UCSF

Rating form completed by:

RUTHERFORD + CHEKENE
ruthchek.com
Evaluator: EMG, BL, JM
Date: 10/10/19

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

REGISTERED PROFESSIONAL ENGINEER
BRET LIZUNDIA
No. S3950
Exp. 12-31-20
STRUCTURAL
STATE OF CALIFORNIA
10-10-19

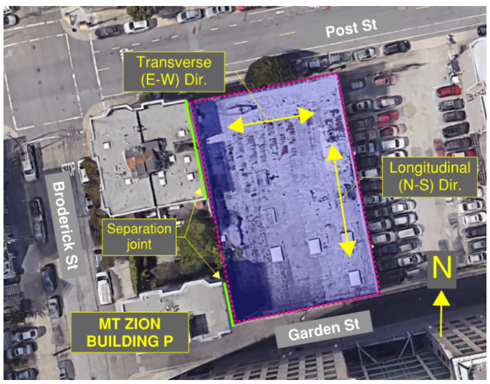
DATE: 2019-10-10

UCSF building seismic ratings

Mount Zion Building P

CAAN #2034
2375 Post Street, San Francisco, CA 94115
UCSF Campus: Mount Zion

Plan



North elevation (looking southeast)

Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	VI	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	Priority A	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application for modification
Ballpark total project cost to retrofit to IV rating	High: $200-$400 per sq. ft.	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?	Yes	

[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 C. W. Zollner Structural Engineer, “Two Story Concrete Garage for Joseph A. Pasqualetti Southline of Post St. 54 ft East of Broderick St.”, dated August 1925, Sheets 1 to 5.

Additional building information known to exist

None

Scope for completing this form

The limited structural drawings, field observation, and field measurements are used as the basis for the completed ASCE 41-17 Tier 1 evaluation. Site visits were made on the September 23 and 25, 2019. The building exterior and portions of the interior were observed.

Brief description of structure

Building P is located near the corner of Post Street and Broderick Street in San Francisco, California. It is a two-story rectangular structure that measures 83’-4” in the east-west direction and 124’-10’ in the north-south direction. It was designed in 1925 from reinforced concrete. The year of construction is unknown. The drawing title block is dated 1925; however, UCSF records show the year of construction to be 1935.

The structure is situated on a sloping site with a high point located slightly below the second floor of the structure on the south elevation and a low point located near the first floor on the north elevation. The total grade change is estimated to be between 10 and 12 ft. Building P contains reinforced shear walls around its perimeter on all four sides. The structure is embedded into the hillside. The south wall in the lower story is below grade and therefore retains soil. The available documentation for Building P is limited.

The original function of the structure is unknown. The construction drawings reference the term “garage,” and a ramp is present inside the structure. It is likely that the structure was designed as a storage facility as opposed to a parking garage. The slabs are likely too thin to support significant vehicle load, and the structure is fully enclosed which would not allow for vehicle exhaust fumes to vent. Building P is currently utilized as a storage facility for the UCSF Mt. Zion campus. On site there is typically one staff member who provides security monitoring at the rear entrance. Additional occupants visit the structure intermittently to either retrieve items or place additional items into storage.

<u>Identification of levels:</u> The building levels are designated as the 1st floor (reference EL. 0’-0”), the 2nd floor (reference EL. 12’-6”), and the roof (reference EL. 28’-4”). These elevations are estimated based upon scaled drawings.

<u>Foundation system</u>: Building P contains isolated spread footings at the interior building columns. The foundations are comprised of truncated pyramid reinforced concrete footings that have a square base ranging in size from 4’-3” x 4’-3” to 5’-6 x 5’-6”. The available details are partially legible and show reinforcing bars at the bottom only. The bar spacing is estimated to be 6” o.c., and the size of the reinforcing is unknown.

The columns located around the building perimeter are partially embedded into the exterior concrete walls. They are supported by isolated rectangular footings. The footings are 7’-6” and 8’-0” long by 14” and 16” wide. The footing length is oriented parallel to the wall, and the outside face of the footing aligns with the exterior face of the wall. The walls are thickened at their base to form what are referred to on the drawings as “footing beams.” At these “beams,” the wall width is increased to 14” and 16” towards the inside of the structure. The bottom of the wall is located approximately 1’-0” below the 1st floor elevation, and the wall thickening extends for a height of 3’-0” and 4’-0” above the bottom of wall. The reinforcing in these locally thickened elements is unknown. It appears these beams are intended to span between the isolated footings located at the building columns.

The construction documents note the "soil bearing pressure as 3 tons". We interpret this to be the soil bearing capacity under dead and live load. This is a reasonably high capacity, and no signs of foundation issues such as settlement or differential movement were observed in the structure.

<u>Structural system for vertical (gravity) load:</u> The gravity load-carrying system is comprised of reinforced concrete columns that support reinforced concrete beams and girders. The columns at the 1st floor are spaced at 15'-3" and 21'-5" in the transverse (east-west) direction and 16'-3", 21'-7", and 22'-3" in the longitudinal (north-south) direction. The columns at the 2nd floor are spaced at 41'-8" in the transverse (east-west) direction and 16'-3", 21'-7", and 22'-3" in the longitudinal (north-south) direction. The column sizes and reinforcing are unknown. Field measurements performed using a hand-held metal detector (Zircon Contractor TriScanner Pro) indicate that the columns likely contain longitudinal reinforcing in their corners tied by transverse reinforcement spaced at an average of approximately 10" o.c.

At the second floor, a 4" thick reinforced concrete slab spans 6'-7" and 7'-6 ½" to concrete beams that are oriented in the north-south direction. The beams range in width from 6" to 16" and in depth from 16" to 30". They are reinforced with square longitudinal bars at the top and bottom that have sides measuring 5/8", ¾", 1", and 1 1/8". The beams contain 3/8" diameter stirrups spaced at 4", 9", and 12"o.c. The more closely spaced ties are located near the beam ends, and the tie spacing increases towards the middle of the beam. The beams are supported by reinforced concrete girders that span in the east-west direction between columns. While the top of the girders are flat, the bottom is slightly arched with an increase in depth at the building columns. The girders are 16" wide and are 3'-9" deep at the face of column and 2'-6" deep at mid-span. The drawing details are partially legible and indicate the girders are designed as T-beams with 8 - 5/8"x 5/8" bars located at the top of the slab and 6 - 1" x 1" bars located at the bottom of the beam. The girders contain ½" diameter stirrups spaced at 6", 8", and 15" o.c.

The roof is wood-framed and contains straight sheathing over 1 ¾" x 9 ½" joists spaced at 30" o.c. The sheathing size is unknown; however, it is estimated to be 1 x 6 sheathing. The wood framing is placed over reinforced concrete girders which are oriented in the east-west direction. Field observation indicates that there is no positive connection between the wood framing and concrete structure.

The concrete girders are tapered and contain a sloped top profile and a flat bottom and span 41'-8". The top surface is likely sloped to drain as the high point is located at the mid-span of the roof. The girders are 14" wide by 40" deep at the column face and 56" deep at mid-span. They are reinforced with two 7/8" x 7/8" bars at the top and five 1" x 1" bars at the bottom. They contain ½" diameter ties spaced at 6", 9", 12", 18", 24", and 44" apart. Concrete beams oriented perpendicular to the girders are located at quarter points along the girder span. These beams measure 12" x 14" and were likely provided for lateral bracing of the girder compression flange.

<u>Structural system for lateral forces:</u> The lateral load-resisting system is comprised of 6" thick reinforced concrete shear walls around the building perimeter. There are two walls in each direction as the structure does not contain interior walls. The walls are relatively solid on the east, west, and south elevations. The south wall is below grade between the 1st and 2nd floor. It was thickened to 12" in order to retain soil. The wall located on the north elevation is heavily penetrated. It is typically 6" thick but contains 12" thick wall piers between multiple window and roll-up door openings. The wall reinforcing size and spacing is not available on the current drawings. However, field measurements using a hand-held metal detector indicate that the vertical and horizontal reinforcing is spaced at an average spacing of approximately 14"o.c. Given the relatively thin wall thickness, it is likely that a single layer of reinforcing is located at the mid-depth of the concrete cross-section. The structure contains a concrete gravity frame with columns that are partially embedded into the walls. The columns support concrete beams that were constructed at the underside of each floor level along the inside face of the walls.

Field observation indicates that some wall penetrations were infilled with CMU block. The extent of the infill is unknown as it is not shown on the available drawings. It was, however, observed for multiple openings on the east, south and north elevations. For the purpose of this assessment, CMU infill is considered to be solid concrete.

