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.*

DATE: 2019-10-10

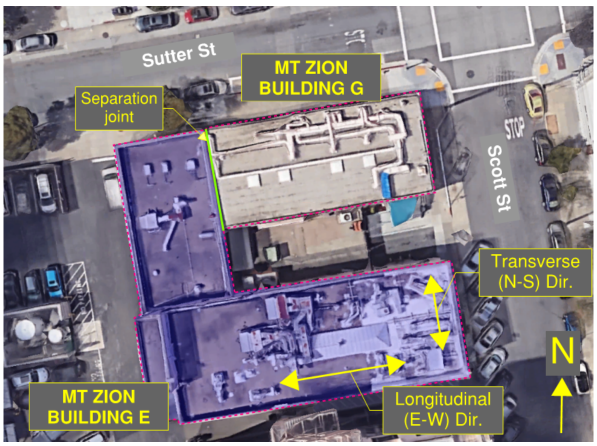
**UCSF building seismic ratings**

## Mount Zion, Building E (Harold Brunn Research Institute)

CAAN #2024
1657 Scott Street, San Francisco, CA 94115
UCSF Campus: Mount Zion

10-10-19

Plan




East elevation (looking west)


| Rating summary | Entry | Notes |
|---|---|---|
| UC Seismic Performance Level (rating) | V | 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 B | Priority A=Retrofit ASAP<br>Priority B=Retrofit at next permit application for modification |
| Ballpark total project cost to retrofit to IV rating | High ($200-400/sf) | 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**

- Architectural drawings by Schubart and Friedman Architects, "Alterations to Existing Building E, New Medical Science Research Building, San Francisco, California," dated 5 September 1962, Sheets A-1, A-4 to A-9, A-14, A-15.
- Structural drawings by I. Thompson Structural Engineer, "Alterations to Existing Building E, New Medical Science Research Building, San Francisco, California," dated 5 September 1962, Sheets S1 to S8.
- Architectural drawings by Fong & Chan Architects, "Harold Brunn (E-1) Building Lab Alterations, UCSF at Mount Zion," dated 25 September 1991, Sheets A-0.1, A-1 to A-6.
- Structural drawings by Nishkian and Associates Consulting and Structural Engineers, "Harold Brunn (E-1) Building Lab Alterations, UCSF at Mount Zion," dated 25 September 1991, Sheets S-1 to S-2.

**Additional building information known to exist**

None

**Scope for completing this form**

The architectural and structural drawings from the 1962 and 1991 renovations were used as the basis for the completed ASCE 41-17 Tier 1 evaluation. A site visit was made on 20 September 2019 where the building exterior and portions of the interior were observed.

**Brief description of structure**

Building E is located on a flat site near the corner of Scott Street and Sutter Street in San Francisco, California. It is adjacent to Building G and is separated from that structure with a 3" wide gap. Originally built in 1930, it is an L-shaped structure with two distinct portions. The north wing is a one-story section that measures 67'-0" in the north-south direction by 35'-0" in the east-west direction. It contains a wood-framed roof that is supported by unreinforced masonry walls located around its perimeter. The south wing is a two-story section that measures 46'0" in the north-south direction by 124'10"in the east-west direction. It contains concrete slabs at the floor levels and a concrete frame infilled with unreinforced masonry around its perimeter. The original drawings are not currently available for review.

In 1962, Building E was renovated. At that time, gunite was added to the face of all of the masonry walls and the wood framed roof was strengthened. In addition, a new reinforced concrete stair tower that measures 46'-0" in the north-south direction by 9'-8" in the east-west direction was constructed. During this renovation, the north wing was converted into an animal facility that housed dogs and "small animals," while the south wing functioned as a research unit.

In 1991, the south wing of the building was renovated and utilized as a research laboratory. It housed functions such as pathology, flouroscopy, and immunology. At that time, the structural modifications were limited to minor roof openings and the addition of a small air handler on the roof.

