3).

M_s = System modification factor; M_s shall be taken as equal to 9.0 for buildings being evaluated to the Collapse Prevention Performance Level, equal to 6.0 for buildings being evaluated to the Life Safety Performance Level, and equal to 2.5 for buildings being evaluated to the Immediate Occupancy Performance Level for columns and beams satisfying the checklist items for compactness and column axial stress. If the columns or beams do not satisfy the checklist statements for compactness and column axial stress for the Immediate Occupancy Performance Level, then this item must be marked "Noncompliant".

Story	SMRF ID	Column Section	Beam Section	No. columns	No. beams	Column Z (in^3)	Beam Z (in^3)
Penthouse Floor & Roof - Fifth Floor	"A"	W14X233	W24X94	3	2	1308.0	1016.0
	"B"	W14X311	W24X94	4	3	2412.0	1524.0
			Σ =	7	Σ =	3,720	2,540
Fifth Floor - Fourth Floor	"A"	W14X233	W24X94	3	2	1308.0	1016.0
	"B"	W14X311	W24X94	8	6	4824.0	3048.0
			Σ =	11	Σ =	6,132	4,064
Fourth Floor - Third Floor	"B"	W14X311	W24X94	8	6	4824.0	3048.0
Third Floor - Second Floor	"B"	W14X342	W24X117	8	6	5376.0	3924.0
Second Floor - First Floor	"B"	W14X342	W27X146	8	6	5376.0	5568.0

									Demand		Capacity		Acceptance Criteria	
Story	Story Shear (kips)	M_s	n_c	n_f	h (ft)	h (in)	Column Z (in^3)	Beam Z (in^3)	Column f_j^{avg} (ksi)	Beam f_j^{avg} (ksi)	Column Fy (ksi)	Beam Fy (ksi)	Column	Beam
Penthouse Floor & Roof - Fifth Floor	2,327	9.0	7	2	13.00	156.0	3720.0	2540.0	7.6	11.1	50	50	OK	OK
Fifth Floor - Fourth Floor	4,006	9.0	11	3	13.00	156.0	6132.0	4064.0	7.8	11.7	50	50	OK	OK
Fourth Floor - Third Floor	5,339	9.0	8	2	13.00	156.0	4824.0	3048.0	12.8	20.2	50	50	OK	OK
Third Floor - Second Floor	6,144	9.0	8	2	13.00	156.0	5376.0	3924.0	13.2	18.1	50	50	OK	OK
Second Floor - First Floor	6,483	9.0	8	2	13.00	156.0	5376.0	5568.0	13.9	13.5	50	50	OK	OK

Notes:

1 - The number of columns correspond to the wide flange steel columns in the seismic-force resisting frame.

2 - Each direction of loading has the same number of MF bays, same member sizes, and same spans. Therefore, this calculation is applicable in both the N-S and E-W direction.

3 - All the beams have moment-reisting connections at both ends; therefore, per section 4.4.3.9, the beam plastic section moduli is multiplied by 2.

4 - The columns within the moment frames are oriented about their strong axis. Zx is used in the above calculation.

5 - The flexural stress check is compliant if f_j < Fy.

Panel Zones

The shear demand on the panel zone associated with a plastic hinge forming in the reduced section of the beam is:

$$\Sigma M_p = \Sigma Z_{x,RBS} \times F_{ye,beam}$$

$$V_{p,RBS} = \frac{\Sigma M_p}{L_{hinge}}$$

$$V_p = \frac{\Sigma M_p + V_{p,RBS} \times e}{d_{beam}}$$

where:

M_p, Expected yielding moment capacity of the reduced section of the beam, $Mp = Ry \times Z_{RBS} \times Fy$

ΣM_p, Sum of the expected yielding moment capacities of beams

V_p, Expected shear in panel zone due to beam yielding

$F_{ye,\ beam}$, Expected strength of beams equal to Ry x Fy

$F_{ye,\ column}$, Expected strength of columns equal to Ry x Fy

$F_{ye,\ plate}$, Expected strength of doubler plate

$Z_{x,RBS}$, Strong axis plastic modulus at the reduced beam section

d_{beam}, Beam depth

d_{column}, Column depth

$t_{w,column}$, Column web thickness

P_r, Column axial demand

P_c, Column axial capacity

t_p, Doubler plate thickness

V_e, Panel zone expected capacity

e, distrance from the face of the column to the center of the RBS.

E, Elastic modulus 29000 ksi

The expected panel zone capacity is conservatively calculated neglecting the effect of panel zone deformation on frame stability, in accordance with AISC 360-16, Section J10.6 (a).