The second-floor diaphragm consists of a 4" thick reinforced concrete slab that is dowelled into the perimeter concrete walls. The dowel size and spacing are not available in the current drawings. However, the details indicate that the slab top bars are hooked at the back of the walls and the bottom bars embed as straight bars. Given a wall thickness of 6”, it is unlikely that the bars are fully developed.

The wood-to-concrete connections were observed in the field at an interior girder and at a perimeter wall condition. At the interior concrete girder, the wood joists are oriented perpendicular to the girders and bear directly on the top of the concrete framing. Wood blocking is provided at the between the joists on top of the girders. At the exterior wall condition, a concrete beam is cast on the inside face of the wall. The exterior wall extends above this beam to form the parapet. The wood joists bear on the top of the concrete beam with a slight gap between the end of the joist and the inside face of the wall. No wood blocking is provided along the face of the wall. In both the interior and exterior conditions, the wood diaphragm does not appear to have a positive connection to the concrete structure. No tension or shear wall-to-roof connections were found. As such, the exterior concrete walls are not braced out-of-plane at the roof level, and the inertial load from the roof mass is reliant on friction resistance at the interface of the wood joist to concrete framing for shear transfer and the ability of the walls to cantilever out-of-plane above the second floor.

Finally, Building P is located in close proximity to adjacent wood frame structures on its west elevation. The seismic gap measured in the field at the south and north elevation are 3” wide.

Building condition: Good. The building engineer indicates that the structure had on-going leaks in the roof, however, it underwent repairs in early 2019.

Building response in 1989 Loma Prieta Earthquake: Unknown.

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 wood roof framing is not positively connected to the concrete frame, therefore; the exterior concrete walls do not have either in-plane or out-of-plane anchorage at the roof.
- The walls at the 2nd floor are dowelled into the concrete slab with short embedment.
- The building contains a significant number of openings on the north wall at the lower story. This wall is one of two walls that comprise the lateral system in the transverse direction.
- The north wall is discontinuous.
- The building columns likely contain non-ductile detailing and are shear-controlled.
- The structure is likely torsionally irregular in the lower story due to the prominent difference in rigidity between the perforated 6” thick north wall and the solid 12” thick south wall.
- The building is located in close proximity to an adjacent structure on its west elevation. The provided gap is 3” wide and the required gap to meet the Tier 1 acceptance criteria is 5.8” wide.

Structural deficiency	**Affects rating?**	**Structural deficiency**	**Affects rating?**
Lateral system stress check (wall shear, column shear or flexure, or brace axial as applicable)	Y	Openings at shear walls (concrete or masonry)	N
Load path	Y	Liquefaction	N
Adjacent buildings	Y	Slope failure	N
Weak story	N	Surface fault rupture	N
Soft story	N	Masonry or concrete wall anchorage at flexible diaphragm	Y
Geometry (vertical irregularities)	N	URM wall height-to-thickness ratio	N
Torsion	Y	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		

The wood framing located at the roof is currently bearing on the concrete beams and girders with no attachment between the two materials to transfer in-plane forces. As such, the building relies on friction as a lateral load transfer mechanism. Similarly, the roof diaphragm does not offer out-of-plane anchorage for the walls. The 6" thick walls do not have sufficient capacity to cantilever above the 2nd floor slab. When checked under this load condition, they are severely overstressed.

The exterior walls are thin and flexible in the out-of-plane direction. In the east-west direction, the main building girders tie the east and west exterior walls together. These girders frame into the building columns, and this assembly may function as a back-up moment frame. However, due to the large 42'-2" span of the girders, the strength and stiffness of the moment frame will likely be minimal. In addition, the beams are twice as deep as the columns; therefore, the columns are likely to hinge prior to the beams. Both the beams and columns have non-ductile reinforcing, and their displacement capacity will be limited. In the north-south direction, small 12"x14" secondary beams tie the north and south wall together. These beams are spaced 21'-0" apart and do not align with building columns. Therefore, no back-up moment frame exists in this direction.

In a large seismic event, the 2nd story has the potential to partially collapse. In the north-south direction, if the walls pull away from the minimal restraint offered by the secondary concrete beams, the walls are likely to displace outward such that the roof joists may lose bearing resulting in partial collapse of the roof. Alternatively, if the walls pull away from the secondary beams and plastic hinges have formed above the 2nd floor slab, the wall itself will fall outwards in addition to the roof. In the east-west direction, if plastic hinges form at the top and bottom of the columns or the column fails in shear and in the walls hinge above or fail at the 2nd floor slab, then large portions of the building length may collapse laterally.

Summary of review of nonstructural life-safety concerns, including at exit routes. [2]

A hollow clay tile partition was observed above the entry ramp between the 2nd floor and roof. Although there are exits to the exterior at both the 1st and 2nd floor; the ramp is the only interior connection between the floors.

[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.

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	None observed
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.	None observed

Basis of Seismic Performance Level rating

Building P is a squat rectangular reinforced concrete structure that was designed in 1925. It has a plan aspect ratio of approximately 1W:1.5L and a vertical aspect ratio of approximately 1V:3H in its short direction. It contains reinforced concrete shear walls around its entire perimeter which are continuous to the foundation. In the east-west direction, the average in-plane shear stress under the BSE-2E ground motion is 47 psi between the 1st and 2nd floor and 58 psi the 2nd floor and roof. In the north-south direction, the average in-plane shear stress is 27 psi between the 1st and 2nd floor and 50 psi 2nd floor and roof. These stresses meet the Tier 1 acceptance criterion of 100 psi prescribed by ASCE 41-17. The floor diaphragms are geometrically regular and do not contain split levels, re-entrant corners, or large openings. Despite its regular shape, Building P has seismic deficiencies that include the lack of a positive in-plane and out-of-plane ties between the roof and walls, a torsional irregularity in the transverse direction, a potentially overstressed wall at the north elevation, and inadequate seismic separation between the adjacent structures.

The lack of ties between the roof and wall results in the lack of a typical lateral force-resisting system at the second story. The walls are severely overstressed when evaluated as cantilevers to resist out-of-plane loads as a backup system. Any redundancy offered by the concrete gravity frame is minimal due to their non-ductile detailing, light reinforcing and long spans. As a result, there is the possibility of a second story collapse in a large earthquake, and a Seismic Performance Level rating of Level VI is assigned to Building P.

In addition, there are other, less significant seismic deficiencies. The wall located on the north elevation between the 1st and 2nd floor is 6” thick with 12” thick wall piers located between large window and door openings. The cross-sectional area of this wall is 45% of the wall located on the south elevation. As such, the center of rigidity will shift towards the south wall and the structure likely contains a torsional irregularity in the transverse direction.

The ASCE 41-17 stress check is based upon the total wall area in each direction. It does not account for flexibility of the diaphragm attributed to the slab thickness and span. In the transverse direction, the slab at the 2nd floor is 4” thick and spans 124’-10” between the exterior walls. This diaphragm is likely semi-rigid and the force distribution to the north wall will be higher than predicted using a rigid diaphragm assumption which distributes load based upon relative rigidity of the walls. When checked assuming one-half of the building mass is tributary to the north wall, the stresses in this wall increases from 58 psi to 124 psi in the lower story. This exceeds the ASCE 41-17 limit of 100 psi.

Finally, Building P contains inadequate seismic separation on its west elevation. The measure seismic joint is 3” wide and the gap required by ASCE 41-17 for an interstory drift ratio of 1.5% is 5.8”. It is likely that Building P will drift less than predicted due to its stiff shear wall lateral system. However, the flexibility of the adjacent structure is unknown.

It is also not known if the floor levels of the two structures align. There is the potential for increased damage due to pounding.

The building is assigned a Seismic Performance Level Rating of VI due to the lack of connection between the roof diaphragm and the exterior walls, and the potential for collapse of the second story.

Recommendations for further evaluation or retrofit

It is recommended that this structure be retrofit. The diaphragm should be anchored to the exterior walls around the perimeter of the structure to provide both in-plane and out-of-plane connection to the exterior walls. The diaphragm should also be strengthened with plywood, crossties, and sub-diaphragm detailing. Interior shear walls may be added in the transverse direction and located towards the northern end of the structure. These would help reduce the stresses on the north wall, help to mitigate torsion, and reduce the span of the straight-sheathed diaphragm.