Currently, the structure is vacant and is utilized for storage. It is intermittently occupied by the building engineers who perform routine maintenance and by individuals adding or removing stored items.

<u>Identification of levels:</u> The building levels are designated as the first floor (reference EL. 0'-0"), the second floor (reference elevation EL. 13'-1"), the low roof (estimated reference EL. 15'-6"), and the high roof (reference EL. 25'-7"). The exterior grade is approximately flat.

<u>Foundation system:</u> The original foundations are only shown in sections and details in the 1962 renovation drawings. They appear to be strip footings located below the exterior load bearing masonry walls. The reinforced concrete stair tower that was added in 1962 contains reinforced concrete strip footings below the concrete walls. They are 9 ½" thick by 1'-10" wide and are reinforced with 2-#5 bars in the direction parallel to the footing. The wall vertical reinforcing hooks into the footings with extended hooks that serve as the footing transverse reinforcement. The dowels match the size and spacing of the wall vertical reinforcing and are #4 bars on each face spaced at 16" o.c.

<u>Structural system for vertical (gravity) load:</u> Building E is comprised of two distinct portions. The north wing is a one-story structure constructed with exterior load bearing masonry walls that supports a wood framed roof. The low roof consists of wood trusses that span 35'-0" in the east-west direction. They are spaced at 3'-0" o.c. and utilize 2 x 6 chord members and 1 x 6 diagonal members. The trusses support 1 x 4 roof sheathing. The connection details of this structure are unknown as the original construction documents are not available for review. In 1962, this wing was converted into an animal facility. At that time, concrete masonry block walls were added to serve as interior partitions between the animal cages. It is unknown if these partitions connect to the underside of the existing wood roof. Due to the challenge of making this connection around the existing wood trusses, it is likely that it was not installed. For the purpose of this evaluation, it is assumed that this connection is not present, and the CMU partitions do not contribute to the mass of the low roof.

The south wing is a two-story concrete frame structure that contains infill unreinforced brick masonry walls. The high roof is constructed with a 6" thick concrete slab that spans approximately 7'-0" to concrete beams. The beams span up to 23'-9" in the north-south direction to concrete girders. These girders are supported by concrete columns and span between 10'-0" to 27'-0" in the east-west direction. At the second floor, the beams are oriented in the east-west direction and are spaced at approximately 2'-0" o.c. They span between 10'-0" to 20'-0" to concrete girders that are oriented in the north-south direction. The slab at the second floor is estimated to the 6 ½" thick. The available drawings do not show columns located around the perimeter of the south wing; however, these were observed in the field. The available drawings also do not show the size of the concrete members or their reinforcing.

<u>Structural system for lateral forces:</u> The lateral force-resisting system consists of the original 1930 unreinforced masonry shear walls that were retrofit with gunite in 1962. The masonry is 3 wythes thick and measures between 12" to 13". It contains a header course that varies in spacing. The masonry appears to have been infilled into a concrete frame. In some locations, a slight gap was observed between the top of the masonry wall and the underside of the concrete structure. A scratch test was performed in the field, and the mortar was easily scored which indicates the mortar contains limited cement. The masonry walls contain thick mortar joints, and the quality of construction varies from bay to bay. In general, it appears to be of low to medium quality. The details of the surrounding concrete frame are unknown. Given the building vintage, it is likely that non-ductile detailing was used. The columns and beams are likely shear-controlled and are expected to have limited displacement ductility.