(i) For $P_r \le 0.4P_c$

$$V_e = 0.6\left(F_{ye,\text{column}} t_{w,\text{column}} + F_{ye,\text{plate}} t_p\right) d_{\text{column}}$$

(ii) For $P_r > 0.4P_c$

$$V_e = 0.6\left(F_{ye,\text{column}} t_{w,\text{column}} + F_{ye,\text{plate}} t_p\right) d_{\text{column}} \left(1.4 - \frac{P_r}{P_c}\right)$$

Beam Size	Z_x (in^3)	Thickness flange, tf (in)	Depth, d (in)	RBS Cut, "c" (in)	$Z_{x,\ RBS}$ (in^3)
W24x94	254	0.875	24.3	2.25	161.8
W24X117	327	0.85	24.3	3	207.4
W27x146	464	0.975	27.4	3.5	283.6

Panel Zone Demand

Story	Column Section	Beam Section	Beam location in frame	No. Beams at joint	L_{hinge} (Length between hinges, ft)	Beam Fy (ksi)	$Z_{x,\ RBS}$ (in^3)	R_y	d_{beam} (in)	ΣM_p (kip-ft)	$V_{p,RBS}$ (kip)	e (in)	$0.8V_p$ (kip)
Penthouse Floor & Roof - Fifth Floor	W14X233	W24X94	Interior	2	19.0	50	161.8	1.1	24.3	1,483	78.0	14.0	622
	W14X233	W24X94	End	1	19.0	50	161.8	1.1	24.3	741	39.0	14.0	311
	W14X311	W24X94	Interior	2	18.9	50	161.8	1.1	24.3	1,483	78.4	14.0	622
	W14X311	W24X94	End	1	18.9	50	161.8	1.1	24.3	741	39.2	14.0	311
Fifth Floor - Fourth Floor	W14X233	W24X94	Interior	2	19.0	50	161.8	1.1	24.3	1,483	78.0	14.0	622
	W14X233	W24X94	End	1	19.0	50	161.8	1.1	24.3	741	39.0	14.0	311
	W14X311	W24X94	Interior	2	18.9	50	161.8	1.1	24.3	1,483	78.4	14.0	622
	W14X311	W24X94	End	1	18.9	50	161.8	1.1	24.3	741	39.2	14.0	311
Fourth Floor - Third Floor	W14X311	W24X94	Interior	2	18.9	50	161.8	1.1	24.3	1,483	78.4	14.0	622
	W14X311	W24X94	End	1	18.9	50	161.8	1.1	24.3	741	39.2	14.0	311
Third Floor - Second Floor	W14X342	W24X117	Interior	2	18.5	50	207.4	1.1	24.3	1,901	102.5	16.0	805
	W14X342	W24X117	End	1	18.5	50	207.4	1.1	24.3	951	51.3	16.0	403
Second Floor - First Floor	W14X342	W27X146	Interior	2	18.4	50	283.6	1.1	27.4	2,600	141.5	17.0	981
	W14X342	W27X146	End	1	18.4	50	283.6	1.1	27.4	1,300	70.7	17.0	491

Notes:

1 - The number of beams at the joint represents the number of beam hinges forming at a joint. At the end of a bay, one beam hinge forms. At the interior bay, two beam hinges form.

2 - L is taken as thedistance between the centerline of the reduced beam section.

3 - M_p is the plastic moment capacity of the beam hinge, ΣM_p = (No. beams at joint) x Ry x Fy x $Z_{x,\ RBS.}$

4 - $V_{p,\ RBS}$ is the shear force associated with the development of M_p at the reduced beam section, $V_{p,\ RBS}$ = (No. beams at joint) x Ry x Fy x $Z_{x,\ RBS}$ / L_{hinge}.

5 - e is the distance from the face of the column to the center of the reduced beam section as specified on Sheet S701 in Detail 6.

Column Axial Demand, P_r

Story	Column Section	Column location in frame	Trib. Area (ft^2)	Dead Load: Unit weight (psf)	Dead Load: DL (kips)	Live Load: Unit weight (psf)	Live Load: LL (kips)	1.1DL + 0.275LL (kips)
Penthouse Floor & Roof - Fifth Floor	W14X233	Interior	513.8	135	69.6	20	10.3	79.4
	W14X233	End	513.8	135	69.6	20	10.3	79.4
	W14X311	Interior	256.9	135	34.8	20	5.1	39.7
	W14X311	End	256.9	135	34.8	20	5.1	39.7
Fifth Floor - Fourth Floor	W14X233	Interior	513.8	90	115.8	80	51.4	141.5
	W14X233	End	513.8	90	115.8	80	51.4	141.5
	W14X311	Interior	256.9	172	79.0	80	25.7	93.9
	W14X311	End	256.9	172	79.0	80	25.7	93.9
Fourth Floor - Third Floor	W14X311	Interior	256.9	90	102.1	80	46.2	125.0
	W14X311	End	256.9	90	102.1	80	46.2	125.0
Third Floor - Second Floor	W14X342	Interior	256.9	90	125.2	80	66.8	156.1
	W14X342	End	256.9	90	125.2	80	66.8	156.1
Second Floor - First Floor	W14X342	Interior	256.9	90	148.3	80	87.3	187.2
	W14X342	End	256.9	90	148.3	80	87.3	187.2