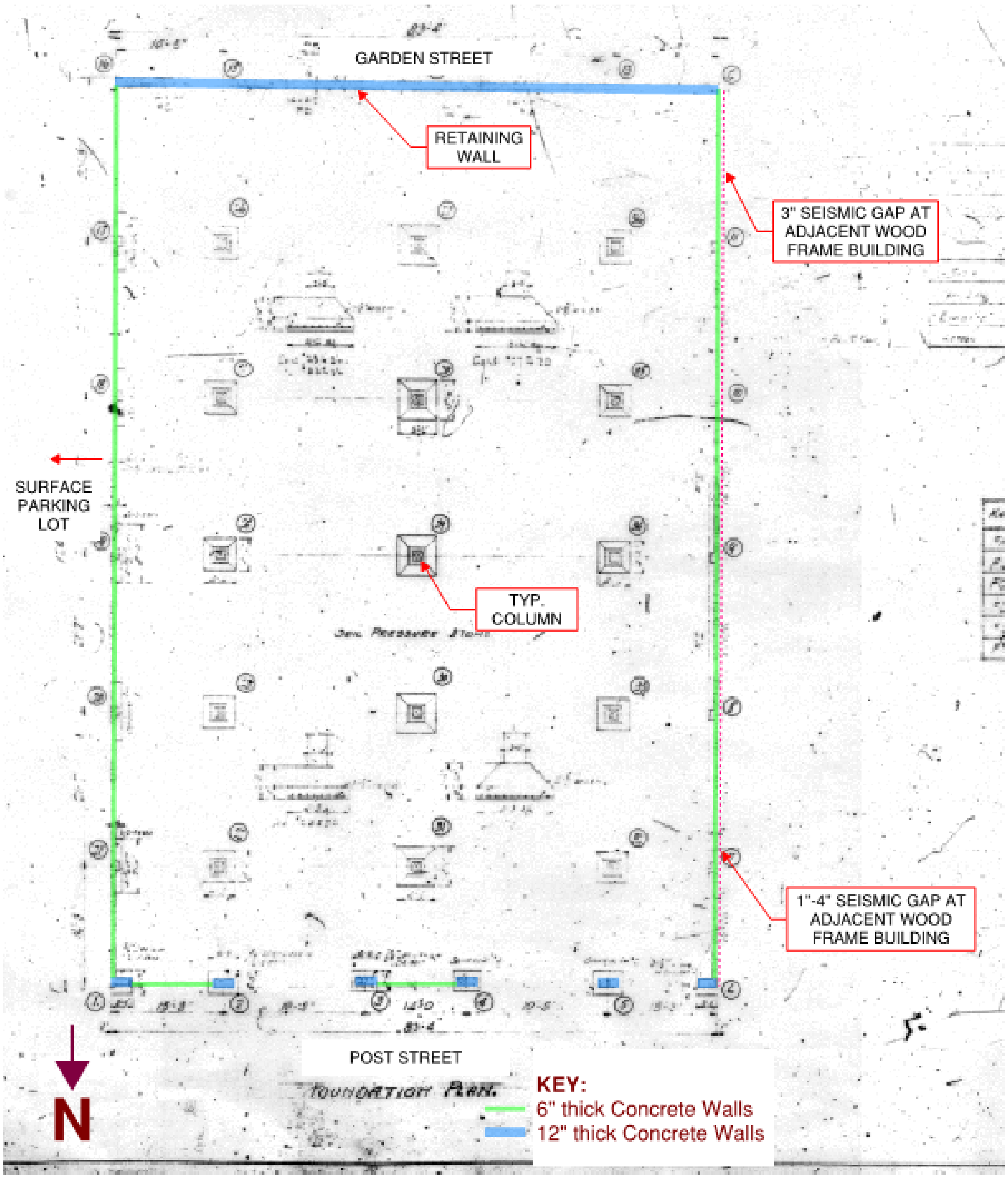
Peer review comments on rating

The structural members of the UCSF Seismic Review Committee (SRC) reviewed the evaluation on 10 October 2019 and were unanimous that the Seismic Performance Level Rating is Level VI. Retrofit is recommended.

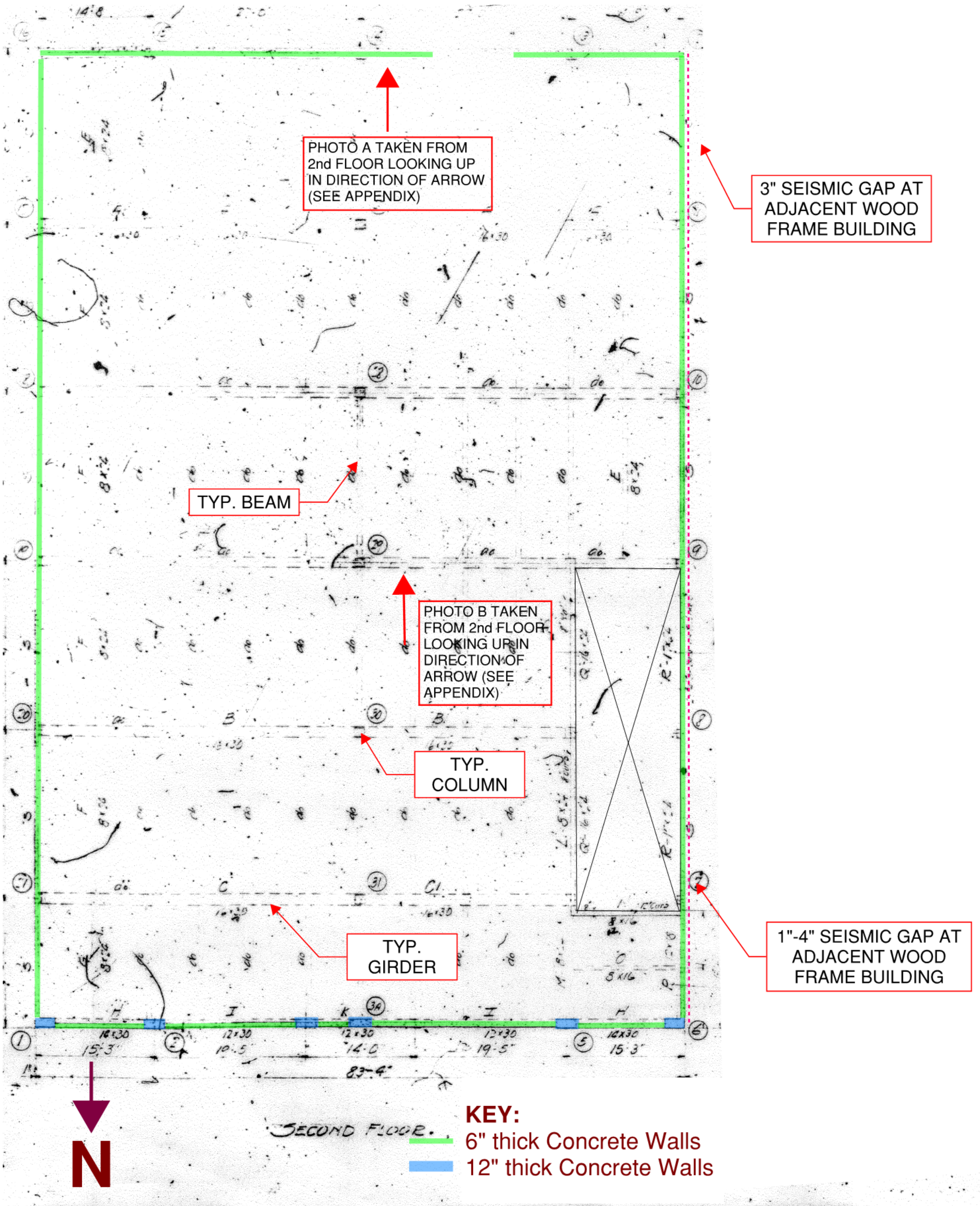
Additional building data	Entry	Notes
Latitude	37.78393	
Longitude	-122.44087	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	2	
Number of stories (basements) below lowest perimeter grade	0	
Building occupiable area (OGSF)	20,800	
Risk Category per 2016 CBC 1604.5	II	
Building structural height, h_n	28.33 ft	Structural height defined per ASCE 7-16 Section 11.2
Coefficient for period, C_t	0.020	Estimated using ASCE 41-17 equation 4-4 and 7-18
Coefficient for period, β	0.75	Estimated using ASCE 41-17 equation 4-4 and 7-18
Estimated fundamental period	0.25 sec	Estimated using ASCE 41-17 equation 4-4 and 7-18
Site data		
975-year hazard parameters S_s, S_1	1.437g, 0.560g	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, 1.740	UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019)

