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

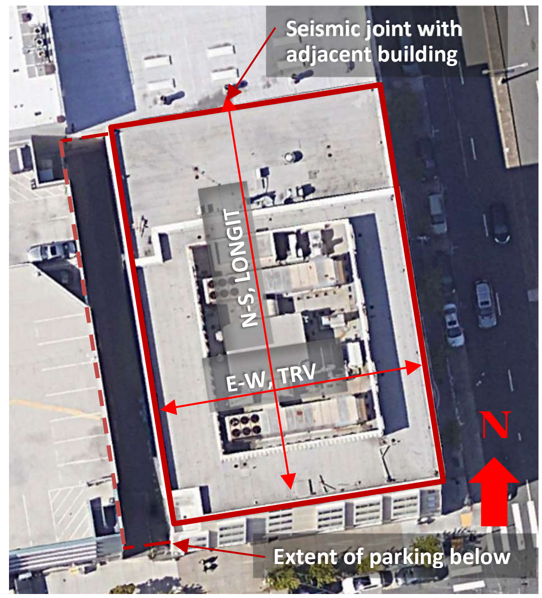
UCSF building seismic ratings

1701 Divisadero, MOB 2

CAAN #2036
1701 Divisadero St., San Francisco, CA 94115
UCSF Campus: Mt. Zion

REGISTERED PROFESSIONAL ENGINEER JOSEPH RICHARD MAFFEI No. 3694 STRUCTURAL STATE OF CALIFORNIA

DATE: 2020-06-26



Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	V	Based on drawing review and Tier 1 evaluation[1] plus additional calculations.
Rating basis	Tier 1	ASCE 41-17
Date of rating	2019	
Recommended UCSF priority category for retrofit	Priority B	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application for modification
Ballpark total construction cost to retrofit to IV rating[2]	High ($200/sf to $400/sf)	Based on overall square footage of building, including parking levels.
Is 2018-2019 rating required by UCOP?	Yes	Building previously rated IV but does not have a fully documented previous review
Further evaluation recommended?	Tier 2	

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

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

Building information used in this evaluation

- Structural drawings by Steven Tipping + Associates, "1701 Divisadero Medical Office Building," as-builts, 1996-08-15 (14 sheets)
- Architectural drawings by Kaplan McLaughlin Diaz, "1701 Divisadero Medical Office Building," as-builts, 1996-08-01 (17 sheets)
- Project documents from the records of Tipping Structural Engineers, listed on page 12 of this report.

Additional building information known to exist

- Structural steel fabrication drawings by Gayle Manufacturing

Scope for completing this form

We reviewed the as-built building drawings and carried out an ASCE 41-17 Tier 1 evaluation. We walked through the building on 20 November 2020 to confirm that the building generally matches the original drawings and to check for non-structural life-safety issues. We viewed some original project correspondence files at the office of Tipping Structural Engineers.

Brief description of structure

Mt. Zion Medical Office Building (MOB) 2 is a rectangular, low-slope roof building that provides space for clinics, offices, and a pathology laboratory. It has an overall floor area of 120,500 square feet, divided approximately equally between above-ground office space on 5 levels, and below-ground parking on 4 levels. The parking levels are each approximately 113'x136' in plan and abut the UCSF 2420 Sutter Street parking structure (CAAN #3062) to the west. There is access to the neighboring parking structure at the B1 level. Above ground, the building steps back 25' on the west side so that floors 1 through 3 are approximately 88'x136' in plan. Floors 1 through 3 are adjacent to a 30' tall building to the north where there is a 6" seismic separation. Floors 4 and 5 step back on the north side and are approximately 88' x 95' in plan. The upper roof supports 2 air handler units, enclosed along with stair and elevator equipment penthouses by a screen wall. The building's exterior is EIFS on steel studs. The overall height of the building is approximately 68' to the top of the 3'-6" parapet.

Steven Tipping + Associates structural engineers and the architectural firm of Kaplan McLaughlin Diaz designed the building in 1995. Plant Construction built the shell for Pacific Union Development in 1996.

<u>Identification of levels:</u> The main entry to the building is at ground level on the 1st floor. The remaining above-ground levels are floors 2 through 5. Below ground parking is on levels B1 through B4, B4 being the lowest level.

<u>Structural system for vertical (gravity) load:</u> Above the 1st floor, the typical floor and roof system is 2" deep metal decking with 2½" of lightweight concrete fill, supported on steel wide-flange composite beams spaced at up to 8'-9" on center, spanning north-south to steel wide-flange composite girders spaced at up 27'-6" on center, spanning to W10 or W14 columns at up 26'-4" on center. The 1st floor and floors B1 through B3 are primarily 9" thick 2-way concrete slabs supported by perimeter concrete walls and by 20" and 22" diameter concrete columns with drop panels.

<u>Foundation system:</u> The building has a 2' thick concrete mat slab foundation, the base of which is approximately 40' below adjacent grade. The mat slab is held down with an array of 1⅜" diameter by 35' long high-strength threaded bar tie-downs grouted in 12" diameter holes.

<u>Structural system for lateral forces:</u> Floors 2 through 5 and the upper roof are supported laterally by welded steel moment-resisting frames at the perimeter on all 4 sides of the building. There is a 5th 5-story moment frame oriented east-west at grid line 3, which is the north façade at the 4th and 5th floors. (See figure 2.) Lateral forces are transmitted to the moment frames by the concrete-filled metal decking at each level acting as a diaphragm.

Perimeter concrete walls and 2 interior concrete walls provide lateral support for the below-grade parking structure. The perimeter walls are 12" thick at level B1, and 14" thick at levels B2, B3, and B4. The interior concrete walls, which are on either side of the access ramps, are 8" thick.

Building condition

The building appears in generally good condition, and UCSF personnel noted that there are no current maintenance issues. There is rust on some rooftop mechanical equipment. There is evidence of moisture intrusion through the perimeter concrete walls at level B4.

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

Identified seismic deficiencies of the building include the following:

Structural feature or potential deficiency	Finding/notes
Welded moment connections	The steel moment-resisting frame connections use welded beam flange cover plates as shown in Figure 3 below. Because of the cover plates, this connection configuration is significantly stiffer and stronger than a typical pre-Northridge welded steel moment frame where the ends of the beam flanges are unreinforced. It is likely that the beam flange reinforcement will protect the connection by forcing flexural yielding in the beams to occur away from the welded connections. This connection configuration is different, however, from the prequalified welded flange plate connection in FEMA 350 (2000) and there is no prequalified welded flange plate connection in AISC 358-16. This may be because there are concerns about the reliability of this connection compared to the connection types that are prequalified. We expect that the ductility capacity of the connections is substantially greater than that of typical pre-Northridge connections, however there is still a potential for connection fracture under earthquake action.
Moment frame connection panel zone strength	The panel zones in 31 of 42 connections that were checked in this Tier 1 evaluation do not have enough capacity to resist 80% of the maximum shear demand that can placed on them by adjacent beams. Demand to capacity ratios were as high as 3.0. Panel zone yielding during a strong earthquake would lead to softening of the moment-resisting frames, increasing inter-story drifts and increasing the likelihood of damage, e.g., to cladding and partitions.
Inter-story drift	Inter-story drift ratios exceed the Tier 1 drift check 3% limit in the lower stories in the north-south direction. Increased inter-story drift would lead to increased building damage.
1st floor offset	Below the 1st floor, lateral resistance is provided by concrete walls, primarily at the perimeter of the parking areas at lines 1, 7, A, and F. The steel moment-resisting frame above the 1st floor on Line B (the west façade of the building) is in-line with an 8" thick concrete wall below that is of insufficient size to resist shear demands that could be delivered by the moment frame. The steel moment-resisting frame on Line 3 does not have a wall in line with it in the parking levels below. Shears from the Line 7 and Line 3 moment frames therefore are to be carried through the 1st floor slab to the below-grade perimeter walls. Diaphragm stresses at the 1st floor have not been evaluated as part of this Tier 1 study. Furthermore, the offset of the above-ground stories with respect to the parking floors below-ground results in plan torsion that effects wall shear demands. The Tier 1 quick check of wall shears indicates that seismic demand-capacity ratios are low, however, (0.2 to 0.5) indicating that this torsion effect will not adversely affect the building's seismic performance.
Site class D spectral shape	Per footnote 4, the earthquake demands are based on an F_v factor that does not include the requirements of Section 11.4.8-3 of ASCE 7-16. If such requirements were to be included, for this building with T=0.99 seconds (using ASCE 41-17 equation 4-4 based on building height), demands would increase by a factor of about 1.5. The Quick Check of inter-story drift ratios would then be noncompliant for all stories and directions, with values exceeding 7% at one location.

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

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

We did not identify any non-structural life-safety concerns during our walkthrough. There is a natural gas-fueled water heater at the upper roof level with seismic restraints, and there is an earthquake-activated automatic gas shutoff device at the main shutoff.

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

Discussion of rating

We rate the building as V (poor). There is a case to be made that the building could be IV (fair), but it would probably take a nonlinear dynamic analysis to convincingly demonstrate this. Beneficial features of the structure include the cover-plated beam connections that should have substantially more ductility than typical pre-Northridge connections. Column splices all use full-penetration welds (even for gravity columns); The structure passes the Tier 1 strong-column-weak-beam checks and, more importantly, our approximate assessment using plastic analysis shows that story mechanism formation is unlikely. Our evaluation found acceptable wall and moment-frame column stresses. The building's rectangular configuration and balanced moment frame and shear wall locations should contribute to good collapse-prevention seismic performance. Conversely, panel zones are weak, and the frame does not meet the story drift requirements of Tier 1.

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

Recommendations for further evaluation or retrofit

Further evaluation recommended?	Yes
Likelihood of showing better rating	Unlikely Possible Good chance
Likelihood of showing worse rating	Unlikely Possible Good chance
Evaluation needed to clarify the necessary retrofit scope?	Yes, it could be used to determine how much connection retrofitting or other strengthening is needed to meet IV.
Discussion of priority assignment	We suggest Priority B because retrofit would be disruptive and best accomplished along with remodeling or other work.

Peer review comments on rating

The structural members of the UCSF Seismic Review Committee (Lizundia, Moore, Phipps) reviewed the presentation of this evaluation on 8 January 2019, and they reviewed this report. The SRC is unanimous that a Seismic Performance Level Rating of V is appropriate.

SRC members had concerns about the column base plates (Figure 4) that do not provide full fixity for the columns. SRC members also felt that that the frame was light and were concerned about the weak panel zones and uncertainty about how effective the cover-plated connections would be.