In 1962, 3" and 4" thick layers of gunite was added to the face of the existing masonry walls. The gunitewas added to the outside face of wall, except where Building E abuts Building G. At this location, the gunite was added to the interior face of wall. The gunite is reinforced with #3 bars spaced at 12" o.c. in the vertical and horizontal direction. The placement of the vertical bars is staggered and shifts between the inside and outside face of the gunite. The gunite is connected to the concrete floor beams with 3/8" diameter anchors spaced at 2'-0" o.c. The gunite does not appear to be directly connected to the masonry, and the masonry is assumed to not be positively connected to the concrete frame. The gunite wall does not extend to the top of the existing footings. It is embedded approximately 1'-0' below grade and anchored to the outside face of the foundation stem wall. As such, it does not have bearing capacity for vertical gravity and seismic overturning loads. In addition to the gunite, a reinforced concrete stair tower as added on the west elevation of the south wing. It is comprised of 8" thick reinforced concrete load bearing walls that contain #4 vertical bars on each face that are spaced at 16" o.c and #3 horizontal bars on each face that are spaced at 11" o.c. The walls support a 7" reinforced concrete landing and stair stringers. The east wall of the stair tower is connected to the original west masonry wall with a row of 5/8" diameter bars spaced at 2'-0" o.c. at the top of the wall and with 3/8" diameter bolts spaced at 4'-0" o.c. at the bottom of the wall.

The existing wood diaphragm located on the north wing low roof was strengthened in 1962. At that time, a 3/8" thick layer of plywood was added over the original 1 x 4 wood sheathing. The plywood was connected with 8d stronghold hi-load nails spaced at 8" o.c. at the panel edges and with 1" x 0.135" stronghold hi-load nails spaced at 12" o.c. in the field. The connection of the roof diaphragm to the exterior masonry walls was also strengthened in 1962. Wood blocking and ledger members were added at the face of the masonry walls around the perimeter of the diaphragm. The ledger is bolted to the masonry wall with a 5/8" anchor spaced at approximately 14" o.c. However, the wood ledger is loaded in cross grain bending for out-of-plane loads. The original wood trusses act as diaphragm cross ties in the east-west direction; however, no cross ties are present in the north-south direction.

Building condition: The roof is in poor condition. On-going leaks at the roof were noted by the building engineer and observed in the field. Buckets are located to catch rainwater and the paint is peeling from the underside of the roof framing. Floor tiles are missing in a number of locations. Otherwise, the structure is in relatively good condition.

Building response in the 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:

- In the north wing, the exterior wall-to-diaphragm connection relies upon cross-grain bending of the wood ledger.
- No diaphragm cross ties are present in the N-S direction of the north wing.
- The exterior masonry walls in the north wing support gravity load from the wood roof with no secondary support.
- The masonry wall height-to-thickness ratios exceed those recommended by ASCE 41-17.
- Gaps were observed at the top of the masonry infill and the underside of the concrete frame.
- Given the building vintage, it is likely that the concrete frame contains non-ductile detailing and is shear controlled.
- The gunite is anchored to the concrete floor beams with a single row of horizontal bolts. The gunite does not appear to be connected to the masonry and the masonry is assumed to not be connected to the concrete frame.
- The gunite walls do not extend to the top of the existing foundations and not have any bearing capacity.
- The building is L-shaped and contains some geometric irregularities including a re-entrant corner and split-level diaphragms.
- Both the masonry and concrete walls are overstressed in the N-S direction between the second floor to the high roof when the load is distributed to the walls based upon relative rigidity.
- The concrete walls are overstressed in both directions between the second floor to the high roof when the load is assumed to be resisted by the concrete only with no participation from the masonry.

| 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 | 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 | Y |
| Geometry (vertical irregularities) | Y | URM wall height-to-thickness ratio | Y |
| Torsion | N | URM parapets or cornices | Y |
| Mass – vertical irregularity | N | URM chimney | N |
| Cripple walls | N | Heavy partitions braced by ceilings | N |
| Wood sills (bolting) | N | Appendages | N |
| Diaphragm continuity | Y | | |

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

In the south wing, Building E contains hollow clay tile partitions located along one side of the main corridor. In addition to risk imposed by the presence of this partition, in some locations, the lateral bracing from the adjacent

[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.