Ground motion parameters S_{cs}, S_{c1}	1.437g, 0.974g	UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019)
S_a at building period	1.44g	W = 2,359 kips, V base = 4,068 kips
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: 1925 Pre-dates UBC	Applicable code assumed
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 north-south	C2 Concrete Shear Walls with stiff Diaphragms (1st to 2nd floor) C2a Concrete Shear Walls with flexible Diaphragms (2nd floor to roof)	
Model building type east-west	C2 Concrete Shear Walls with stiff Diaphragms (1st to 2nd floor) C2a Concrete Shear Walls with flexible Diaphragms (2nd floor to roof)	
FEMA P-154 score	N/A	Not applicable as an ASCE 41 Tier 1 evaluation was performed
Previous ratings		
Most recent rating	IV	The 2013 rating of IV was contingent upon completion of a retrofit scope identified at that time. The retrofit was not installed.
Date of most recent rating	2013	

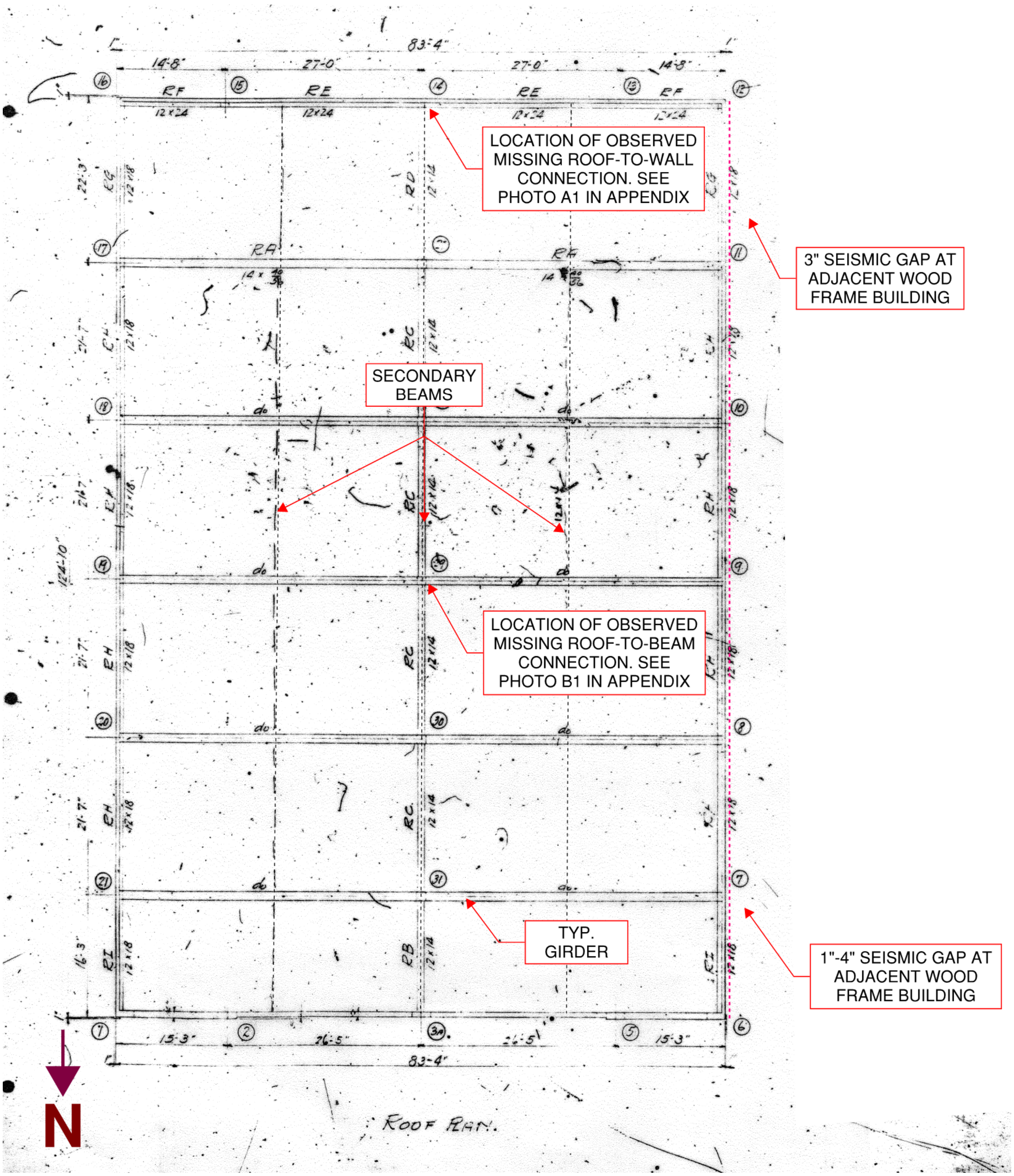
2nd most recent rating	-	
Date of 2nd most recent rating	-	
3rd most recent rating	-	
Date of 3rd most recent rating	-	
Appendices		
ASCE 41 Tier 1 checklist included here?	Yes	Refer to attached checklist file



Lateral force-resisting system at 1st Floor



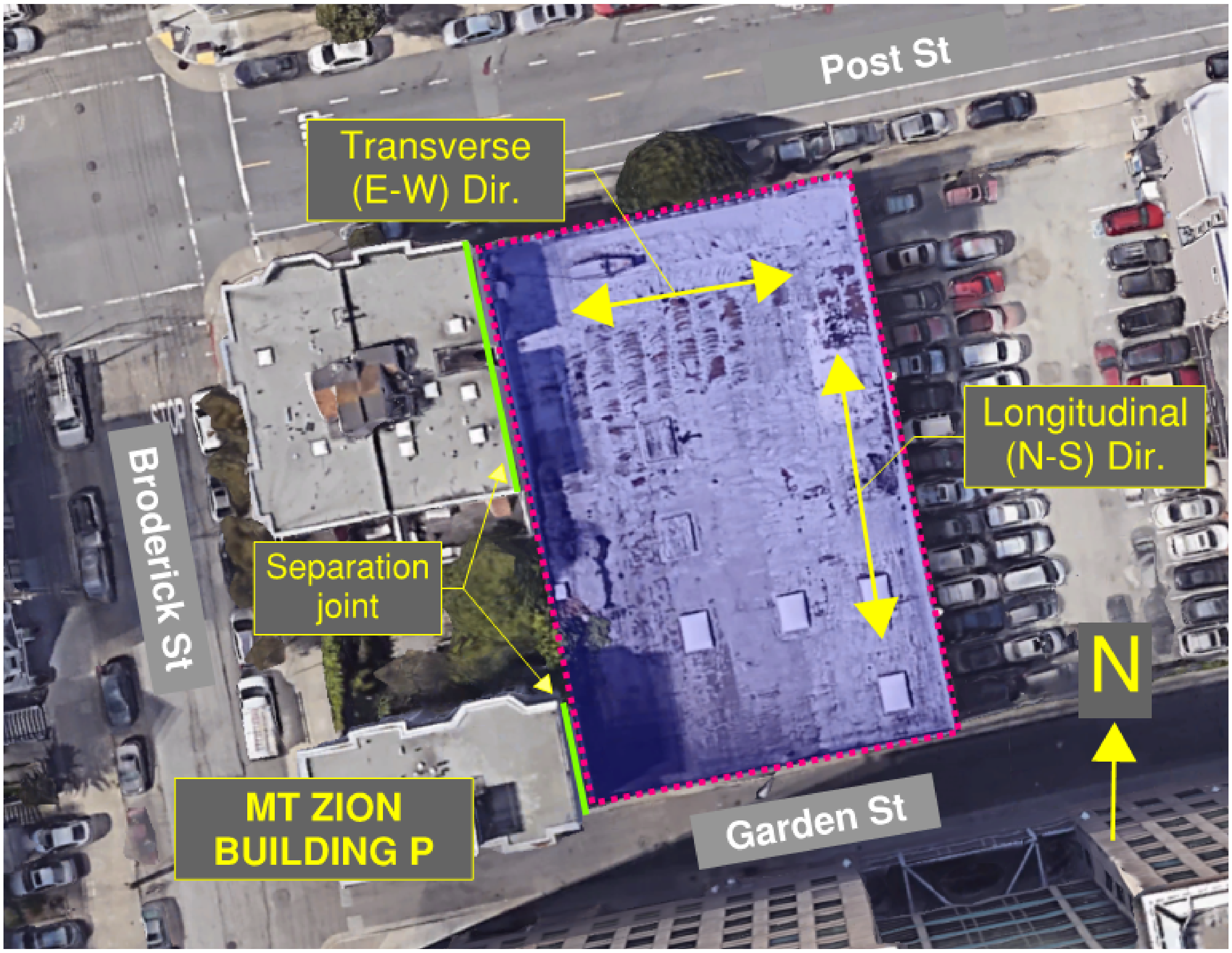
Lateral force-resisting system at 2nd Floor