Additional building data	Entry	Notes
Latitude	37.784567	
Longitude	- 122.440416	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	5	
Number of stories (basements) below lowest perimeter grade	4	Parking levels B1 through B4
Building occupiable area (OGSF)	120,515	From UCOP spreadsheet
Risk Category per 2016 CBC 1604.5	II	
Building structural height, h_n	65 ft	Structural height defined per ASCE 7-16 Section 11.2
Estimated fundamental period	0.99 sec	Estimated using ASCE 41-17 equation 4-4
Site data		
975 yr hazard parameters S_s, S_1	1.433, 0.558	
Site class	D	
Site class basis	Study	UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Site parameters F_a, F_v	1.0, 1.742[4]	
Ground motion parameters S_{cs}, S_{c1}	1.433, 0.972	
S_a at building period	0.985	
Site V_{s30}	308 m/s	
V_{s30} basis	Estimated	UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Liquefaction potential	No	

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

Liquefaction assessment basis	Study	UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Landslide potential	No	
Landslide assessment basis	Study	UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Active fault-rupture identified at site?	No	
Fault rupture assessment basis	Study	UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Site-specific ground motion study?	No	
Applicable code		
Applicable code or approx. date of original construction	Built: 1996 Code: 1991 UBC	Code identified on Sheet S1.1
Applicable code for partial retrofit	None	No partial retrofit known
Applicable code for full retrofit	None	No full retrofit known
Model building data		
Model building type	C2 Concrete shear walls S1 Steel moment frame	C2 at levels - (below ground) S1 at levels - (above ground)
FEMA P-154 score	0.8	
Previous ratings		
Most recent rating	IV	2013 report
Date of most recent rating	2013-10-07	Basis: qualitative assessment based on document review
2nd most recent rating	-	
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

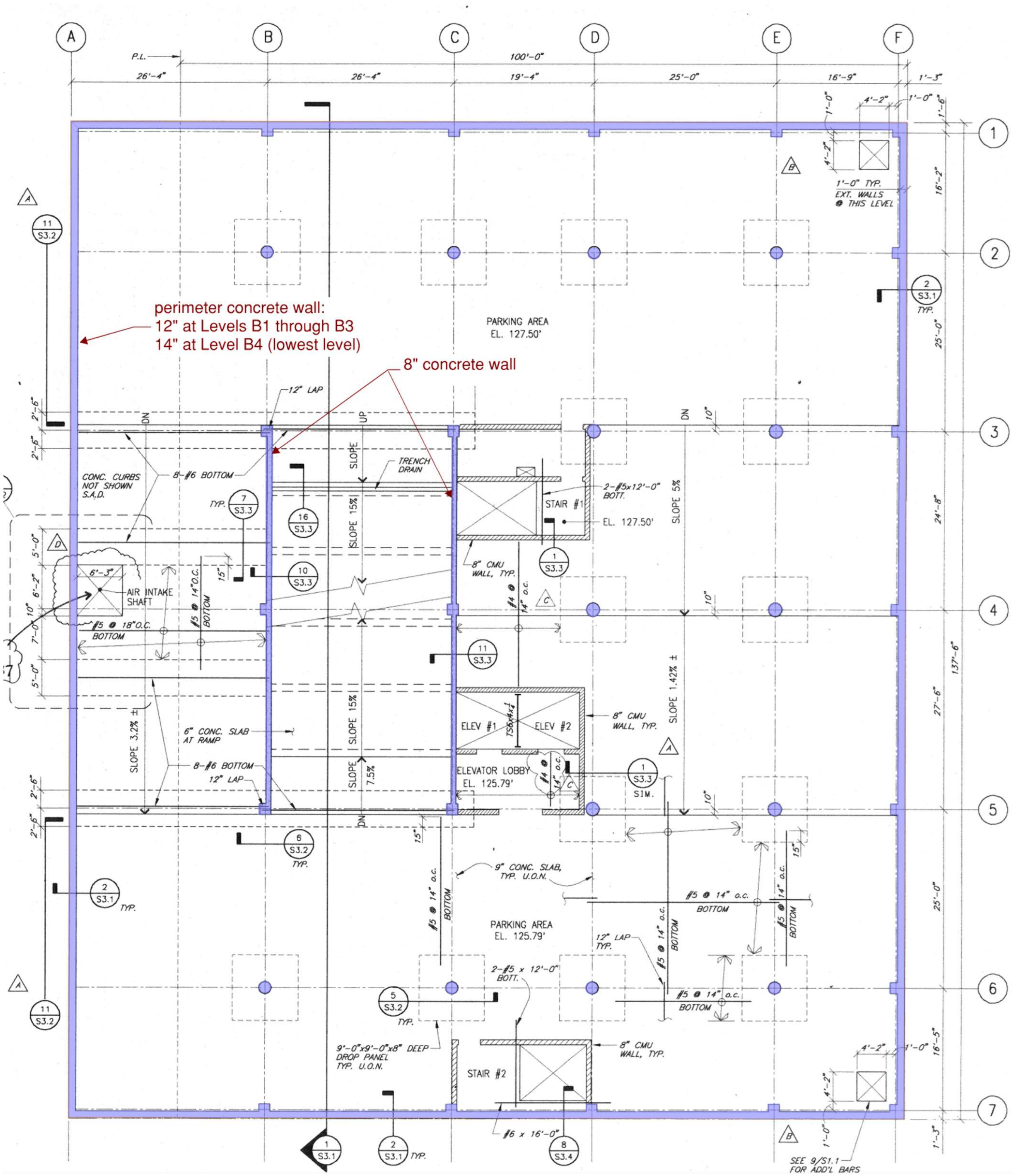


Figure 1: Concrete wall location plan at levels B1 through B4

North

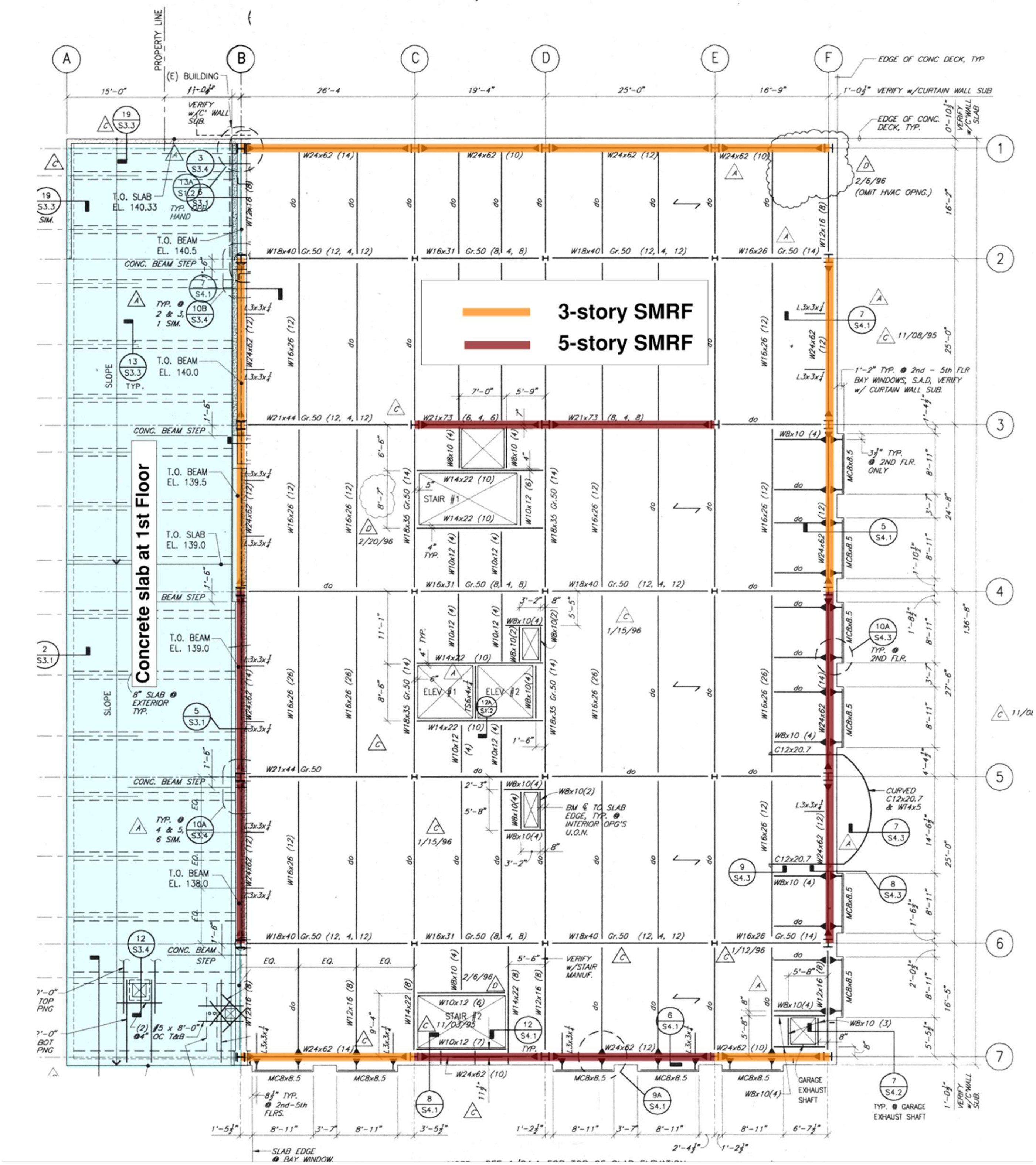
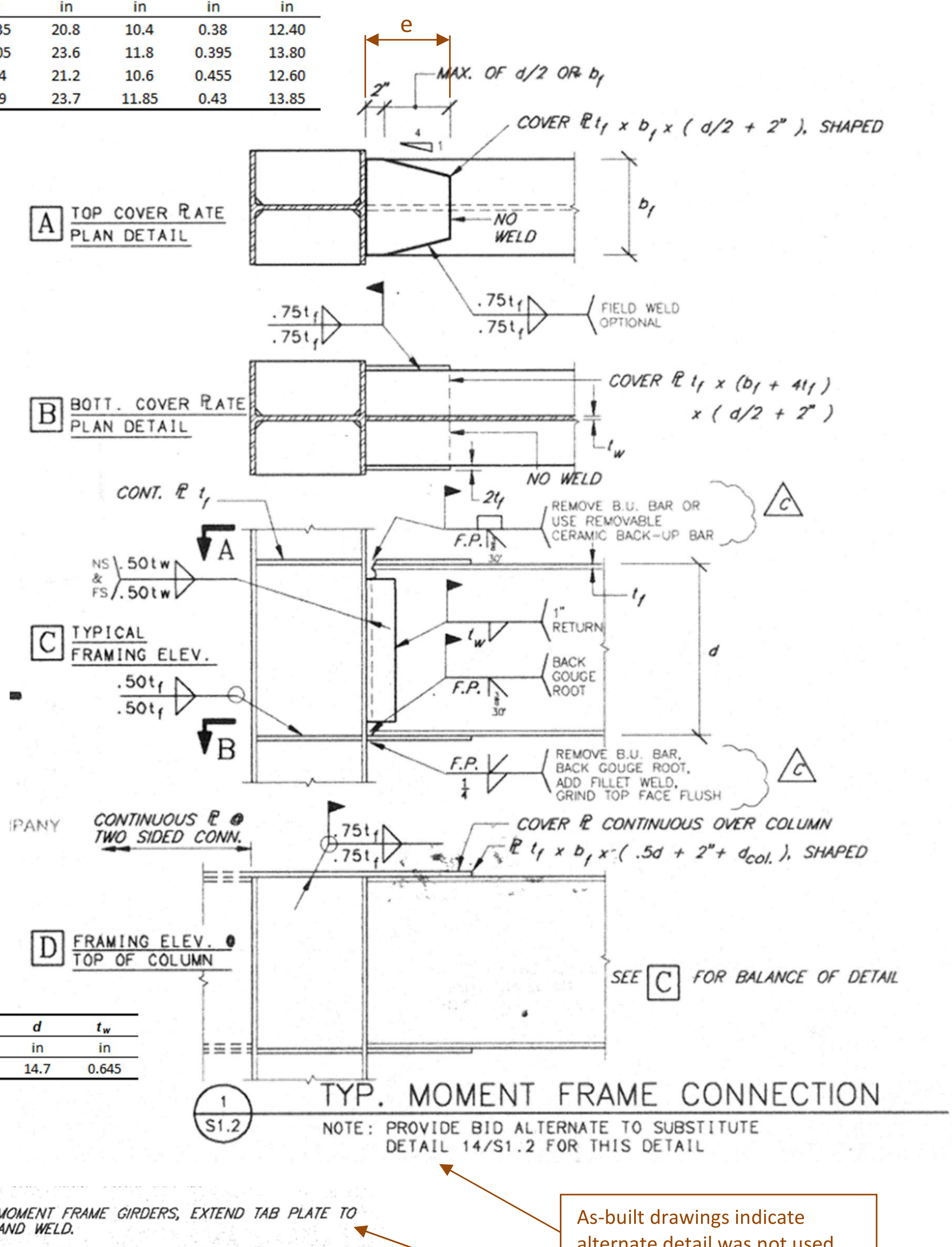