Column Axial Capacity, P_c

Story	Column Section	Column location in frame	$F_{y,column}$ (ksi)	r_y (in)	K	L (in)	KL/r	F_e (ksi)	F_y/F_e	F_{cr} (ksi)	A_g (in^2)	P_c (kips)
Penthouse Floor & Roof - Fifth Floor	W14X233	Interior	50	4.10	1.2	126	36.9	210.5	0.238	45.3	68.50	3,101
	W14X233	End	50	4.10	1.2	126	36.9	210.5	0.238	45.3	68.50	3,101
	W14X311	Interior	50	4.20	1.2	126	36.0	220.8	0.226	45.5	91.40	4,157
	W14X311	End	50	4.20	1.2	126	36.0	220.8	0.226	45.5	91.40	4,157
Fifth Floor - Fourth Floor	W14X233	Interior	50	4.10	1.2	126	36.9	210.5	0.238	45.3	68.50	3,101
	W14X233	End	50	4.10	1.2	126	36.9	210.5	0.238	45.3	68.50	3,101
	W14X311	Interior	50	4.20	1.2	126	36.0	220.8	0.226	45.5	91.40	4,157
	W14X311	End	50	4.20	1.2	126	36.0	220.8	0.226	45.5	91.40	4,157
Fourth Floor - Third Floor	W14X311	Interior	50	4.20	1.2	126	36.0	220.8	0.226	45.5	91.40	4,157
	W14X311	End	50	4.20	1.2	126	36.0	220.8	0.226	45.5	91.40	4,157
Third Floor - Second Floor	W14X342	Interior	50	4.24	1.2	126	35.7	225.1	0.222	45.6	101.00	4,602
	W14X342	End	50	4.24	1.2	126	35.7	225.1	0.222	45.6	101.00	4,602
Second Floor - First Floor	W14X342	Interior	50	4.24	1.2	126	35.7	225.1	0.222	45.6	101.00	4,602
	W14X342	End	50	4.24	1.2	126	35.7	225.1	0.222	45.6	101.00	4,602

Note: L is taken to be the clear buckling length of the column. At a minimum this is 13 ft story height reduced by a 24in deep beam and a 6" thick slab

Panel Zone Check

Story	Column Section	Column location in frame	P_r/P_c	$F_{y,column}$ (ksi)	R_y	$F_{ye,column}$ (ksi)	$t_{w,column}$ (in)	d_{column} (in)	t_p (in)	Capacity V_e (kips)	Demand $0.8V_p$ (kips)	DCR	Acceptance criteria
Penthouse Floor & Roof - Fifth Floor	W14X233	Interior	0.03	50	1.1	55.0	1.07	16.0	0.0	565	622	1.10	NG
	W14X233	End	0.03	50	1.1	55.0	1.07	16.0	0.0	565	311	0.55	OK
	W14X311	Interior	0.01	50	1.1	55.0	1.41	17.1	0.0	796	622	0.78	OK
	W14X311	End	0.01	50	1.1	55.0	1.41	17.1	0.0	796	311	0.39	OK
Fifth Floor - Fourth Floor	W14X233	Interior	0.05	50	1.1	55.0	1.07	16.0	0.0	565	622	1.10	NG
	W14X233	End	0.05	50	1.1	55.0	1.07	16.0	0.0	565	311	0.55	OK
	W14X311	Interior	0.02	50	1.1	55.0	1.41	17.1	0.0	796	622	0.78	OK
	W14X311	End	0.02	50	1.1	55.0	1.41	17.1	0.0	796	311	0.39	OK
Fourth Floor - Third Floor	W14X311	Interior	0.03	50	1.1	55.0	1.41	17.1	0.0	796	622	0.78	OK
	W14X311	End	0.03	50	1.1	55.0	1.41	17.1	0.0	796	311	0.39	OK
Third Floor - Second Floor	W14X342	Interior	0.03	50	1.1	55.0	1.54	17.5	0.0	889	805	0.91	OK
	W14X342	End	0.03	50	1.1	55.0	1.54	17.5	0.0	889	403	0.45	OK
Second Floor - First Floor	W14X342	Interior	0.04	50	1.1	55.0	1.54	17.5	0.0	889	981	1.10	NG
	W14X342	End	0.04	50	1.1	55.0	1.54	17.5	0.0	889	491	0.55	OK

Notes:

1 - R_y is the ratio of the expected yield strength to the specified minimum yield stress of the material and is obtained from Table A3.1 / AISC 360-16 for ASTM A572.

2 - The gravity axial demand is based on the combination 1.1DL + 0.275 per ASCE 41-16.

3 - Column compressive strength is limited by the weak axis radius of gyration.

4 - Column compressive strength is determined based on the limt sate of flexural buckling, per Section E3 / AISCE 360-16

5 - Per Det 6 & 8 / S701, the columns panel zones do not contain doubler plates.

Strong Column - Weak Beam

Per Section E3.4a in AISC 341-16:

The following relationship shall be satisfied at beam-to-column connections:

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} > 1.0 \qquad \text{(E3-1)}$$

Material properties for columns and beams:

F_y = 50 ksi

R_y = 1.1 Ry is the ratio of the expected yield strength to the specified minimum yield stress of the material and is obtained from Table ASCE 41-17 Table 9-4 for A572 Gr. 50 material.