Lateral force-resisting system at roof

APPENDIX A

Additional Images

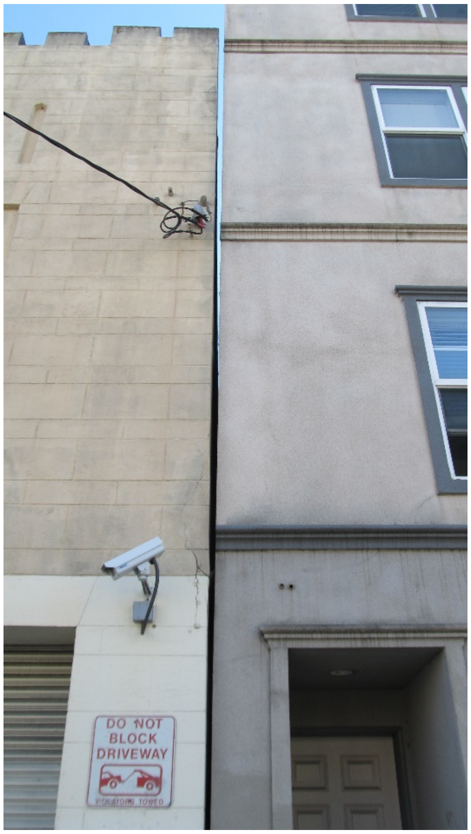

Post St
Transverse (E-W) Dir.
Longitudinal (N-S) Dir.
Broderick St
Separation joint
N
MT ZION BUILDING P
Garden St

Plan

North elevation (looking southeast)

East elevation (looking southwest)

South elevation (looking northeast)



Separation gap on north elevation (looking south)

Adjacent wood frame building to the west (looking south)

Separation gap on south elevation (looking north)

Concrete framing at underside of the second floor (looking south)

Wood roof framing bearing on concrete girders

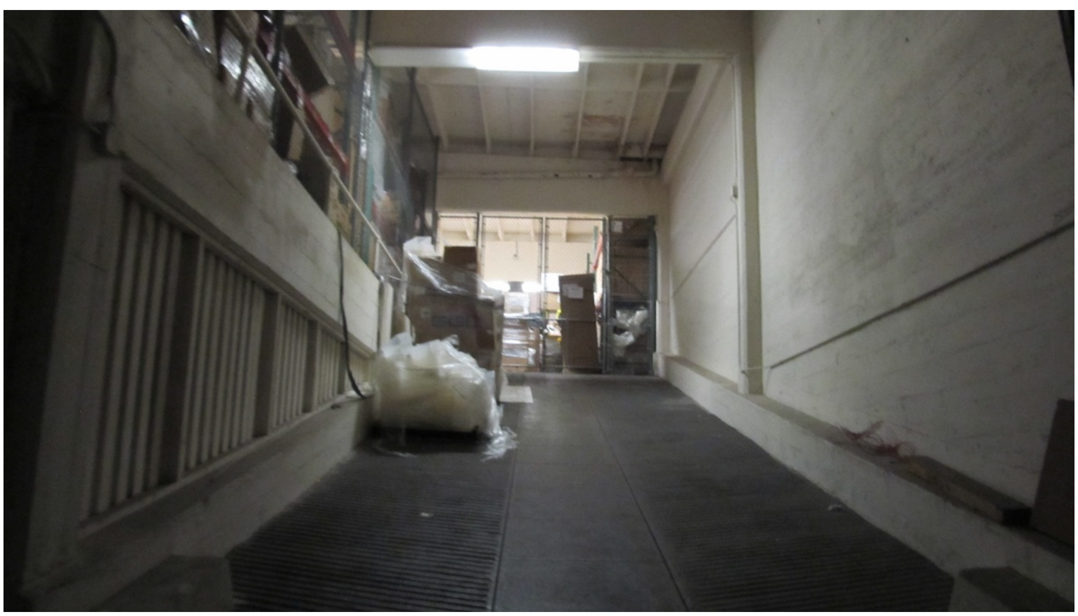
Photo A: Wood joist bearing on south concrete wall ledge with no positive connection (see close-up of circled connection in Photo A1 below)

Photo A1: Wood joist bearing on south concrete wall ledge with no positive connection (inside face of concrete parapet to the left and wood joist to the right)

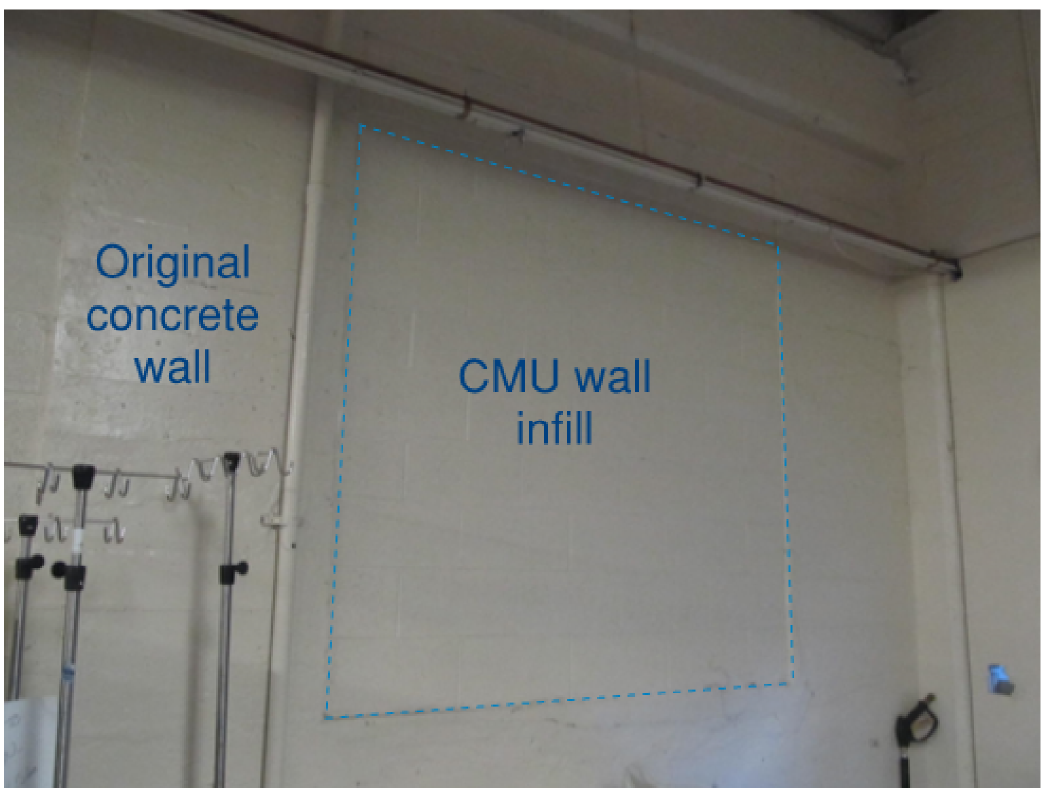
Photo B: Interior wood joist bearing on concrete girder with no positive connection (see close-up of circled similar connection in Photo B1 below)

Photo B1: Interior wood joist bearing on concrete girder with no positive connection (wood blocking to the left and wood joist to the right)

Hollow clay tile partition above ramp (looking north)


Ramp up to the second floor (looking south)

Storage at the second floor (looking southeast)



CMU wall infill on east elevation (looking southeast)

APPENDIX B

ASCE 41-17 Tier 1 Checklists (Structural)

UC Campus:	**San Francisco**			Date:	**10/10/2019**		
Building CAAN:	**2034**	Auxiliary CAAN:		By Firm:	**RUTHERFORD + CHEKENE**		
Building Name:	**UCSF Mt. Zion Building P**			Initials:	**EGM**	Checked:	**BL**
Building Address:	**2375 Post St, San Francisco, CA 94115**			Page:	**1**	of	**3**

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

LOW SEISMICITY

BUILDING SYSTEMS - *GENERAL*

C	NC	N/A	U	Description
○	●	○	○	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:** The wood framed roof does not contain a positive connection to the concrete roof framing. The wood framing relies on bearing and friction under its self-weight.
○	●	○	○	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:** Building P is 32’-2” tall from the first floor to the top of the parapet on the north side of the structure. The required gap is 5.8”. The gap measured in the field is approximately 1” wide at the base and 4” wide at the top. The gap at the rear, south elevation is approximately 3” wide. Given the stiffness of a concrete shear wall building, it is possible that a 6” gap would not be required. It appears that the adjacent building is a wood-framed structure; however, this is not confirmed. It is also unknown if the floor elevations of the two structures align.
○	○	●	○	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 mezzanine levels in the structure.