Figure 2: Moment frame location plan

North

beam section	b_f	t_f	d	$d/2$	t_w	e
	in	in	in	in	in	in
W21X50	6.53	0.535	20.8	10.4	0.38	12.40
W24X55	7.01	0.505	23.6	11.8	0.395	13.80
W21X73	8.3	0.74	21.2	10.6	0.455	12.60
W24X62	7.04	0.59	23.7	11.85	0.43	13.85



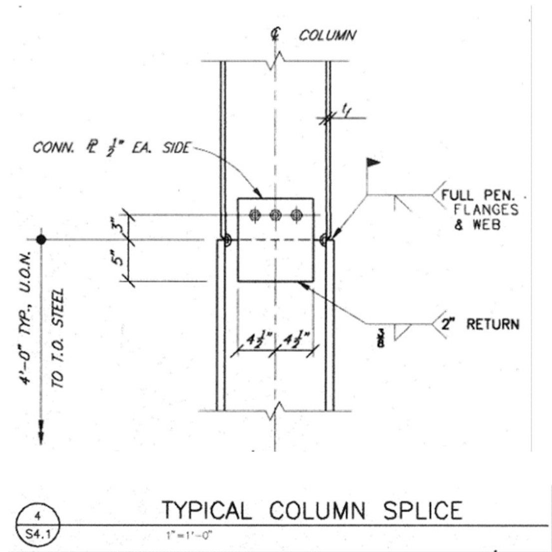
column section	b_f	t_f	d	t_w
	in	in	in	in
W14X132	14.7	1.03	14.7	0.645

As-built drawings indicate alternate detail was not used.

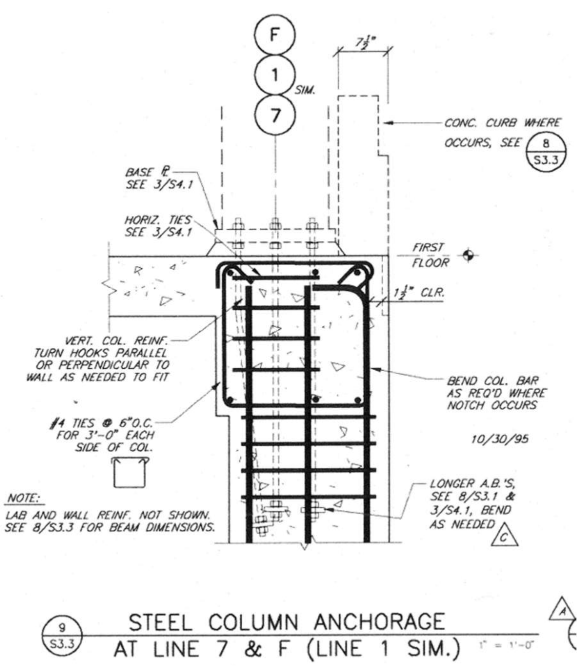
Note on shear tab detail 2/S1.2

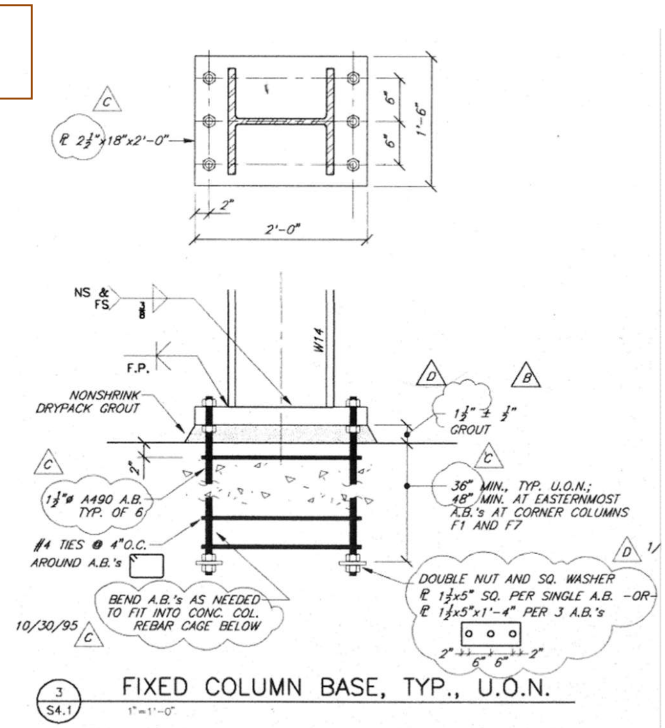
Figure 3: Moment frame beam-column connection

Column splices occur 4' above 3rd floor level



All moment columns: W14x132





Base plate detail at perimeter moment frames only. (Not used at Line 3.)