F_{ye} = 55 ksi

Overturning Moment, M_{ot}

Floor Levels	Story Height	Cum. Height	Story Force, Fx (kips)	Overturning Moment, M_{ot}
Penthouse Floor & Roof	13.0	0.0	2,327	-
Fifth Floor	13.0	13.0	1,678	30,257
Fourth Floor	13.0	26.0	1,333	82,332
Third Floor	13.0	39.0	805	151,736
Second Floor	13.0	52.0	340	231,607
First Floor	0.0	65.0	-	315,892

Column Axial Seismic Force, P_E

Story	M_{ot} (kips-ft)	SMRF ID	Column Section	Beam Section	A_{col} (in^2)	$I_{x,col}$ (in^4)	Total lines of SRMF at story	No. Columns in single SRMF	$L_{SRMF\ Line}$ (ft)	$I_{group\ of\ cols}$ (ft^4)	σ_E (kips/ft^2)	P_E (kips)
Penthouse Floor & Roof - Fifth Floor	30,257	"A"	W14X233	W24X94	68.5	3,010	2	3	45.33	489	701	333
	30,257	"B"	W14X311	W24X94	91.4	4,330	2	4	68.00	1,631	315	200
Fifth Floor - Fourth Floor	82,332	"A"	W14X233	W24X94	68.5	3,010	3	3	45.33	489	1,271	605
	82,332	"B"	W14X311	W24X94	91.4	4,330	3	4	68.00	1,631	572	363
Fourth Floor - Third Floor	151,736	"B"	W14X311	W24X94	91.4	4,330	2	4	68.00	1,631	1,581	1,004
Third Floor - Second Floor	231,607	"B"	W14X342	W24X117	101	4,900	2	4	68.00	1,803	2,184	1,532
Second Floor - First Floor	315,892	"B"	W14X342	W27X146	101	4,900	2	4	68.00	1,803	2,979	2,089

Notes:

1 - The SMRF ID "A" takes place on Grids 2 & F at the stories between the fourth floor and the roof. The SMRF ID "B" takes place on Grids 1, 5, B & G at the stories between the first and fifth floor, and on Grids 5 & B at story between the fifth floor and the roof.

2 - The column axial seismic force demand is computed using the following equations:

$$\sigma_E = \frac{1}{Total\ lines\ of\ SMRF} \times M_{ot} \times \frac{L_{SMRF\ Line}}{2} \times \frac{1}{I_{group\ of\ cols}}$$

$$P_E = \sigma_E \times A_{col}$$

Sum of the Expected Flexural Strengths of the Columns, ΣM_{pc}

Story	SMRF ID	Column Section	A_{col} (in^2)	$Z_{x,col}$ (in^3)	Column location in frame	Trib. Area (ft^2)	Dead Load: Unit weight (psf)	Dead Load: DL (kips)	Live Load: Unit weight (psf)	Live Load: LL (kips)	P_G = 1.1DL + 0.275LL (kips)	P_E (kips)	P_r (kips)	P_r / A_g (ksi)	No. Cols at joint	ΣM_{pc} (kips-in)
Penthouse Floor & Roof - Fifth Floor	"A"	W14X233	68.5	436	Interior	540.2	135	73.1	20	10.8	83.4		83.4	1.2	1	23,449
	"A"	W14X233	68.5	436	End	513.8	135	69.6	20	10.3	79.4	333.4	412.8	6.0	1	21,353
	"B"	W14X311	91.4	603	Interior	256.9	135	34.8	20	5.1	39.7		39.7	0.4	1	32,903
	"B"	W14X311	91.4	603	End	256.9	135	34.8	20	5.1	39.7	200.1	239.8	2.6	1	31,583
Fifth Floor - Fourth Floor	"A"	W14X233	68.5	436	Interior	513.8	90	119.4	80	51.9	145.6		145.6	2.1	2	46,106
	"A"	W14X233	68.5	436	End	513.8	90	115.8	80	51.4	141.5	604.8	746.4	10.9	2	38,459
	"B"	W14X311	91.4	603	Interior	256.9	172	79.0	80	25.7	93.9		93.9	1.0	2	65,091
	"B"	W14X311	91.4	603	End	256.9	172	79.0	80	25.7	93.9	363.0	457.0	5.0	2	60,300
Fourth Floor - Third Floor	"B"	W14X311	91.4	603	Interior	256.9	90	102.1	80	46.2	125.0		125.0	1.4	2	64,680
	"B"	W14X311	91.4	603	End	256.9	90	102.1	80	46.2	125.0	1003.6	1128.6	12.3	2	51,438
Third Floor - Second Floor	"B"	W14X342	101	672	Interior	256.9	90	125.2	80	66.8	156.1		156.1	1.5	2	71,843
	"B"	W14X342	101	672	End	256.9	90	125.2	80	66.8	156.1	1531.9	1688.0	16.7	2	51,458
Second Floor - First Floor	"B"	W14X342	101	672	Interior	256.9	90	148.3	80	87.3	187.2		187.2	1.9	2	71,429
	"B"	W14X342	101	672	End	256.9	90	148.3	80	87.3	187.2	2089.4	2276.6	22.5	2	43,626

Notes:

1 - The gravity axial demand for columns and beams is based on the combination 1.1DL + 0.275LL per ASCE 41-16.