MEP distribution system is kicked to these partitions and will impose loads out-of-plane during a seismic event. Storage of items labeled as "biohazards" were observed in unbraced refrigerators. In addition, the building facility manager indicates the chemical waste from the adjacent hospital is stored inside the structure temporarily until it is transported for disposal. The lateral bracing of these items is unknown.

| 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 | Unbraced refrigerators that contain items labeled as "biohazards" were observed. Chemical waste from the hospital is also stored in the structure. |
| 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 | Hollow clay tile partitions are located adjacent to the main interior corridor. | Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc. | The building engineer notes that natural gas is supplied to Building E. However, it is unknown if the line was capped when the building was vacated. |

**Basis of Seismic Performance Level rating**

Building E is an L-shaped structure constructed in 1930 from unreinforced masonry. It consists of a one-story tall north wing and a two-story south wing. The floor elevations of these portions do not align. Due to this configuration, Building E is geometrically irregular and contains a re-entrant corner and split-level diaphragms. The structure was retrofit in 1962, and at that time, it appears a reasonable attempt was made to tie components of the lateral force-resisting system together and to strengthen the existing structure. Despite these improvements, the building contains structural deficiencies including poor diaphragm-to-wall connections, overstressed walls, and non-ductile detailing of the secondary concrete elements.

In 1962 improvements were made to the wood diaphragm. Plywood was added over the existing 1x sheathing that was nailed to new wood blocking and ledgers located at the interior face of the walls. The center of the ledger is bolted into the wall and is placed in cross grain bending when loaded out-of-plane. In the E-W direction, the existing wood trusses act as crossties for the diaphragm. However, no crossties are present in the N-S direction. This wall is vulnerable to falling outwards.

Although gunite was added to the face of the masonry walls; the walls are overstressed. When the lateral load is assumed to be shared between the masonry and the concrete walls, the average stress in the masonry is 21 psi and 51 psi at the first and second story, respectively, and the average stress in the concrete walls is 60 psi and 144 psi in the first and second story, respectively. When checked assuming that all of the shear is resisted by the concrete walls only, the average stress is 77 psi and 172 psi in the first and second story, respectively. In both cases, the computed stresses in the second story exceeds the ASCE 41-17 limits of 30 psi for masonry and 100 psi for concrete. The gunite walls are only anchored to the masonry walls using a single row of bolts located along the floor level. No anchorage of the masonry to the gunite is present between the floor levels. When the walls were installed, the exterior grade was excavated to expose the top of the existing foundation stem wall. The gunite was bolted into the side of the stem walls and does not extend down to the top of the footings. As such, the gunite walls do not bear on the foundations and any vertical overturning forces in the walls are resisted by the connection bolts in shear.

Finally, the reinforcing details for the 1930 concrete frame are not currently available for review. However, given common practices at that time, it is likely that non-ductile detailing was used. The south wing contains a concrete frame with URM infill, and in some locations a gap between the URM and the concrete members are visible. During a seismic event, as the URM degrades it may fall from the frame. It will be restrained from falling outwards by the gunite walls unless the anchors at the located at floor level fail. Although less common, the URM has the potential to fall inwards as it is does not have restraint in this direction. If the infill falls, the lateral loads would be then be resisted by the remaining concrete frame. This frame is likely non-ductile, and the members shear controlled. As such, it would have limited displacement ductility before failure.

The building is assigned a Seismic Performance Level Rating of V because the poor wall-to-diaphragm connections in the north wing, the potential for the masonry infill to fall or induce a shear failure in the building columns, the over stressed walls, and the presence of a likely non-ductile secondary concrete frame.

**Recommendations for further evaluation or retrofit**

Building E is currently vacated and is used for storage. Intermittent occupants currently enter the structure to retrieve and store items as well as maintain the building. If it is planned to re-occupy the structure in the future, it is recommended that a retrofit be completed at that time.