Figure 4: Moment column connection details

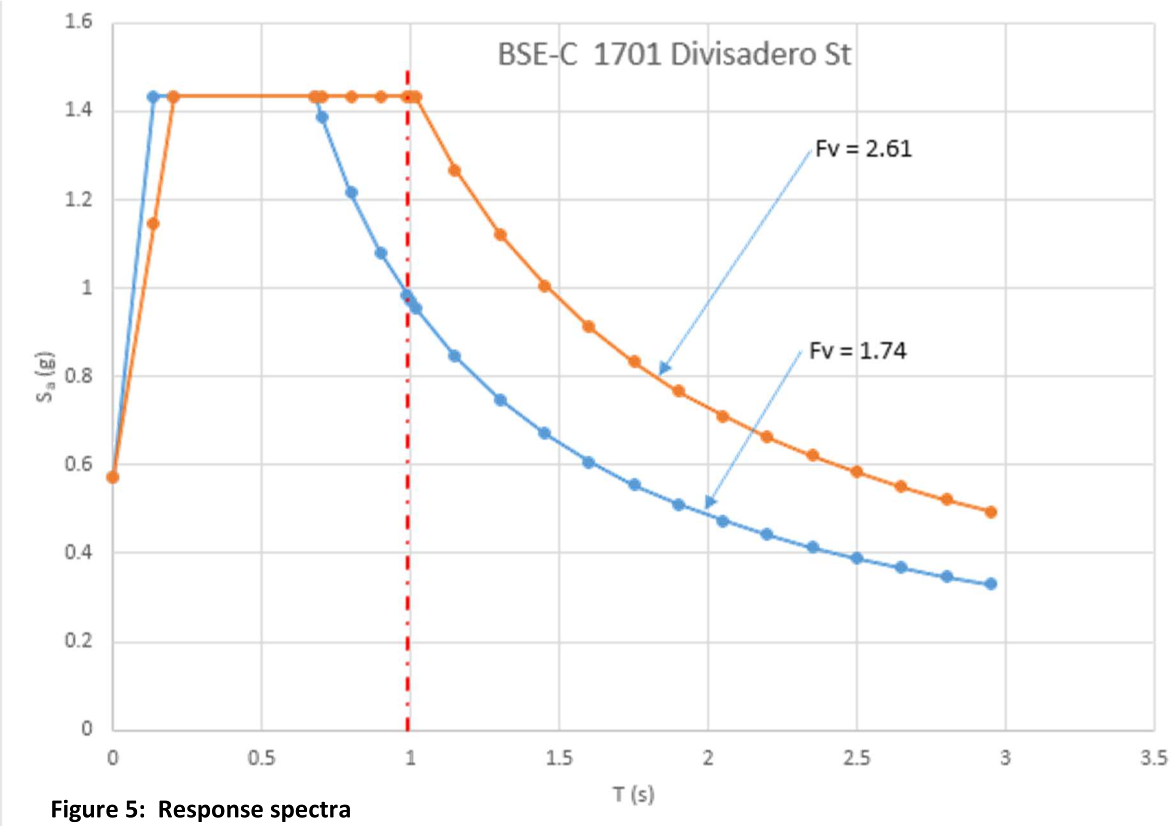


Figure 5: Response spectra

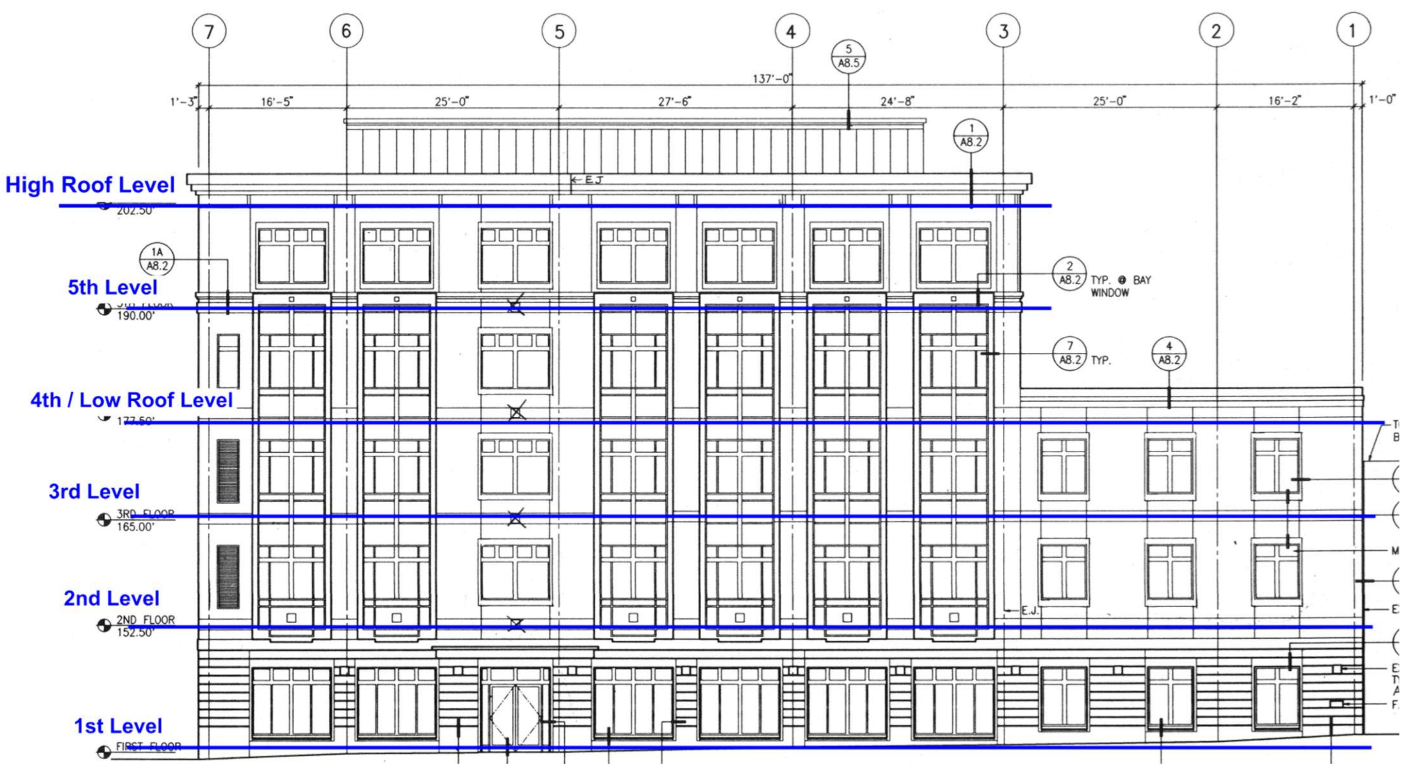


Figure 6: East elevation

Appendix 1: Project timeline with list of documents reviewed at Tipping Structural Engineering

Date	Item	Notes
1994-01-17	Northridge, CA EQ	
1994-09-XX	**SEAOC/AISC/ICBO emergency code change to 1994 UBC**	"2211.7.1.2 Connection Strength…abilty to sustain inelastic rotation … considering steel overstrength and strain hardening" …
1994-09-XX	**Steel Moment Frame Advisory No. 1**	**SAC** collections of papers and topical reports prepared by practicing engineers, building officials, industry groups, and researchers
1994-10-XX	**Steel Moment Frame Advisory No. 2**	
1995-01-17	Kobe, Japan—Great Hanshin EQ	
1995-02-01	**Steel Moment Frame Advisory No. 3**	**SAC 95-01**
1995-04-14	Permit Issued	
1995-07-31	Bid & Permit set issued	
1995-08-XX	**SAC Interim Guidelines**	**SAC 95-02 / FEMA 267**
1995-08-14	Peer Review Comments 100% CD submittal	
1995-08-17	Bid Addendum A	
1995-08-29	Bid Addendum B	
1995-09-27	Letter Degenkolb to UCSF PEER REVIEW OF STRUCTURAL DESIGN 1701 DIVISADERO MEDICAL CENTER BUILDING CONSTRUCTION DOCUMENTS PHASE UCSF/MOUNT ZION MEDICAL CENTER	1. SMRF Base plate demands 2. Panel zone rigidity 3. SMRF conn per SAC Interim Guidelines 4. Future roof deck 5. Construction dewatering
1995-10-02	Letter UCSF to PUDCo	
1995-10-02	Letter PUDCo to Tipping RE: 1701 ADDENDUM II, PEER REVIEW AND STRUCTURAL UPGRADES	Re Degenkolb backcheck to Addenda A & B
1995-10-02	Letter UCSF to PUDCo	Re Degenkolb backcheck to Addenda A & B
1995-10-12	**S.F. Bay Area Seminar on SAC Interim Guideleines**	
1995-10-19	Tipping letter to Pacific Union Response to 1995-09-27 Degenkolb comments	
1995-10-18	John Wolfe calcs moment conn calcs per Aug 95 SAC Interim Guidelines	
1995-11-29	Shop WPS submittal	+ Transmittals Plant → Tipping → AME
1995-12-15	AME review letter of Shop WPS	Mentions charpy V-Notch testing per Interim Guidelines for bottom flange cover plates
1995-12-19	Shop WPS Revised	w/ Tipping review stamp
1996-01-15	Structural final coordination set	
1996-02-19	Field WPS submittal	
1996-02-28	Letter Lincoln Electric to California Erectors re Charpy V-Notch testing	
1996-02-28	Letter Gayle Manufacturing to Plant re Charpy V-Notch testing for moment frame welds	
1996-03-04	Transmittal of revised WPS Plant to Tipping	
1996-03-05	Letter AME to Tipping	Approval of WPS
1996-03-05	Transmittal of Field WPS Tipping to Plant Construction	
1996-08-15	As-builts	
1996-08-29	Permit complete	
1997-01-XX	**SAC Interim Guideleines Advisory Number 1**	**FEMA 267A (superseded by Advisory No. 2)**
1999-06-XX	**SAC Interim Guideleines Advisory Number 2**	**SAC 99-01 / FEMA 267B**

Appendix 2: Story mechanism study

We performed two additional evaluations to learn more about this building's vulnerability to the formation of a story mechanism. (a) We calculated an index based on SEAOC Blue Book recommendations[5] and (b) we conducted a plastic mechanism analysis, comparing the plastic limit load of a beam mechanism to a story column mechanism. We looked at the potential for story mechanisms below Levels 2 and 5, for seismic forces acting in the building's longitudinal direction. (See Figures A2-1 and A2-2 below.) For both evaluations, we calculated cases with and without consideration of lateral resistance from the gravity columns (those without girder moment connections). We include moment frame columns acting weak-way (out of the plane of the moment frame) along with the gravity columns. With the plastic mechanism analysis, we compared a code-based (inverted triangular) vertical distribution of seismic forces to a uniform distribution.

(a) SEAOC Blue Book index:

The SEAOC Blue Book recommendation for avoidance of a story mechanism is:

$$\sum M_c \Big/ \sum M_b \geq 1$$

where:

$\sum M_c$ = sum of nominal flexural strengths of all columns framing into underside of the level

$\sum M_b$ = sum of nominal flexural strengths at each end of each beam at the level

To account for the reduction in column moment capacity from axial load, we multiply the plastic moment ($M_p = f_y Z$) by a factor of 0.8. This factor is based on our evaluation of the axial-moment interaction for a few sample columns, considering an axial load from the load case 1.0*D* + 0.5*E* (with *E* representing earthquake forces corresponding to the development of beam moment capacity). This 0.5 factor in this load case approximates that not all the columns will experience maximum earthquake forces simultaneously.

We also account for the weak base plates by using the moment strength of the six-anchor base plate instead of the flexural capacity of the columns at the column base at Level 1.

We used an amplification factor of 1.1 for the moment-frame beams to account for composite floor slabs, and a reduction factor of 0.3 times the flexural strength of the full beam section to estimate the moment capacity at shear tab connections. Therefore, the beam flexural strength is taken as $1.1M_p$ at moment connections and as $0.3M_p$ at shear tab connections. Both these factors are average approximations that consider that a beam framing into one side of a column will be in positive moment (with greater composite contribution) while the beam on the opposite side will be in negative moment (with less composite contribution).

Resulting index values are:

Case	Below Level 2	Below Level 5
Including gravity columns	1.03	1.36
Neglecting gravity columns	0.72	0.86

We believe that the results that include gravity columns are a better representation of the actual behavior

(b) Plastic mechanism analysis:

The criterion for the plastic mechanism analysis is:

$$V_{story\ mech} \Big/ V_{beam\ me} \geq 1$$

where:

$V_{story\ mech}$ = base shear corresponding to the development of a story mechanism

$V_{beam\ mech}$ = base shear corresponding to the development of a beam mechanism

[5] Structural Engineers Association of California, "Recommended Lateral Force Requirements and Commentary" Section C402.5, September 1999

We use the same moment capacity assumptions and methods as in the calculation of the Blue Book Index. A value greater than one for this criterion indicates that a story mechanism is less likely than a beam mechanism to occur.

Resulting values for this criterion are:

Case	Below Level 2	Below Level 5
Code distribution including gravity columns	1.61	2.61
Uniform distribution including gravity columns	1.29	3.57
Code distribution neglecting gravity columns	1.04	1.89
Uniform distribution neglecting gravity columns	0.82	2.59

Conclusions

These evaluations indicate that a story mechanism is unlikely as none of the cases that include gravity columns indicate a likely story mechanism. Also, our results show that the building setback that occurs above Level 4 does not result in a story mechanism vulnerability above the setback.