2 - The sum of the projections of the expected flexural strengths of the columns is calculated as follows:

$$\Sigma M_{pc} = (No.cols\ at\ joint) \times Z_{x,col} \times (F_{ye,column} - \frac{P_r}{A_g})$$

3 - The number of columns represents the number of column hinges around a joint. At the top most story, one column hinge resists the beam hinges. At the lower stories, two column hinges resist the beam hinges.

Sum of the Expected Flexural Strengths of the Beams, ΣM_{pb}

Beam Size	Z_x (in^3)	Thickness flange, tf (in)	Depth, d (in)	RBS Cut, "c" (in)	$Z_{x, RBS}$ (in^3)
W24x94	254	0.875	24.3	2.25	161.8
W24X117	327	0.85	24.3	3	207.4
W27x146	464	0.975	27.4	3.5	283.6

Story	SMRF ID	Beam Section	A_b (in^2)	$Z_{x\ RBS,b}$ (in^3)	Column location in frame	No. Beams at joint	L_{hinge} (Length between hinges, ft)	Trib. Area (ft^2)	Dead Load: Unit weight (psf)	Dead Load: V_{DL} (kips)	Live Load: Unit weight (psf)	Live Load: V_{LL} (kips)	$V_G = 1.1V_{DL} + 0.275V_{LL}$ (kips)	V_p (kips)	V_{bL} (kips)	V_{bR} (kips)	d_{col} (in)	e (in)	$d_{col}/2 + e$ (in)	ΣM_{uv} (kips-in)	ΣM_{pb} (kips-in)
Penthouse Floor & Roof - Fifth Floor	"A"	W24X94	27.7	161.8	Interior	2	19.00	270.1	135	36.6	20	5.4	41.7	78.0	119.7	36.3	16.0	14.0	22.0	3,433	21,227
	"A"	W24X94	27.7	161.8	End	1	19.00	270.1	135	36.6	20	5.4	41.7	39.0	80.7	-2.7	16.0	14.0	22.0	1,776	10,673
	"B"	W24X94	27.7	161.8	Interior	2	18.91	128.4	135	17.4	20	2.6	19.8	78.4	98.2	58.6	17.1	14.0	22.6	3,536	21,330
	"B"	W24X94	27.7	161.8	End	1	18.91	128.4	135	17.4	20	2.6	19.8	39.2	59.0	19.4	17.1	14.0	22.6	1,331	10,228
Fifth Floor - Fourth Floor	"A"	W24X94	27.7	161.8	Interior	2	19.00	256.9	90	23.1	20	5.1	26.9	78.0	104.9	51.2	16.0	14.0	22.0	3,433	21,227
	"A"	W24X94	27.7	161.8	End	1	19.00	256.9	90	23.1	20	5.1	26.9	39.0	65.9	12.2	16.0	14.0	22.0	1,449	10,346
	"B"	W24X94	27.7	161.8	Interior	2	18.91	128.4	172	22.1	20	2.6	25.0	78.4	103.4	53.4	17.1	14.0	22.6	3,536	21,330
	"B"	W24X94	27.7	161.8	End	1	18.91	128.4	172	22.1	20	2.6	25.0	39.2	64.2	14.2	17.1	14.0	22.6	1,448	10,345
Fourth Floor - Third Floor	"B"	W24X94	27.7	161.8	Interior	2	18.91	128.4	90	11.6	20	2.6	13.4	78.4	91.8	65.0	17.1	14.0	22.6	3,536	21,330
	"B"	W24X94	27.7	161.8	End	1	18.91	128.4	90	11.6	20	2.6	13.4	39.2	52.6	25.8	17.1	14.0	22.6	1,187	10,084
Third Floor - Second Floor	"B"	W24X117	34.4	207.4	Interior	2	18.55	128.4	90	11.6	20	2.6	13.4	102.5	115.9	89.1	17.5	16.0	24.8	5,075	27,889
	"B"	W24X117	34.4	207.4	End	1	18.55	128.4	90	11.6	20	2.6	13.4	51.3	64.7	37.8	17.5	16.0	24.8	1,601	13,008
Second Floor - First Floor	"B"	W27X146	43.2	283.6	Interior	2	18.38	128.4	90	11.6	20	2.6	13.4	141.5	154.9	128.1	17.5	17.0	25.8	7,286	38,488
	"B"	W27X146	43.2	283.6	End	1	18.38	128.4	90	11.6	20	2.6	13.4	70.7	84.2	57.3	17.5	17.0	25.8	2,167	17,768

Notes:

1 - The number of beams at the joint represents the number of beam hinges forming at a joint. At the end of a bay, one beam hinge forms. At the interior bay, two beam hinges form

2 - L is taken as thedistance between the centerline of the reduced beam section.