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

C	NC	N/A	U	Description
●	○	○	○	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:** In the east-west direction, the total wall area increases from the roof down to the 1st floor. In the north-south direction, the total wall area remains the same between stories.
●	○	○	○	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:** In the east-west direction, the total wall area increases from the roof down to the 1st floor. In the north-south direction, the total wall area remains the same between stories.

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

UC Campus:	San Francisco		Date:	10/10/2019		
Building CAAN:	2034	Auxiliary CAAN:	By Firm:	RUTHERFORD + CHEKENE		
Building Name:	UCSF Mt. Zion Building P		Initials:	EGM	Checked:	BL
Building Address:	2375 Post St, San Francisco, CA 94115		Page:	2	of	3

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

C NC N/A U	
NC	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:** The wall on the north elevation is discontinuous.
C	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:** The structure is rectangular, and the walls are continuous from the roof to the first floor.
N/A	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 building is a two-story structure with a light roof.
NC	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 structure likely has a torsional irregularity in the east-west direction between the 1st and 2nd floor. The wall located on the south elevation at this story is 12” thick with no openings. The wall located on the north elevation is 6” thick and contains significant openings. As a result of this configuration, the center of rigidity will shift to the south.

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

GEOLOGIC SITE HAZARD

C NC N/A U	Description
C	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	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 is located on a gentle slope (approximately 3-degrees) and it not susceptible to landslides.

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

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

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

GEOLOGIC SITE HAZARD

C NC N/A U	
C (selected)	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.

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

FOUNDATION CONFIGURATION

	Description
C NC N/A U C (selected)	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 = 83’-4” in the east-west direction. The building height from the first floor to the roof is H = 28”-4”, B/H = 2.94 Sa = 1.44g for at BSE-2E 0.6x Sa = 0.864 B/H > 0.6 Sa.
C NC N/A U C (selected)	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:** Per “Table 1 - UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards” by Egan (2019), the soil is classified as Site Class C.

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

UC Campus:	San Francisco			Date:	10/10/2019		
Building CAAN:	2034	Auxiliary CAAN:		By Firm:	RUTHERFORD + CHEKENE		
Building Name:	UCSF Mt. Zion Building P			Initials:	EGM	Checked:	BL
Building Address:	2375 Post St, San Francisco, CA 94115			Page:	1	of	4

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:** Shear walls contain embedded columns which support beams located on the inside face of the walls.
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:** Concrete shear walls are located around the building perimeter. There are two walls in each direction.
C NC N/A U (NC 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:** When considering the total wall area in each story, the calculated average wall stress in the concrete walls is 47 psi in the upper story and 58 psi in the lower story. These stresses are within the ASCE 41 limit of 100 psi for f'c = 2,000 psi (assumed compressive strength per Table 4-2 in ASCE 41-17). At the first story, if the walls are checked assuming the lateral load equally splits between the north and south wall, the shear stresses are 55 psi (south elevation), and 124 psi (north elevation). The wall stress in the north walls exceeds the limit of 100 psi.
C NC N/A U (U 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:** The reinforcing is not documented on the available structural drawings. However, field investigation with a hand-held metal detector (stud finder) indicates that reinforcing is likely located at an average spacing of approximately 14” o.c. in each direction within the 6” thick walls. The bar size is unknown. If a 3/8” x 3/8” square bar was used, then ρ = 0.00167. If a 1/2” x 1/2” square bar was used, then ρ = 0.0029.

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

UC Campus:	**San Francisco**			Date:	**10/10/2019**		
Building CAAN:	**2034**	Auxiliary CAAN:		By Firm:	**RUTHERFORD + CHEKENE**		
Building Name:	**UCSF Mt. Zion Building P**			Initials:	**EGM**	Checked:	**BL**
Building Address:	**2375 Post St, San Francisco, CA 94115**			Page:	**2**	of	**4**

ASCE 41-17
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:** At the roof, the exterior concrete walls do not have positive anchorage to the wood framing.
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 exterior concrete walls do not have positive anchorage to the wood framing at the roof. At the second floor, the slab reinforcing is embedded into the exterior 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 building columns located at the interior and along the exterior walls are supported by isolated spread footings. The available drawings indicate the walls are locally thickened at its bottom as this serves as a foundation beam possibly intended to span between the isolated footings. The reinforcing is unknown.

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 column reinforcing is unknown. Given the building vintage and the reinforcing detailing shown details that are available for review, it is likely that the column reinforcing is non-ductile.
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:** This structure does not contain flat slabs.
C ○ **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:** This structure does not contain coupling beams.

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

UC Campus:	**San Francisco**			Date:	**10/10/2019**		
Building CAAN:	**2034**	Auxiliary CAAN:		By Firm:	**RUTHERFORD + CHEKENE**		
Building Name:	**UCSF Mt. Zion Building P**			Initials:	**EGM**	Checked:	**BL**
Building Address:	**2375 Post St, San Francisco, CA 94115**			Page:	**3**	of	**4**

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

Diaphragms (Stiff Or Flexible)

C NC N/A U	Description
C ● **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:** This structure does not contain split levels.
C ○ **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:** The floor opening at the second floor for the ramp is 44'-0" long and the adjacent wall is 124'-10" long. The opening is approximately 35% of the wall 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:** Although secondary concrete framing is present at the roof, these do not have the capability to develop the out-of-plane anchorage into the diaphragm.
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 diaphragm aspect ratio is 1W:1.5L.
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:** Straight sheathing is used and the span between exterior walls is 83'-4". No connection of the diaphragm to the interior or exterior concrete framing is present.
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:** Straight sheathing is used.
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 does not contain "other diaphragms."

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

UC Campus:	San Francisco			Date:	10/10/2019		
Building CAAN:	2034	Auxiliary CAAN:		By Firm:	RUTHERFORD + CHEKENE		
Building Name:	UCSF Mt. Zion Building P			Initials:	EGM	Checked:	BL
Building Address:	2375 Post St, San Francisco, CA 94115			Page:	4	of	4

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

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:** Building has isolated spread footings.

APPENDIX C

UCOP Seismic Safety Policy Falling Hazards Assessment Summary

UC Campus:	**San Francisco**			Date:	**10/10/2019**		
Building CAAN:	**2034**	Auxiliary CAAN:		By Firm:	**Rutherford+Chekene**		
Building Name:	**UCSF Mt. Zion Building P**			Initials:	***EGM***	Checked:	***BL***
Building Address:	**2375 Post Street, 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:** No masonry or stone veneer is located near exit ways or public access areas.
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:** No hazardous material storage was observed inside the building.
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:** Two gas heaters hang from the roof.
P ☐ **N/A** ☐	**Other:** **Comments:**
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

	Seismic Weight	Dead Load	
ROOF	**psf**	**psf**	**Remarks**
Roofing	3.8	3.8	Built-up roofing system, 3-ply and smooth-surface assumed
Sheathing	2.9	2.9	1x straight sheathing assumed
Wood framing	1.5	1.5	Wood joists below straight sheathing
Beams/girders	33	33	Concrete beams below wood framing
MEP	3	3	MEP hung from underside of roof
Lighting and misc.	2	2	Lighting, and misc. hung from underside of roof
Columns	3	0	Reinforced concrete columns
Partitions	0	0	
Total	50	46	

1 - The roof was not accessed during the site visit. Weight is estimated based on description provided by building managers.

2 - The wood framing was measured on the field as 2x10 nominal joists at 30" o.c.

3 - The column schedule in the original structural drawings is illegible. Some columns were measured in the field, obtaining the following dimensions: 15.5"x16", 15"x14", 13.5"x15.5", and 18.75"x. A typical 15"x15" column is used for calculations.

4 - The flat load includes weight of (20) 15" square concrete columns below roof in a 10,403 ft^2 area. Column trib. height is 7'-11".