The analyses above are an approximation of expected behavior, made to estimate the extent to which gravity columns will benefit the structural performance related to story mechanisms. We only assess the performance in the building longitudinal direction, but we observe that the calculation results would be similar in the transverse direction. These approximate analyses assume fully ductile behavior, but in reality, the ductility capacity of the beam moment connections (which have cover plates) is uncertain. The likelihood and effect of connection fracture is uncertain; nonlinear dynamic analysis could provide insight.

Nevertheless, the analyses summarized here show that the gravity columns, plus moment frame columns acting weak-way, provide substantial protection against a story mechanism in the building, at least prior to any potential widespread fracture of connections.

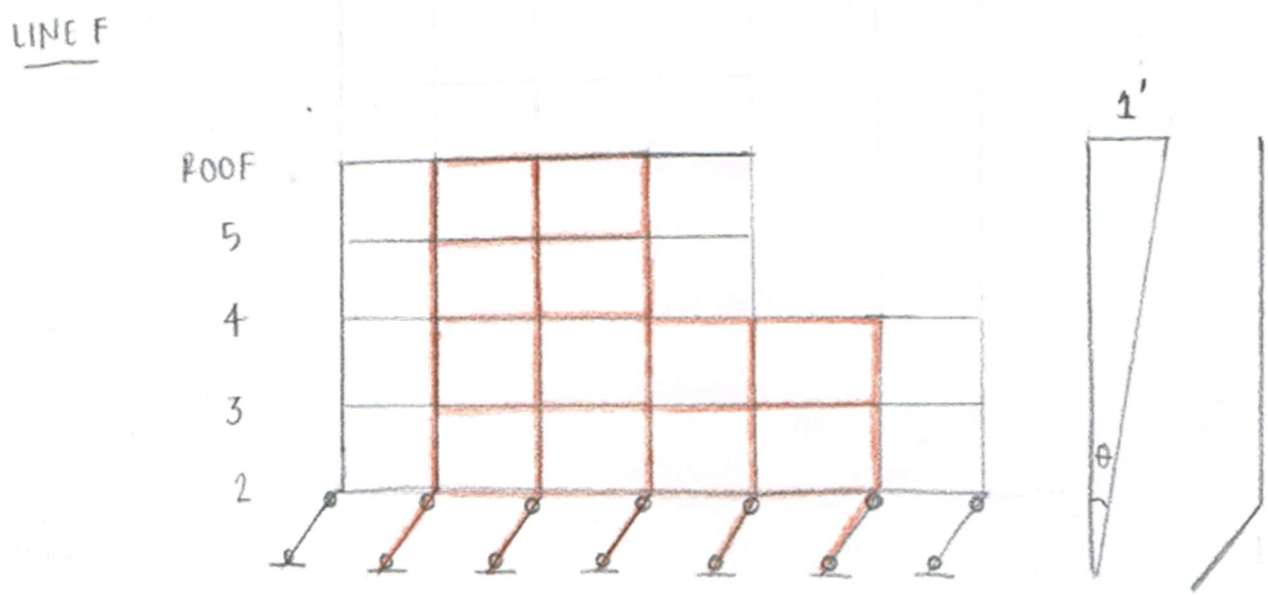


Figure A2-1: Story mechanism below level 2

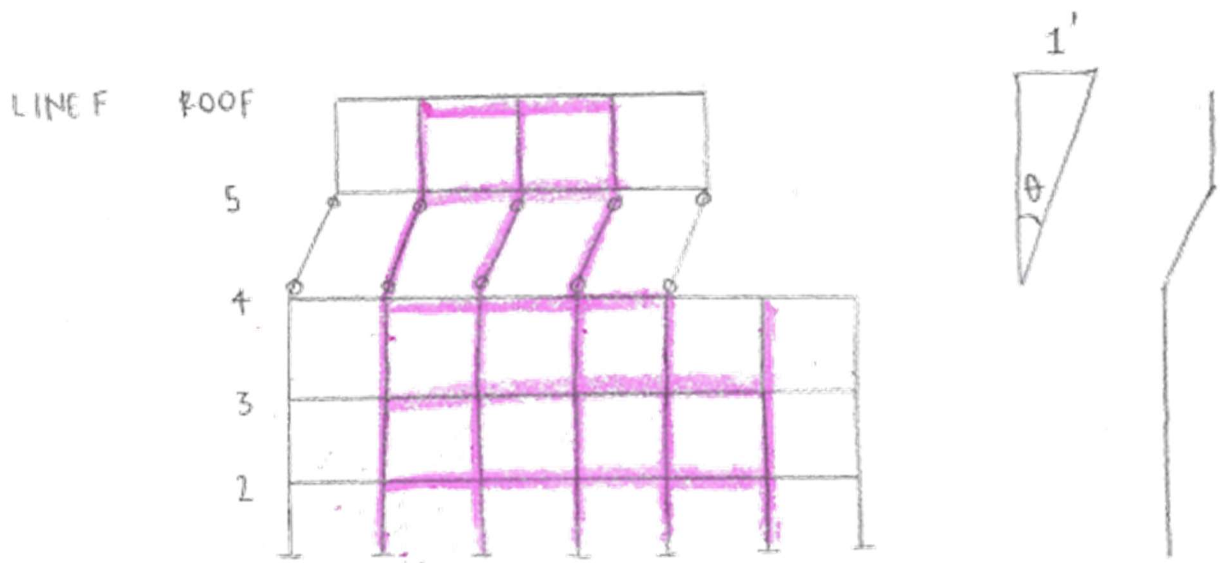
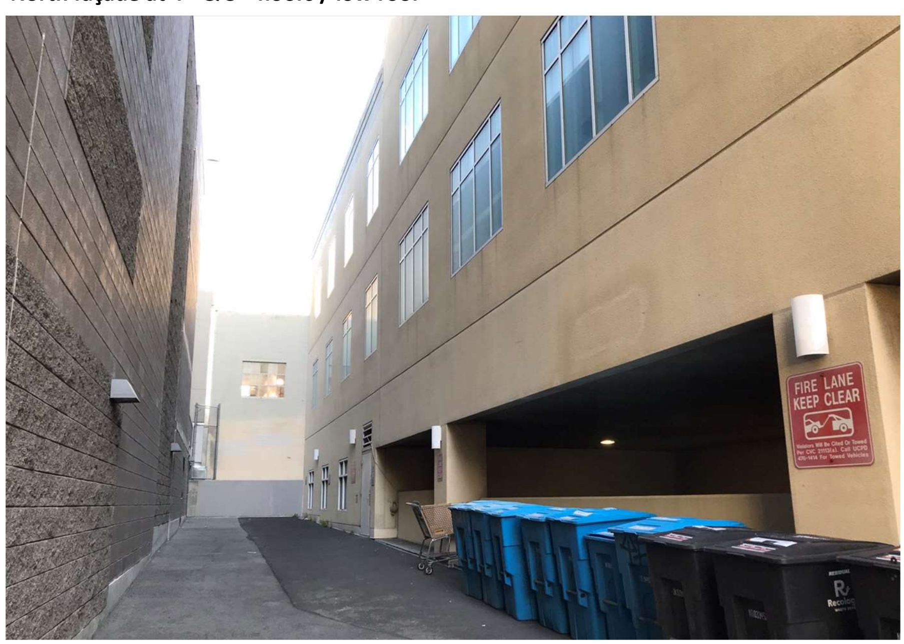


Figure A2-2: Story mechanism below level 5

East and north façades

North façade at 4th & 5th floors / low roof

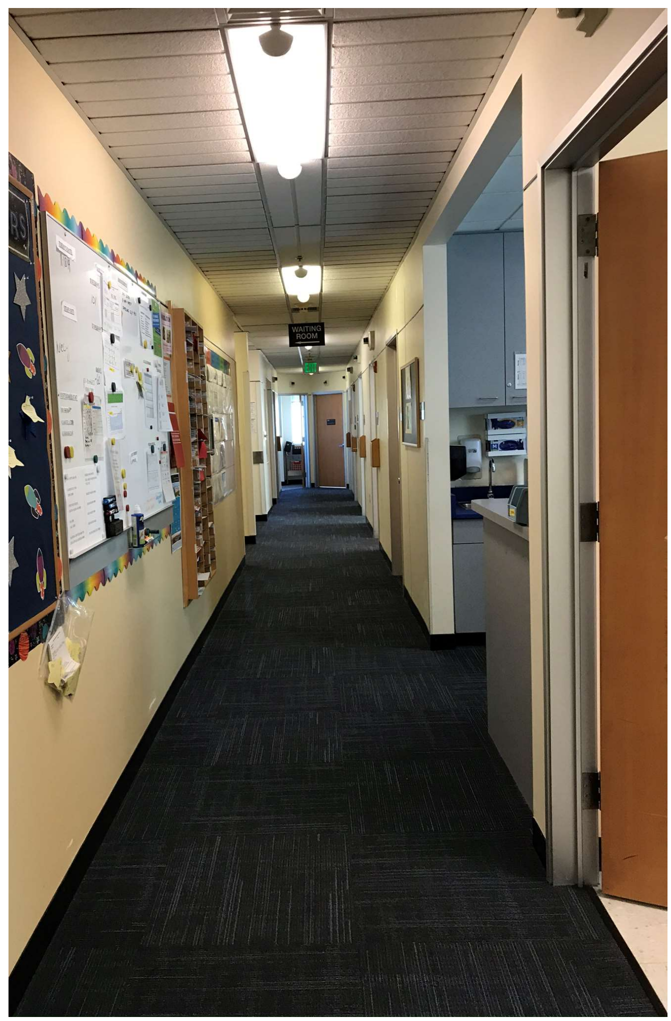
Covered seismic gap at NW corner of building



West façade, alleyway over B1 parking level, adjacent parking structure to the left