3 - V_{bL} and V_{bR} are the beam shear forces at each end of the beam. $V_{bL} = 1.1V_{DL} + 0.275V_{LL} + V_p$ and $V_{bR} = 1.1V_{DL} + 0.275V_{LL} - V_p$.

4 - e is the distance from the face of the column to the center of the reduced beam section as specified on Sheet S701 in Detail 6

5 - M_{UV} is the total moment at the column centerline due to V_{bL} and V_{bR}, $M_{UV} = V_{bL} \times (e + d_{col}/2) + V_{bR} \times (e + d_{col}/2)$.

6 - M_{pb} is the plastic moment capacity of the beam hinge, ΣM_{pb} = (No. beams at joint) x Ry x Fy x $Z_{x,\ RBS}$ + ΣM_{uv}.

Strong Column / Weak Beam Summary Table

Story	SMRF ID	Column Section	Beam Section	Column location in frame	ΣM_{pc} (kips-in)	ΣM_{pb} (kips-in)	$\Sigma M_{pc}/\Sigma M_{pb}$	Joint Strong Element
Penthouse Floor & Roof - Fifth Floor	"A"	W14X233	W24X94	Interior	23,449	21,227	1.10	Strong Column
	"A"	W14X233	W24X94	End	21,353	10,673	2.00	Strong Column
	"B"	W14X311	W24X94	Interior	32,903	21,330	1.54	Strong Column
	"B"	W14X311	W24X94	End	31,583	10,228	3.09	Strong Column
Fifth Floor - Fourth Floor	"A"	W14X233	W24X94	Interior	46,106	21,227	2.17	Strong Column
	"A"	W14X233	W24X94	End	38,459	10,346	3.72	Strong Column
	"B"	W14X311	W24X94	Interior	65,091	21,330	3.05	Strong Column
	"B"	W14X311	W24X94	End	60,300	10,345	5.83	Strong Column
Fourth Floor - Third Floor	"B"	W14X311	W24X94	Interior	64,680	21,330	3.03	Strong Column
	"B"	W14X311	W24X94	End	51,438	10,084	5.10	Strong Column
Third Floor - Second Floor	"B"	W14X342	W24X117	Interior	71,843	27,889	2.58	Strong Column
	"B"	W14X342	W24X117	End	51,458	13,008	3.96	Strong Column
Second Floor - First Floor	"B"	W14X342	W27X146	Interior	71,429	38,488	1.86	Strong Column
	"B"	W14X342	W27X146	End	43,626	17,768	2.46	Strong Column

Notes:

1 - A strong column-weak beam is defined with the following relationship:

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} > 1.0$$

Compact Members

Per Table D1.1 in AISC 341-16:
Acceptance criteria for moderately ductile members:

For flanges:

$$\frac{b_f}{2t_f} < \lambda_{md,flange}$$

$$\lambda_{md,flange} = 0.40\sqrt{\frac{E}{R_y F_y}}$$

For webs:

$$\frac{h_f}{t_w} < \lambda_{md,web}$$

$For\ C_a \leq 0.114$

$$\lambda_{md,web} = 3.96\sqrt{\frac{E}{R_y F_y}}(1 - 3.04C_a)$$

$For\ C_a > 0.114$

$$\lambda_{md,web} = 1.29\sqrt{\frac{E}{R_y F_y}}(2.12 - C_a) \geq 1.57\sqrt{\frac{E}{R_y F_y}}$$

Where:

$$C_a = \frac{P_u}{\Phi_c P_y}$$

$$P_y = R_y F_y A_g$$

Φ_c = 0.9
R_y = 1.1
F_y = 50 ksi
E = 29000 ksi

Columns

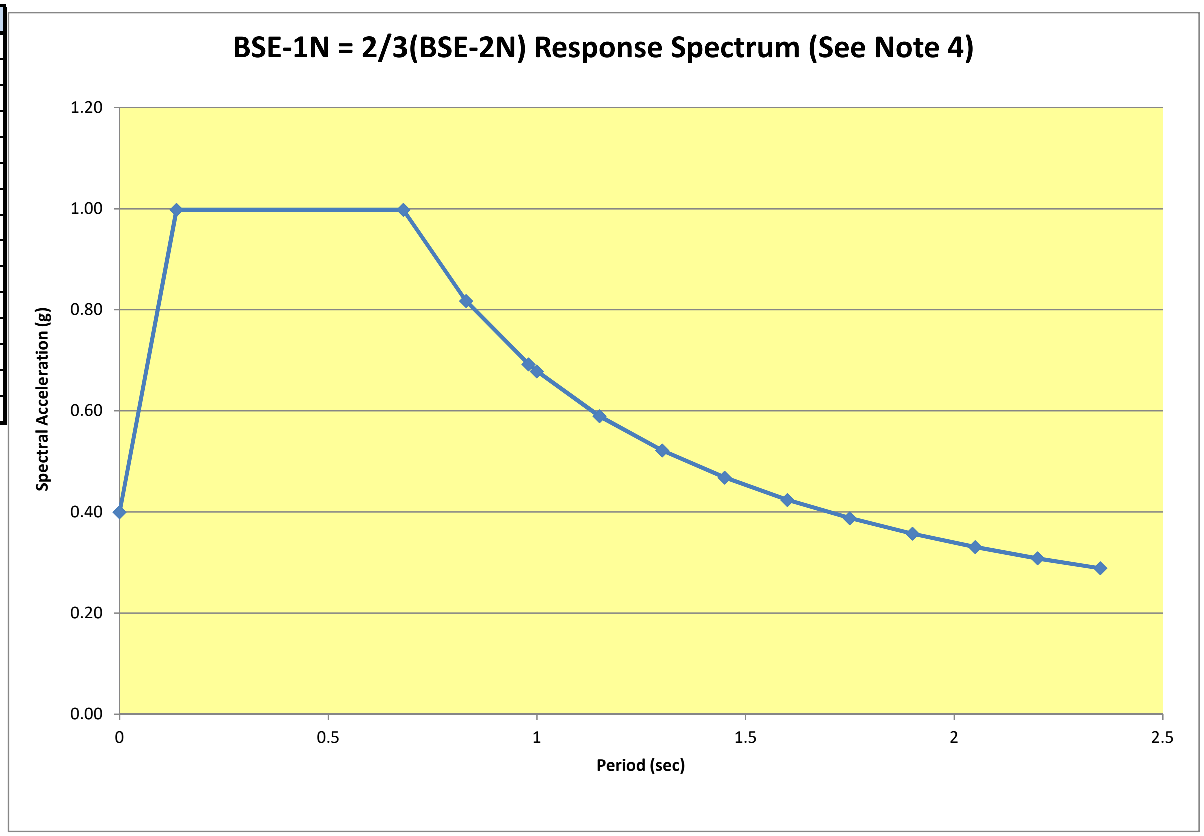
Story	SMRF ID	Column Section	Column location in frame	A_g (in^2)	$b_f/2t_f$	h/t_w	P_u (kips)	P_y (kips)	C_a	$\lambda_{md,flange}$	$\lambda_{md,web}$	Flange compactness	Web compactness
Penthouse Floor & Roof - Fifth Floor	"A"	W14X233	Interior	68.5	4.62	10.7	83	3767.5	0.025	9.2	84.1	OK	OK
	"A"	W14X233	End	68.5	4.62	10.7	413	3767.5	0.122	9.2	59.2	OK	OK
	"B"	W14X311	Interior	91.4	3.59	8.09	40	5027	0.009	9.2	88.5	OK	OK
	"B"	W14X311	End	91.4	3.59	8.09	240	5027	0.053	9.2	76.3	OK	OK
Fifth Floor - Fourth Floor	"A"	W14X233	Interior	68.5	4.62	10.7	146	3767.5	0.043	9.2	79.1	OK	OK
	"A"	W14X233	End	68.5	4.62	10.7	746	3767.5	0.220	9.2	56.3	OK	OK
	"B"	W14X311	Interior	91.4	3.59	8.09	94	5027	0.021	9.2	85.2	OK	OK
	"B"	W14X311	End	91.4	3.59	8.09	457	5027	0.101	9.2	63.0	OK	OK
Fourth Floor - Third Floor	"B"	W14X311	Interior	91.4	3.59	8.09	125	5027	0.028	9.2	83.3	OK	OK
	"B"	W14X311	End	91.4	3.59	8.09	1129	5027	0.249	9.2	55.4	OK	OK
Third Floor - Second Floor	"B"	W14X342	Interior	101	3.31	7.41	156	5555	0.031	9.2	82.3	OK	OK
	"B"	W14X342	End	101	3.31	7.41	1688	5555	0.338	9.2	52.8	OK	OK
Second Floor - First Floor	"B"	W14X342	Interior	101	3.31	7.41	187	5555	0.037	9.2	80.6	OK	OK
	"B"	W14X342	End	101	3.31	7.41	2277	5555	0.455	9.2	49.3	OK	OK

Beams

Beam Section	A_g (in^2)	$b_f/2t_f$	h/t_w	P_u (kips)	P_y (kips)	C_a	$\lambda_{md,flange}$	$\lambda_{md,web}$	Flange compactness	Web compactness
W24X94	27.7	5.18	41.9	0	1523.5	0.000	9.2	90.9	OK	OK
W24X117	34.4	7.53	39.2	0	1892	0.000	9.2	90.9	OK	OK
W27X146	43.2	7.16	39.4	0	2376	0.000	9.2	90.9	OK	OK

Site Parameters

Period (s)	Sa (g) BSE-2N	2/3 x Sa (g)= BSE-1N
0	0.60	0.40
0.14	1.50	1.00
0.68	1.50	1.00
0.83	1.23	0.82
0.98	1.04	0.69
1.00	1.02	0.68
1.15	0.88	0.59
1.30	0.78	0.52
1.45	0.70	0.47
1.60	0.64	0.42
1.75	0.58	0.39
1.90	0.54	0.36
2.05	0.50	0.33
2.20	0.46	0.31
2.35	0.43	0.29



BSE-2N		
β =	0.05	
B_1 =	1.00	
S_S =	1.500	g
S_1 =	0.600	g
F_a =	1.000	g
F_v =	1.700	g
Site Class =	D	
S_{2NS} =	1.500	g
S_{2N1} =	1.020	g
T_0 =	0.14	s
T_s =	0.68	s
T =	0.99	s
(2/3) S_a =	0.69	g (See Note 2)
Tier 1 (2/3) S_a =	**0.69**	g (See Note 3)

Notes:

1- Spectral accelerations based upon site class provided in report "UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards". Procedure as specified in ASCE 41-17, Section 2.4.1.7 is used to develop General Response Spectrum shown above.