	Seismic Weight	Dead Load	
2ND FLOOR	**psf**	**psf**	**Remarks**
Flooring	0	0	
Slab	50	50	4" NWC slab
Beams/girders	42	42	Concrete beams below slab
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	8	0	Reinforced concrete columns
Partitions	0	0	
Total	105	97	

1 - The concrete slab is scaled from the original drawings and is 4" thick.

2 - The column schedule in the original structural drawings is illegible. Some columns were measured in the field, obtaining the following dimensions: 15.5"x16", 15"x14", 13.5"x15.5", and 18.75"x. A typical 15"x15" column is set for calculations.

3 - The flat load includes weight of (30) 15" square columns below and (20) 15" square concrete columns above floor in a 9,744 ft^2 area. Column trib. height is 14'-2".

	Seismic Weight	Dead Load	
RAMP	**psf**	**psf**	**Remarks**
Flooring	0	0	
Slab	100	100	8" NWC slab
Beams/girders	28	28	Concrete beams below slab
MEP	0	0	
Lighting and misc.	0	0	
Columns	0	0	
Partitions	0	0	
Total	128	128	

1 - This flat load is a for a reinforced concrete slab assembly that takes place on the northwest corner of the structure.

2 - The concrete slab is typically 8" thick for the ramp, as shown in Section A-A on sheet 4 in structural drawings.

3 - The reinforced concrete column weight is included in the Second floor flat load table.

Story Weight

wconcrete = 150 pcf

Floor Levels	Floor Area (ft²)[1,2] ROOF	2ND FLOOR	RAMP	Floor Weight (psf) ROOF	2ND FLOOR	RAMP	Height: Height below floor level (ft)	Wall Weight[3,4]: Wall height tributary to each floor level (ft)	Wall Area below (ft²)	Wall Weight below (kips)	Wall Seismic Weight (kips)	Additional Weight (kips)[4]	Total Seismic Weight (kips)
Roof	10,403	0	0	50	105	128	15.83	7.92	211	500	250	47	814
2nd Floor	0	9,744	343	50	105	128	12.50	14.17	242	455	477		1,546
1st Floor													

2,359 **kips**

Notes:
1 - The seismic base is set at the 1st floor. The soil-structure interaction is ignored for the Tier 1 check
2 - Half the area of the ramp is considered for the seismic weight at the Second Floor
3 - The wall weight includes area of exterior and interior concrete walls
4 - Additional weight includes parapet around the perimeter of the building. The parapet height is slightly taller on the northwest and northeast corners, for simplicity, the parapet is is assumed to be 1'-6" tall, and 6" thicl
5 - A sample calculation for the wall seismic weight at 2nd floor is provided below

Wall ID	Thickness (in)	Length (ft)	Concrete/Total area *	Area (ft²)
L1 - 1X	12	83.3	1.00	83.3
L1 - 1XC	12	60.2	0.38	22.9
L1 - 2XC	6	11.6	1.00	5.8
L1 - 1Y	6	124.8	1.00	62.4
L1 - 2Y	6	124.8	1.00	62.4
L1 - 3XC	6	11.5	0.97	5.6
			Σ =	242.5

Wall ID	Thickness (in)	Length (ft)	Concrete/Total area *	Area (ft²)
L2 - 1X	12	2.5	1.00	2.5
L2 - 2X	12	2.5	1.00	2.5
L2 - 3X	12	2.5	1.00	2.5
L2 - 4X	12	2.5	1.00	2.5
L2 - 5X	12	2.5	1.00	2.5
L2 - 6X	12	2.5	1.00	2.5
L2 - 1XC	6	68.3	0.97	33.2
L2 - 2XC	6	83.3	0.90	37.5
L2 - 1Y	6	124.8	1.00	62.4
L2 - 2Y	6	124.8	1.00	62.4
			Σ =	210.5

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

Wall height above = 15.83 ft
Wall height below = 12.50 ft

Wall area above = 210.5 ft^2
Wall area below = 242.5 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 = 477 kips

Period

C_t=	0.02
h_n (ft)=	28.33
B=	0.75

T=	0.25	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 "all other framing system" per ASCE 41-17 Section 4.4.2.4.

3- The building height is taken from the 1st floor to the roof.

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);

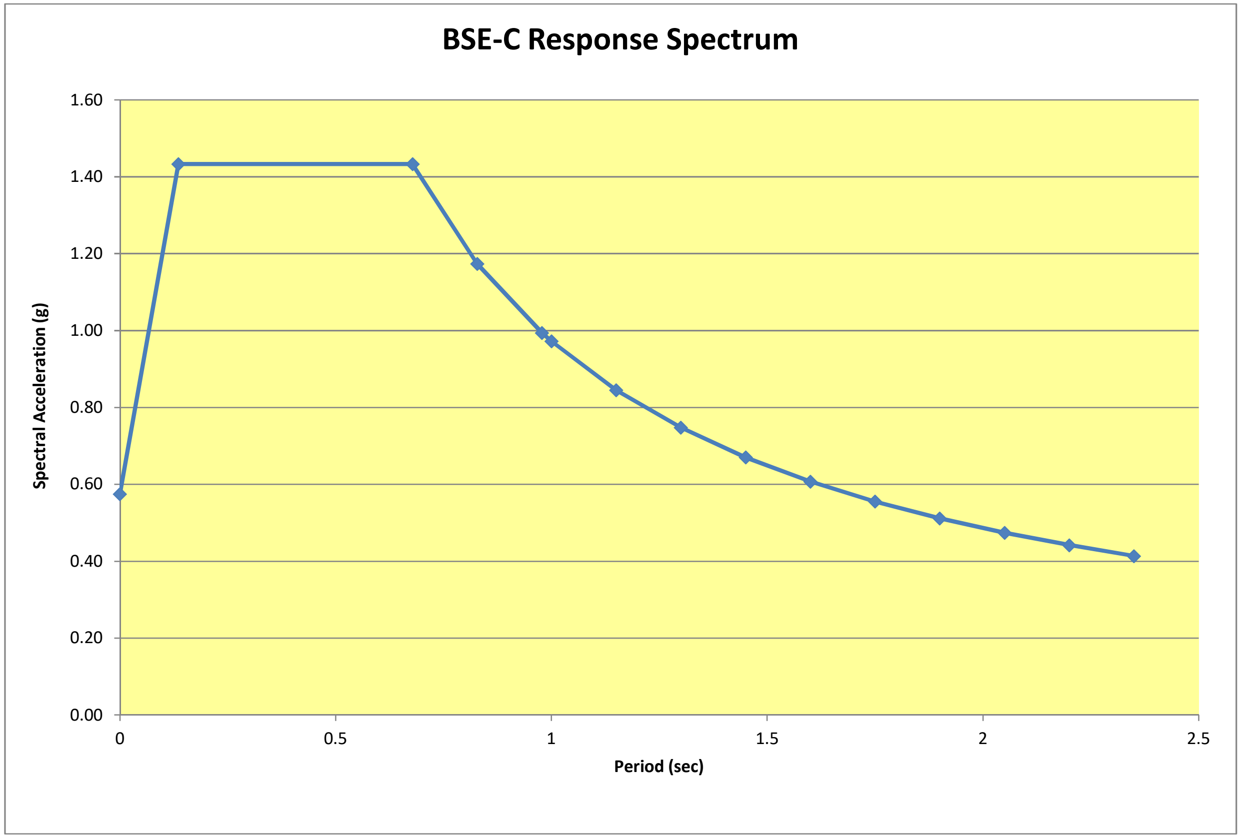
= 0.90 for moment-resisting frame systems of reinforced concrete (Building Type C1); and

= 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	0.85
1.30	0.75
1.45	0.67
1.60	0.61
1.75	0.56
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 =	1.437	g
S_1 =	0.560	g
F_a =	1.000	g
F_v =	1.740	g
Site Class =	D	
S_{CS} =	1.437	g
S_{C1} =	0.974	g
T_0 =	0.14	s
T_s =	0.68	s
T =	0.25	s
S_a =	1.43	g (See Note 2)
Tier 1 S_a =	**1.44**	**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}=	1.437 g	(See Note 2)
S_{C1}=	0.9744 g	(See Note 2)

T=	0.25	s	
Sa=	1.44	g	(See Note 3)
W=	2,359	kips	
C=	1.2	Per ASCE 41-17 Table 4-7	

V=	4,068	kips

k= 1.00

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.

Floor Levels	Story Height	Total Height, H	Weight, W	W x H^k	coeff	Fx	Story Shear, V
	(ft)	(ft)	(kips)			(kips)	(kips)
Roof	15.83	28.33	814	23,050	0.54	2,213	2,213
2nd Floor	12.50	12.50	1,546	19,322	0.46	1,855	4,068
1st Floor							
Σ =	28.3		2,359	42,371	1	4,068	

Notes:

1- The seismic base of building is set at the 1st floor.