Lobby

Typical examination room



Typical hallway

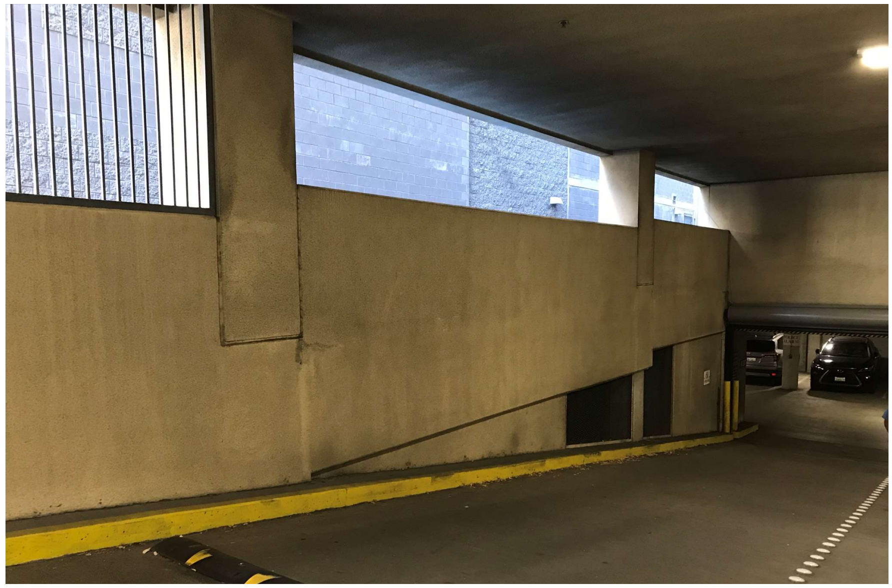
Earthquake-activated
automatic gas shutoff

Gas main service shut off – Level B1

Gas-fired domestic water heater – high roof

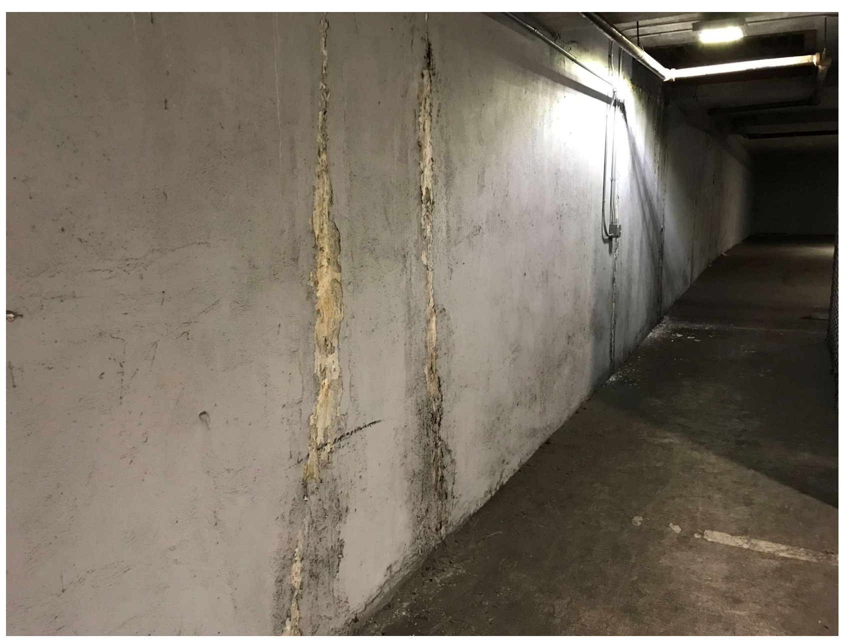
Gas-fired HVAC boiler – high roof

Vehicle ramp to Level B1



Connection to 2420 Sutter Street parking structure at Level B1

Level B4 perimeter wall with evidence of moisture intrusion



UC Campus:	San Francisco		Date:	12/10/2019		
Building CAAN:	**2036**	Auxiliary CAAN:	By Firm:	**MSE**		
Building Name:	Mt. Zion 1701 Divisadero, MOB 2		Initials:	*RBW*	Checked:	*JM*
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ASCE 41-17
Collapse Prevention Basic Configuration Checklist

LOW SEISMICITY

BUILDING SYSTEMS - *GENERAL*

C NC N/A U	Description
C (selected)	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1) **Comments:**
C (selected)	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:** The height of the adjacent building to the north is ~ 32 ft per elevation 2/A3.1. Seismic gap is 6” based on field observation and detail 5/A8.2. 6” > .015·32·12” = 5.8”
N/A (selected)	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3) **Comments:**

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

C NC N/A U	Description
C (selected)	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1) **Comments:**
C (selected)	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2) **Comments:**

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

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C NC N/A U	Description
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:** There are concrete columns or pilasters below all moment frame columns.
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:** In the transverse direction, the change from the 1st floor to B1 is 115’/88.7’ = 1.30
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:** 5th floor level to 4th floor level: 867 kips/561 kips = 1.54
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:** Each story is rectangular in plan with matching lateral load resisting elements on all 4 sides.

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

GEOLOGIC SITE HAZARD

	Description
C ● NC ○ N/A ○ U ○	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building’s seismic performance do not exist in the foundation soils at depths within 50 ft (15.2m) under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1) **Comments:** Per Egan report
C ● NC ○ N/A ○ U ○	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1) **Comments:** Per Egan report

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

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

GEOLOGIC SITE HAZARD

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

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

FOUNDATION CONFIGURATION

	Description
C NC N/A U (●) ( ) ( ) ( )	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3) **Comments: At ground level:** 87.5'/65.0' = 1.35 > 0.6·0.985 = 0.59
C NC N/A U ( ) ( ) (●) ( )	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4) **Comments:** Foundation is a 2' thick mat slab.

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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type C2-C2A

Low And Moderate Seismicity

Seismic-Force-Resisting System

	Description
C NC N/A U (C selected)	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:**
C NC N/A U (C selected)	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:**
C NC N/A U (C selected)	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:** Quick check DCRs are 0.5 or less
C NC N/A U (C selected)	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:** per section 2/S3.1 & detail 10/S3.3

wall thickness (in)	Vert. reinf.	%	horiz. Reinf.	%
8	#5 @ 12”	.0032	# 5 @ 12”	.0032
12	#5 + #4 @ 12”	.0035	#4 + #4 @ 12”	.0028
14	#6 + #5 @ 12”	.0045	#5 + #5 @ 16”	.0028

Connections

	Description
C NC N/A U (N/A selected)	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:**

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

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type C2-C2A

C (selected) **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:** section 2/S3.1
C (selected) **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:** section 2/S3.1

High Seismicity (Complete The Following Items In Addition To The Items For Low And Moderate Seismicity)

Seismic-Force-Resisting System

	Description
C (selected) **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:** See calculations - round concrete columns have sufficient shear capacity.
C (selected) **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:** Detail 5/S3. 2 calls out #4 @ 18" bottom bars in the 9' wide drop panels, so there would be 7 bars, with the middle bar centered on the column.
C (selected) **NC N/A U**	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:**

Diaphragms (Stiff Or Flexible)

	Description
C (selected) **NC N/A U**	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:**

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Collapse Prevention Structural Checklist For Building Type C2-C2A

C (selected) **NC** **N/A** **U**	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:**

Flexible Diaphragms

	Description
C **NC** **N/A** (selected) **U**	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) **Comments:**
C **NC** **N/A** (selected) **U**	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2) **Comments:**
C **NC** **N/A** (selected) **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:**
C **NC** **N/A** (selected) **U**	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) **Comments:**
C **NC** **N/A** (selected) **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:**

Connections

	Description
C **NC** **N/A** (selected) **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:**

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

LOW SEISMICITY

SEISMIC-FORCE-RESISTING SYSTEM

	Description
C NC N/A U C ●	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1) **Comments:** There are 2 moment frame lines in the N-S direction and 3 in the E-W direction.
C NC N/A U NC ●	DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.4.3.1, is less than 0.030. (Commentary: Sec. A.3.1.3.1. Tier 2: Sec. 5.5.2.1.2) **Comments:** Drift ratios at the 3rd and 2nd floors are 0.037 and 0.050 respectively based on the Quick Check procedure.
C NC N/A U C ●	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3) **Comments:** Checked using the Section 4.4.3.6 Quick Check procedure.
C NC N/A U C ●	FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.4.3.9, is less than F_y. Columns need not be checked if the strong column–weak beam checklist item is compliant. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.2) **Comments:** Checked using the Section 4.4.3.9 Quick Check procedure.

CONNECTIONS

	Description
C NC N/A U C ●	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:** ¾” diameter Nelson studs (detail 5/S1.2) occur at all moment frame griders. Steel decking is welded to steel frames per detail 6/S1.2.
C NC N/A U C ●	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1) **Comments:** See details 3/S4.1 and 9/S3.3 for fixed column base connections. (Included in report in figure 4.) See detail 2/S4.1 for pinned base moment frame columns on line 3.

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

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

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

SEISMIC-FORCE-RESISTING SYSTEM

C NC N/A U	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:**
◉ 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 or CMU infill walls at floors 1 through 5.
◉ C ○ NC ○ N/A ○ U	MOMENT-RESISTING CONNECTIONS: All moment connections can develop the strength of the adjoining members based on the specified minimum yield stress of steel. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1). **Comments:** Full-penetration flange welds considered non-compliant at Tier 1 per A3.1.3.4

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

SEISMIC-FORCE-RESISTING SYSTEM

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:** Flange cover plates are used, per detail 1/S1.2, that are either 1.0x or 1.5x the girder flange thickness. These, along with the girder flanges, have full-penetration welds to the columns.
○ 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:** This is non-conforming at the majority of panel zones, with DCRs as high as 3.0

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

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

C ◉ **NC** ○ **N/A** ○ **U** ○	COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web. (Commentary: Sec. A.3.1.3.6. Tier 2: Sec. 5.5.2.2.3) **Comments:** See detail 4/S4.1 ((Included in report in figure 4.)
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:** See calculations. Lowest percentage (levels 2 & 3 in frame on line 7) is 60% > 50%
C ◉ **NC** ○ **N/A** ○ **U** ○	COMPACT MEMBERS: All frame elements meet section requirements in accordance with AISC 341, Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.4) **Comments:** See calculations.