2 - Per Section 2.4.1.7 of ASCE 41-17, use of spectral response acceleration in the extreme short-period range ($T < T_0$) shall only be permitted in dynamic analysis procedures and only for modes other than the fundamental mode.

3- Per Section 4.4.2.3 for Tier 1 screening in ASCE 41-17, the spectral acceleration, Sa, is computed as the least value of S_{X1}/T, and S_{XS}.

4- BSE-1N is the Performance Objective Equivalent to New Building Standards, taken as (2/3)BSE-2N.

5- BSE-2N represents the ground shaking based on the MCE_R, per ASCE 7.

Story Drift for Moment Frames in N-S and E-W Direction for BSE-1N

Per Section 4.4.3.1 in ASCE 41-17:

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

where

D_r = Drift ratio: interstory displacement divided by story height;
k_b = I/L for the representative beam;
k_c = I/h for the representative column;
h = Story height (in.);
I = Moment of inertia (in.^{4});
L = Beam length from center-to-center of adjacent columns (in.);
E = Modulus of elasticity (kip/in.^{2}); and
V_c = Shear in the column (kip).

Elastic modulus: 29000 ksi

C_d = 5.5 (See Note 3)
R = 8 (See Note 3)
I_e = 1.0 (Importance factor based on risk category II)

Note: BSE-1N = 2/3 BSE-2N
Sa (BSE-1N) = 0.69 (See Note 4)
Cs = 0.09 (See Note 5)
Sa (BSE-C) = 0.98
Cs / Sa (BSE-C) = 0.09

Story	Column Section	Beam Section	BSE-C Story Shear (kips)	BSE-1N Story Shear (kips)	Columns		Column Geometry			Beam Geometry					Elastic Drift	Inelastic Drift	Allowable Drift	Acceptance Criteria
					Total No. Cols per Floor	V_c (kips)	I_c (in^4)	h_c (ft)	h (in)	I_b (in^4)	L_b (ft)	L (in)	k_c (in^3)	k_b (in^3)	δ_{xe}	δ_x	Δ_a	
Penthouse Floor & Roof - Fifth Floor	W14X233	W24X94	2,327	204	7	29	3010	13.00	156.0	2700	22.67	272.0	19.3	9.9	0.002	0.011	0.02	OK
	W14X311	W24X94	2,327	204	7	29	4330	13.00	156.0	2700	22.67	272.0	27.8	9.9	0.002	0.010	0.02	OK
Fifth Floor - Fourth Floor	W14X233	W24X94	4,006	350	11	32	3010	13.00	156.0	2700	22.67	272.0	19.3	9.9	0.002	0.012	0.02	OK
	W14X311	W24X94	4,006	350	11	32	4330	13.00	156.0	2700	22.67	272.0	27.8	9.9	0.002	0.011	0.02	OK
Fourth Floor - Third Floor	W14X311	W24X94	5,339	467	8	58	4330	13.00	156.0	2700	22.67	272.0	27.8	9.9	0.004	0.020	0.02	OK
Third Floor - Second Floor	W14X342	W24X117	6,144	537	8	67	4900	13.00	156.0	3540	22.67	272.0	31.4	13.0	0.003	0.018	0.02	OK
Second Floor - First Floor	W14X342	W27X146	6,483	567	8	71	4900	13.00	156.0	5660	22.67	272.0	31.4	20.8	0.003	0.014	0.02	OK

Notes:
1 - The number of columns correspond to the wide flange steel columns in the seismic-force resisting frame.
2 - Each direction of loading has the same number of MF bays, size of MF members, and spans. Therefore, the calculation above ia applicable in both directions.
3 - The response modification coefficient, R, and the deflection amplification factor, Cd, are obtained from Table 12.2-1 / ASCE 7-16.
4 - Spectral accelerations based upon site class provided in report "UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards".
5 - The seismic response coefficient, Cs, is calculated per Section 12.8.1.1 / ASCE 7-16. Cs = Sa (BSE-1N) / (R/I_e).
6 - BSE-1N is used as the hazard level for life safety performance level for new structures. It is calculated as 2/3(BSE-2N).
7 - In accordance with Eq. 12.8-15 / ASCE 7-16, the acceptance criteria is defined as: C_d x δxe ≤ Δ_a.
8 - For this steel moment frame structure with the associated Seismic Design Category D, the redundancy factor, ρ, is assumed to be 1.0.