2- 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".

3- 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}.

4- 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 = 2000 psi (See Note 3)

Longitudinal (N-S direction)					
Global Distribution					
Story	**Story Shear**	**Wall Area**	**Average Shear Stress Demand**	**Tier 1 Shear Stress Limit**	**Wall OK?**
	(kips)	(in^2)	(psi)	(psi)	
Roof - 2nd Floor	2,213	17,976	27	100	OK
2nd Floor - 1st Floor	4,068	17,976	50	100	OK

Transverse (E-W direction)					
Global Distribution					
Story	**Story Shear**	**Wall Area**	**Average Shear Stress Demand**	**Tier 1 Shear Stress Limit**	**Wall OK?**
	(kips)	(in^2)	(psi)	(psi)	
Roof - 2nd Floor	2,213	10,362	47	100	OK
2nd Floor - 1st Floor	4,068	15,641	58	100	OK

Shear Check for walls between the 1st to 2nd floor assuming the lateral shear is split equally					
Local Distribution between Second Floor to First Floor					
Wall Location	**Story Shear**	**Wall Area**	**Average Shear Stress Demand**	**Tier 1 Shear Stress Limit**	**Wall OK?**
	(kips)	(in^2)	(psi)	(psi)	
South Elevation (Garden St)	2,034	8,202	55	100	OK
North Elevation (Post St)	2,034	3,641	124	100	NG

Notes:

1 - The shear stress check is performed using the ASCE 41-17 Tier 1 screening criteria and the BSE-C site modified spectral response parameters.

2 - 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.

3 -Table 4-2 in ASCE 41-17 is used as a reference to determine f'c = 2 ksi as default concrete compressive strength for Tier 1 Quick Checks.

Table 4-2. Default Compressive Strengths (f'_c) of Structural Concrete (kip/in.^{2})

Time Frame	Beams	Slabs and Columns	Walls
1900–1919	2	1.5	1
1920–1949	2	2	2
1950–1969	3	3	2.5
1970–Present	3	3	3

4 - The local distribution table in the transverse (E-W) direction of analysis assumes that the story shear is distributed based on tributary areas between the two exterior walls. This distribution is appropriate when assuming the 4" thick concrete slab behaves as a flexible diaphragm.

Cantilever Extension Wall Check Under BSE-2E Response Spectra

Per Section 7.2.11.2 in ASCE 41-17:

7.2.11.2 Out-of-Plane Strength of Walls. Wall components shall have adequate strength to span between locations of out-of-plane support when subjected to out-of-plane forces calculated using Eq. (7-13), but not less than forces calculated using Eq. (7-14):

$$F_p = 0.4 S_{XS} \chi W_p \quad (7\text{-}13)$$

$$F_{p,\min} = 0.1 \chi W_p \quad (7\text{-}14)$$

1 - Equation 7-13 presumes there is a restraint at the top and bottom of the wall and thus no dynamic amplification.
However, at Building P there is no restraint at the roof level so dynamic amplification is likely. Thus, these force demands represent a low bound.
2- Per Section 7.2.11 in ASCE 41-17, "Actions that result from application of the forces specified in this section shall be considered **force-contolled**."

Per Section 7.2.11.2 in ASCE 41-17:

7.5.2.2.2 Acceptance Criteria for Force-Controlled Actions for LSP or LDP. Force-controlled actions in primary and secondary components shall satisfy Eq. (7-37):

$$\kappa Q_{CL} > Q_{UF} \quad (7\text{-}37)$$

where

Q_{CL} = Lower-bound strength of a force-controlled action of an element at the deformation level under consideration. Q_{CL}, the lower-bound strength, shall be determined considering all coexisting actions on the component under the loading condition by procedures specified in Chapters 8 through 12, 14, and 15.

κ = Knowledge factor defined in Section 6.2.4.

Q_{UF} = Force-controlled action caused by gravity loads in combination with earthquake forces;

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J} \quad (7\text{-}35)$$

Design parameters

Equations 7-13 and 7-14

S_{XS} =	1.437 g	
χ =	0.8	(Collapse Prevention Performance Level, Table 7-2 in ASCE 41-17)

Equations 7-35 and 7-37

κ =	0.75	(Table 6-1 / ASCE 41-17, for default material properties)
χ =	1.0	(Collapse Prevention Performance Level)
C_1C_2 =	1.0	
J =	2	(Force-delivery reduction factor, high seismicity level)

Out-of-Plane Force per Unit Area, F_p

Cantilever Wall Weight from 2nd floor to roof

Wall height =	15.83	ft	
Wall thickness =	6	in	
$W_{p,wall}$ =	1.187	klf	(1 ft strip x 15.83 ft tall x 6" thick x 150pcf)

Roof weight

Flat load =	46.3	psf	
Trb. Area =	62.417	ft^2	(1 ft strip x 62.42 ft in the N-S direction)
$W_{p,roof}$ =	2.89	klf	

F_p =	0.460	W_p	(Eq. 7-13 / ASCE 41-17)
$F_{p,\,min}$ =	0.080	W_p	(Eq. 7-14 / ASCE 41-17)
F_p =	0.460	W_p	(Maximum of Eq. 7-13 and 7-14)
F_p =	1.876	klf	(W_p = $W_{p,wall} + W_{p,roof}$)

Material properties

f'_c =	2,000	psi	(See Note 3)
f_y =	40,000	psi	(See Note 3)
b =	12	in	(Unit width)
d =	3	in	(One layer of reinforcement at center assumed
β_1 =	0.85	in	
ρ =	0.0048		(#4 @14"o.c. assumed, See Note 5)
A_s =	0.171	in^2/ft	
c =	0.395	in	

Moment Demand

M_E =	25.38	kips-ft/ft	($M_E = F_{p\,wall}$ x Wall Height /2 + $F_{p\,roof}$ x Wall Height)
M_{UF} =	12.69	kips-ft/ft	(Eq. 7-35 / ASCE 41-17)

Moment Capacity

M_{CL} =	19,416	lb-in/ft	(See Note 1)
M_{CL} =	1.62	kips-ft/ft	
κM_{CL} =	1.2	kips-ft/ft	
M_{UF} / (κM_{CL}) =	10.46		
Acceptance criteria	NG		

Shear Demand

V_E =	1.88	klf	($V_E = F_p$)
V_{UF} =	0.94	klf	(Eq. 7-35 / ASCE 41-17)

Shear Capacity

V_{CL} =	3,220	plf	(See Note 2)
V_{CL} =	3.2	klf	
κV_{CL} =	2.4	klf	
V_{UF} / (κV_{CL}) =	0.39		
Acceptance criteria	OK		

Notes:

1 - The lower-bound moment capacity of the wall is obtained using the following formula:

$$M_{CL} = A_s f_y d \left(1 - 0.59 \frac{A_s f_y}{f'_c bd}\right)$$

2 - The lower-bound shear force capacity of the wall considers only the concrete contribution, as the vertical reinforcing steel is unknown.

$$V_{CL} = 2\sqrt{f'_c}bd$$

3 - Tables 4-2 and 4-3 in ASCE 41-17 are used as a reference to determine f'c = 2 ksi and fy = 40 ksi as default material strengths for Tier 1 Quick Checks.

Table 4-2. Default Compressive Strengths (f'_c) of Structural Concrete (kip/in.^{2})

Time Frame	Beams	Slabs and Columns	Walls
1900–1919	2	1.5	1
1920–1949	2	2	2
1950–1969	3	3	2.5
1970–Present	3	3	3

Table 4-3. Default Yield Strengths (f_y) of Reinforcing Steel (kip/in.^{2})

		Structural[a]	Intermediate[a]	Hard[a]				
	Grade	33	40	50	60	65	70	75
Year	Minimum Yield[a] (kip/in.^{2})	33	40	50	60	65	70	75
1911–1959		X	X	X		X		
1959–1966		X	X	X	X	X	X	X
1966–1987			X	X	X	X	X	
1987–present			X	X	X	X	X	X

Note: An entry of X indicates that the grade was available in those years.
[a] The terms structural, intermediate, and hard became obsolete in 1968.

4 - The force-delivery reduction factor, J, equals to 2.0 considering the ductility of the reinforcement steel in the cantilever wall when resisting the out-of-plane demand.

5 - Available drawings did not provide reinforcing information. Reinforcing spacing was estimated in the field with an electronic metal locator.