DIAPHRAGMS (STIFF OR FLEXIBLE)

	Description
C ◉ **NC** ○ **N/A** ○ **U** ○	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3) **Comments:** Line 7, 2nd floor (stair #2 and vent shaft openings:) (13.8+4)/87.4 = 20% Line 3, 2nd floor (garage exhaust shaft) 7/44.3 = 16%

FLEXIBLE DIAPHRAGMS

	Description
C ○ **NC** ○ **N/A** ◉ **U** ○	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) **Comments:**
C ○ **NC** ○ **N/A** ◉ **U** ○	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2) **Comments:**
C ○ **NC** ○ **N/A** ◉ **U** ○	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) **Comments:**

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

C NC N/A U	
○ ○ ◉ ○	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) **Comments:**
C NC N/A U ○ ○ ◉ ○	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5) **Comments:**

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UCOP SEISMIC SAFETY POLICY
Falling Hazard Assessment Summary

P	N/A	Description
☐	☒	**Heavy ceilings, features or ornamentation above large lecture halls, auditoriums, lobbies, or other areas where large numbers of people congregate (50 ppl or more)** **Comments:** none observed
☐	☒	**Heavy masonry or stone veneer above exit ways or public access areas** **Comments:** none observed
☐	☒	**Unbraced masonry parapets, cornices, or other ornamentation above exit ways or public access areas** **Comments:** none observed
☐	☒	**Unrestrained hazardous material storage** **Comments:** none observed
☐	☒	**Masonry chimneys** **Comments:** none observed
☐	☒	**Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.** **Comments:** There is a gas-fired water heater at the upper roof level with seismic restraints, and there is an earthquake-activated automatic gas shutoff device at the main shutoff.
☐	☐	**Other:** **Comments:**

Falling Hazards Risk: Low

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

SEISMIC EVALUATION OF EXISTING BUILDINGS - TIER 1 SCREENING

ASCE 41-17 Chapter 4

General (parentheses indicate ASCE 41-17 reference)

Building	Mt. Zion MOB 2, 1701 Divisadero		
Architect	Kaplan McLaughlin Diaz		
Structural Engineer	Steven Tipping + Associates		
Location	1701 Divisadero St., San Francisco, CA 94115		
Design date	1995		
Latitude	37.7855		Google Earth
Longitude	-122.4402		
Stories above grade	5		

Seismic parameters

Risk Category	II		CBC 2016 Table 1604.5
Site Class	D		Egan report
Liquefaction hazard	Very Low		Egan report
S_{cS}	1.433	g	Egan report
S_{c1}	0.972	g	Egan report

Scope

Performance level	CP	(4.1.1, Table 2-1)
Seismic hazard level	BSE-C	(4.1.2, Table 2-1)
Level of seismicity	High	(4.1.3, Table 2-5)
Building type	S1: Steel moment frames with stiff diaphragms	(4.2.2, Table 3-1)
	C2: Concrete shear walls with stiff diaphragms	

Material properties

				Notes	
Steel	F_y	50	ksi	SMRF cols A572 Gr 50	(Table 4-5)
Steel	F_y	49	ksi	SMRF bms A36	(Table 4-5)
Steel	E	29000	ksi		(4.2.3)
Concrete	f'_c	4000	psi	Walls	
Reinf.	f_y	60000	psi		

Checklists

Benchmark building	No	UCOP Seismic Program Guidebook v. 1.3 Table 1
Checklist(s) req'd	ASCE 41-17 Collapse Prevention Structural Checklist for Building Type S1	
	ASCE 41-17 Collapse Prevention Structural Checklist for Building Type C2	
	ASCE 41-17 Collapse Prevention Basic Configuration	
	UCOP SEISMIC SAFETY POLICY Falling Hazard Assessment Summary	

Seismic forces

V	3751	kip	$V = CS_aW$	= 0.98W	(Eq. 4-1)
W	3810	kip	building weight		(4.5.2.1)
C	1.0				(Table 4-7)
S_a	0.985	g	$S_a = S_{x1}/T \leq S_{XS}$		(Eq. 4-3)
T	0.99	sec	$T = C_t h_n^{\beta}$		(Eq. 4-4)
C_t	0.035				(4.4.2.4)
β	0.80				(4.4.2.4)
h_n	65.0	ft	building height		

Story forces

Above ground

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

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

k 1.24 k = 1.0 for T < 0.5, 2.0 for T > 2.5, linear interpolation between (4.4.2.2)

						(4-2a)	(4-2b)	4 - 5	1 -3
Level	w	story ht	h	wh^k	F_{story}	F_{story}	V_{story}	M_{OT}	M_{OT}
	kip	ft	ft			kip	kip	kip·ft	kip·ft
high roof	593		65.0	106535	0.30	1142			
5	561	12.50	52.5	77247	0.22	828	1142	14277	
4/ low roof	867	12.50	40.0	85168	0.24	913	1970	38907	
3	889	12.50	27.5	54797	0.16	587	2883		36043
2	900	12.50	15.0	26120	0.07	280	3471		79430
1		15.00	0.0				3751		135694
totals	3810			349867	1.0	3751			

Below ground

$PGA = 0.4 \cdot S_{xs}$

$F_{story} = PGA \cdot w$

$V_{story} = \Sigma_{above} F_{story}$

Level	w	story ht	h	PGA	F_{story}	V_{story}
	kip	ft	ft	g	kip	kip
1	3065			0.57	1757	
B1	2869	10.00	-10	0.57	1645	5508
B2	2790	8.59	-19	0.57	1599	7152
B3	2920	8.58	-27	0.57	1673	8752
B4		8.91	-36			10425

Shear stress in shear walls

			(4-9)	(4-9)			
Story	$A_{w\,N\text{-}S}$	$A_{w\,E\text{-}W}$	v_{NS}^{avg}	v_{EW}^{avg}	D/C_{NS}	D/C_{EW}	*Notes*
	in^2	in^2	psi	psi			
B1	39427	33120	31	37	0.2	0.3	stress check w/ full story force ignoring 8" Line B & C 8" walls (cons.)
B2	46166	38640	34	41	0.3	0.3	
B3	46166	38640	42	50	0.3	0.4	
B4	46166	38640	50	60	0.4	0.5	
Total							

M_s 4.5 (Table 4-8)

v_{limit} 126 psi $v_{limit} = 2\sqrt{f_c'} \geq 100$ psi, f'_c is spec'd strength

$v^{avg} = (1/M_s)(V_{story}/A_w)$ (4-9)

Shear strength of concrete columns at parking levels dets 1 & 5/S3.2

check that $\phi V_n > 2\phi M_n/L$, ignoring axial load

Grid	Level	Diameter	L	spiral	long	ϕM_n	$2\phi M_n/L$	d	b_w	V_c	V_s	ϕV_n	$\phi V_n >$
		in	in			k ft	k	in	in	kip	kip	kip	$2\phi M_n/L$?
D4	B1	22	103	#4 @ 6"	8-#8	205.8	48.0	17.6	22.0	49.0	70.4	89.5	yes
	B2	22	86.08	#4 @ 6"	8-#8	205.8	57.4	17.6	22.0	49.0	70.4	89.5	yes
	B3	22	86.08	#4 @ 6"	8-#10	263.6	73.5	17.6	22.0	49.0	70.4	89.5	yes
D2	B1	20	103	#4 @ 6"	8-#8	171.9	40.1	16.0	20.0	40.5	64.0	78.4	yes
	B2	20	86.08	#4 @ 6"	8-#8	171.9	47.9	16.0	20.0	40.5	64.0	78.4	yes
	B3	20	86.08	#4 @ 6"	8-#10	213.4	59.5	16.0	20.0	40.5	64.0	78.4	yes

Drift check (4.4.3.1)

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

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

direction	story	column section	I_c in^4	h in	k_c	beam section	I_b in^4	L in	k_b	V_{story} kip	n_col	V_c kip	D_r
E-W	5	W14X132	1530	150	10.2	W24x55	1350	270	5.0	1142	6	190	0.024
	3	W14X132	1530	150	10.2	W24X62	1550	262.25	5.9	2883	13	222	0.026
	2	W14X132	1530	150	10.2	W24X62	1550	262.25	5.9	3471	13	267	0.031
N-S	5	W14X132	1530	150	10.2	W27x94	3270	313	10.4	1142	6	190	0.016
	3	W14X132	1530	150	10.2	W24X62	1550	306.5	5.1	2883	10	288	0.037
	2	W14X132	1530	165	9.3	W24X62	1550	306.5	5.1	3471	10	347	0.050

Column axial stress check (4.4.3.6)

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

direction	E/W	N/S		
V	3751	3751	kip	
n_f	3	2		total no.of frames in the direction of loading
h_n	65.0	65.0	ft	
L	87.4	102.2	ft	
col_sec	W14X132	W14X132		end column section
A_{col}	38.80	38.80	in^2	
M_s	2.5	2.5		CP
p_{ot}	6.39	8.20	ksi	
F_y	50	50	ksi	(Table 4-5)
$P_{ot} < 0.1F_y$	YES	YES		

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

check 2 representative SMRFs

Line 6 (south façade, transverse direction)

Line F (east façade, longitudinal direction)

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0$$ AISC 341-16 Eq E3-1

$$\sum M_{pc}^* = \sum Z_c (F_{yc} - \alpha_s P_r / A_g)$$ AISC 341-16 Eq E3-1

All SMRF columns: W14X132

Z_c	234	in^3
A_g	38.8	in^2
d_c	14.7	in
t_w	0.645	in
α_s	1.0	(factored loads)
P_r	$P_{grav} + P_{eq}$	

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

Frame			6					F		
Col	B	C	D	E	F	6	5	4	3	2
5		17.4	16.8	15.9		16.0	20.3	20.2		
4		36.1	35.1	33.0		33.4	42.3	42.0		
3	35.6	57.2	55.6	52.3	23.6	52.8	67.0	66.6	44.6	16.2
2	49.0	78.7	76.4	72.0	40.7	72.7	92.2	91.6	68.4	35.9
1	75.4	121.1	117.6	110.8	42.6	96.7	117.2	106.6	73.5	40.2

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

Frame			7					F		
I_{frame} (ft^4)			1300.5					1773.3		
Col	B	C	D	E	F	6	5	4	3	2
$\overline{x}$ (ft)	46.02	19.68	0.35	24.65	41.40	51.37	26.37	1.13	25.80	50.80
5		153.3	13.6	167.0		133.7	4.3	138.0		
4		417.8	37.2	455.0		364.4	11.8	376.1		
3	154.6	484.0	38.4	537.8	139.1	505.0	84.0	379.2	70.6	139.1
2	191.3	563.6	39.8	637.6	191.3	674.4	170.9	383.0	155.7	306.6
1	666.8	666.8	41.6	766.9	523.8	893.9	283.6	387.8	266.0	523.7

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

Frame			6					F		
Col	B	C	D	E	F	6	5	4	3	2
5		170.7	30.5	182.8		149.7	24.6	158.2		
4		454.0	72.3	488.1		397.8	54.1	418.2		
3	190.3	541.2	93.9	590.2	162.7	557.9	150.9	445.8	115.2	155.3
2	240.3	642.3	116.2	709.5	232.0	747.1	263.0	474.6	224.1	342.4
1	742.2	788.0	159.2	877.6	566.4	990.7	400.8	494.4	339.5	563.9

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

Frame										
Col	B	C	D	E	F	6	5	4	3	2
5		10671	11516	10597		10797	11552	10746		
4		8962	11264	8757		9301	11374	9178		
3	10553	8436	11134	8141	10719	8335	10790	9011	11005	10764
2	10251	7826	10999	7421	10301	7195	10114	8838	10349	9635
1	7224	6948	10740	6407	8284	5725	9283	8718	9653	8299

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

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

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

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

beam sections

Frame	7				F			
level / bay	B - C	C - D	D - E	E - F	6 - 5	5 - 4	4 - 3	3 - 2
L (ft)	26.33	19.33	24.30	16.75	25.00	25.23	24.67	25.00
5		W24X55	W24X55		W24X55	W24X55		
4	W24x62	W24x62	W24x62	W24x62	W24X55	W24X55	W24X55	W24X55
3	W24x62	W24x62	W24x62	W24x62	W24x62	W24x62	W24x62	W24x62
2	W24x62	W24x62	W24x62	W24x62	W24x62	W24x62	W24x62	W24x62

beam	Z_{bx}	e	$d_c/2$+e
	in^3	in	in
W21X50	110	12.40	19.75
W24X55	134	13.80	21.15
W24x62	153	13.85	21.20

e distance is per detail 1/S2.1

M_{pr} kip·in

Frame	7				F			
level / bay	B - C	C - D	D - E	E - F	6 - 5	5 - 4	4 - 3	3 - 2
5		6566	6566		6566	6566		
4	7497	7497	7497	7497	6566	6566	6566	6566
3	7497	7497	7497	7497	7497	7497	7497	7497
2	7497	7497	7497	7497	7497	7497	7497	7497

M_v kip·in

Frame	7				F			
level / bay	B - C	C - D	D - E	E - F	6 - 5	5 - 4	4 - 3	3 - 2
5		1464	1114		1078	1066		
4	1162	1677	1276	2004	1078	1066	1095	1078
3	1162	1677	1276	2004	1234	1221	1253	1234
2	1162	1677	1276	2004	1234	1221	1253	1234

M_{pb} kip·in

Frame	7				F			
level / bay	B - C	C - D	D - E	E - F	6 - 5	5 - 4	4 - 3	3 - 2
5		8030	7680		7644	7632		
4	8659	9174	8773	9501	7644	7632	7661	7644
3	8659	9174	8773	9501	8731	8718	8750	8731
2	8659	9174	8773	9501	8731	8718	8750	8731

$\sum M^*_{pc}$ kip·in

Frame	7					F				
level/col	B	C	D	E	F	6	5	4	3	2
5		19633	22780	19354		20098	22925	19924		
4	10553	17398	22398	16897	10719	17637	22164	18189	11005	10764
3	20803	16262	22133	15561	21019	15530	20903	17849	21354	20398
2	17474	14774	21739	13828	18585	12920	19397	17556	20001	17934

$\sum M^*_{pb}$ kip·in

Frame	7					F				
level/col	B	C	D	E	F	6	5	4	3	2
5		8030	15710	7680		7644	15276	7632		
4	8659	17832	17946	18274	9501	7644	15276	15293	15305	7644
3	8659	17832	17946	18274	9501	8731	17449	17468	17481	8731
2	8659	17832	17946	18274	9501	8731	17449	17468	17481	8731

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

Frame	7					F				
level/col	B	C	D	E	F	6	5	4	3	2
5		2.44	1.45	2.52		2.63	1.50	2.61		
4	1.22	0.98	1.25	0.92	1.13	2.31	1.45	1.19	0.72	1.41
3	2.40	0.91	1.23	0.85	2.21	1.78	1.20	1.02	1.22	2.34
2	2.02	0.83	1.21	0.76	1.96	1.48	1.11	1.01	1.14	2.05

Flexural stress check -beams (4.4.3.9)

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

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

M_s 9.0 CP

direction	level	h	V_j	n_f	n_c	beam	Z_b	n_{mc}	$Z_b \cdot n_{mc}$	Z	f_j^{avg}
		in	kip			sections	in^3		in^3	in^3	ksi
E-W	high roof	150	1142	2	6	W21X50	110	2	220	880	16.22
						W21X50	110	2	220		
						W21X50	110	2	220		
						W21X50	110	2	220		
	5	150	1970	2	6	W24X55	134	2	268	1072	22.98
						W24X55	134	2	268		
						W24X55	134	2	268		
						W24X55	134	2	268		
	4/ LR	150	2883	3	13	W24X55	134	2	268	2870	10.88
						W24X55	134	2	268		
						W24X55	134	2	268		
						W24X55	134	2	268		
						W21X73	172	2	344		
						W21X73	172	2	344		
						W24X62	153	2	306		
						W24X55	134	2	268		
						W24X55	134	2	268		
						W24X55	134	2	268		
	3	150	3471	3	13	W24X62	153	2	306	3136	11.99
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W21X73	172	2	344		
						W21X73	172	2	344		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
	2	180	3751	3	13	W24X62	153	2	306	3136	15.55
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W21X73	172	2	344		
						W21X73	172	2	344		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		

N-S	roof	150	1142	2	6	W21X50	110	2	220	880	16.2
						W21X50	110	2	220		
						W21X50	110	2	220		
						W21X50	110	2	220		
	5	150	1970	2	6	W24X55	134	2	268	1072	23.0
						W24X55	134	2	268		
						W24X55	134	2	268		
						W24X55	134	2	268		
	4/LR	150	2883	2	10	W24X55	134	2	268	2144	14.01
						W24X55	134	2	268		
						W24X55	134	2	268		
						W24X55	134	2	268		
						W24X55	134	2	268		
						W24X55	134	2	268		
						W24X55	134	2	268		
						W24X55	134	2	268		
	3	150	3471	2	10	W24X62	153	2	306	2448	14.77
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
	2	180	3751	2	10	W24X62	153	2	306	2448	19.15
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		
						W24X62	153	2	306		

Flexural stress check - columns (4.4.3.9)

M_s 9.0 CP

direction	level	h	V_j	n_f	n_c	column	Z_c	Z	f_j^{avg}
		in	kip			sections	in^3	in^3	ksi
E-W	high roof	150	1142	2	6	W14X132	234	1404	10.17
	5	150	1970	2	6	W14X132	234	1404	17.54
	4/LR	150	2883	3	13	W14X132	234	3042	10.27
	3	150	3471	3	13	W14X132	234	3042	12.36
	2	180	3751	3	13	W14X132	234	3042	16.03
N-S	high roof	150	1142	2	6	W14X132	234	1404	10.17
	5	150	1970	2	6	W14X132	234	1404	17.54
	4/LR	150	2883	2	10	W14X132	234	2340	12.84
	3	150	3471	2	10	W14X132	234	2340	15.45
	2	180	3751	2	10	W14X132	234	2340	20.04

Panel zone capacity check

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

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

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

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

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

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

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

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

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beam	*d*	*tf*
	in	in
W21X50	20.8	0.535
W24X55	23.6	0.505
W24x62	23.7	0.59

$0.6 \cdot F_y \cdot d_c \cdot t_w =$ 284 kips

$P_y = F_y \cdot A_g =$ 1940 kips

P_r/P_y

Frame			6					F		
Col	B	C	D	E	F	6	5	4	3	2
5		0.09	0.02	0.09		0.08	0.01	0.08		
4		0.23	0.04	0.25		0.21	0.03	0.22		
3	0.10	0.28	0.05	0.30	0.08	0.29	0.08	0.23	0.06	0.08
2	0.12	0.33	0.06	0.37	0.12	0.39	0.14	0.24	0.12	0.18
1	0.38	0.41	0.08	0.45	0.29	0.51	0.21	0.25	0.17	0.29

$(1.4 \cdot P_r/P_y)$

Frame			6					F		
Col	B	C	D	E	F	6	5	4	3	2
5		1	1	1		1	1	1		
4		1	1	1		1	1	1		
3	1	1	1	1	1	1	1	1	1	1
2	1	1	1	1	1	1	1	1	1	1
1	1.02	0.99	1	0.95	1	0.89	1	1	1	1

R_n kip

Frame			6					F		
Col	B	C	D	E	F	6	5	4	3	2
5		284	284	284		284	284	284		
4		284	284	284		284	284	284		
3	284	284	284	284	284	284	284	284	284	284
2	284	284	284	284	284	284	284	284	284	284
1	289	283	284	270	284	253	284	284	284	284

$\sum M^*_{pb}$ kip·in

Frame			7					F		
level/col	B	C	D	E	F	6	5	4	3	2
5		8030	15710	7680		7644	15276	7632		
4	8659	17832	17946	18274	9501	7644	15276	15293	15305	7644
3	8659	17832	17946	18274	9501	8731	17449	17468	17481	8731
2	8659	17832	17946	18274	9501	8731	17449	17468	17481	8731

$0.8\ V_b$ kip

Frame			7					F		
level/col	B	C	D	E	F	6	5	4	3	2
5		266.50	521.39	254.89		253.68	506.98	253.30		
4	285.18	587.32	591.06	601.85	312.93	253.68	506.98	507.54	507.93	253.68
3	324.68	668.66	672.93	685.21	356.27	327.39	654.27	655.00	655.50	327.39
2	324.68	668.66	672.93	685.21	356.27	327.39	654.27	655.00	655.50	327.39

$0.8V_b/R_n$

Frame	7					F				
level/col	B	C	D	E	F	6	5	4	3	2
5		0.94	1.83	0.90		0.89	1.78	0.89		
4	1.00	2.06	2.08	2.12	1.10	0.89	1.78	1.78	1.79	0.89
3	1.14	2.35	2.37	2.41	1.25	1.15	2.30	2.30	2.30	1.15
2	1.12	2.37	2.37	2.54	1.25	1.29	2.30	2.30	2.30	1.15

Compact member check

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

Check moment frame members using Table D1.1

Section	F_y	R_y	b/t	λ_{md}	Check	h/t_w	λ_{md}	Check
W14X132	50	1.1	7.15	9.2	C	17.7	63	C
W21X50	37	1.1	6.10	10.7	C	49.4	74	C
W24X55	37	1.5	6.94	9.1	C	54.6	63	C
W21X73	37	1.5	5.60	9.1	C	41.2	63	C
W24X62	37	1.5	5.97	9.1	C	50.1	63	C