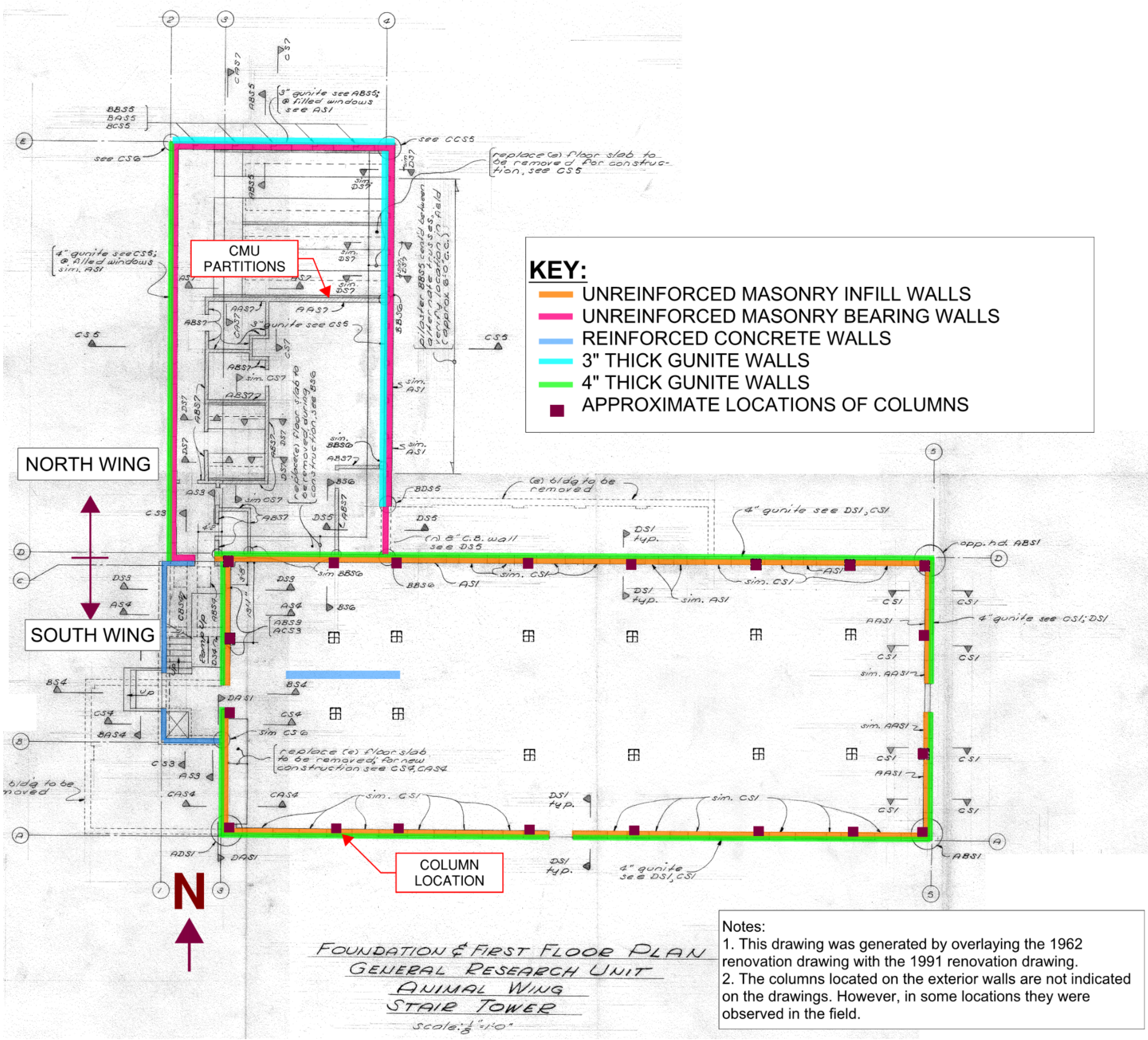
**Peer review comments on rating**

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 V.

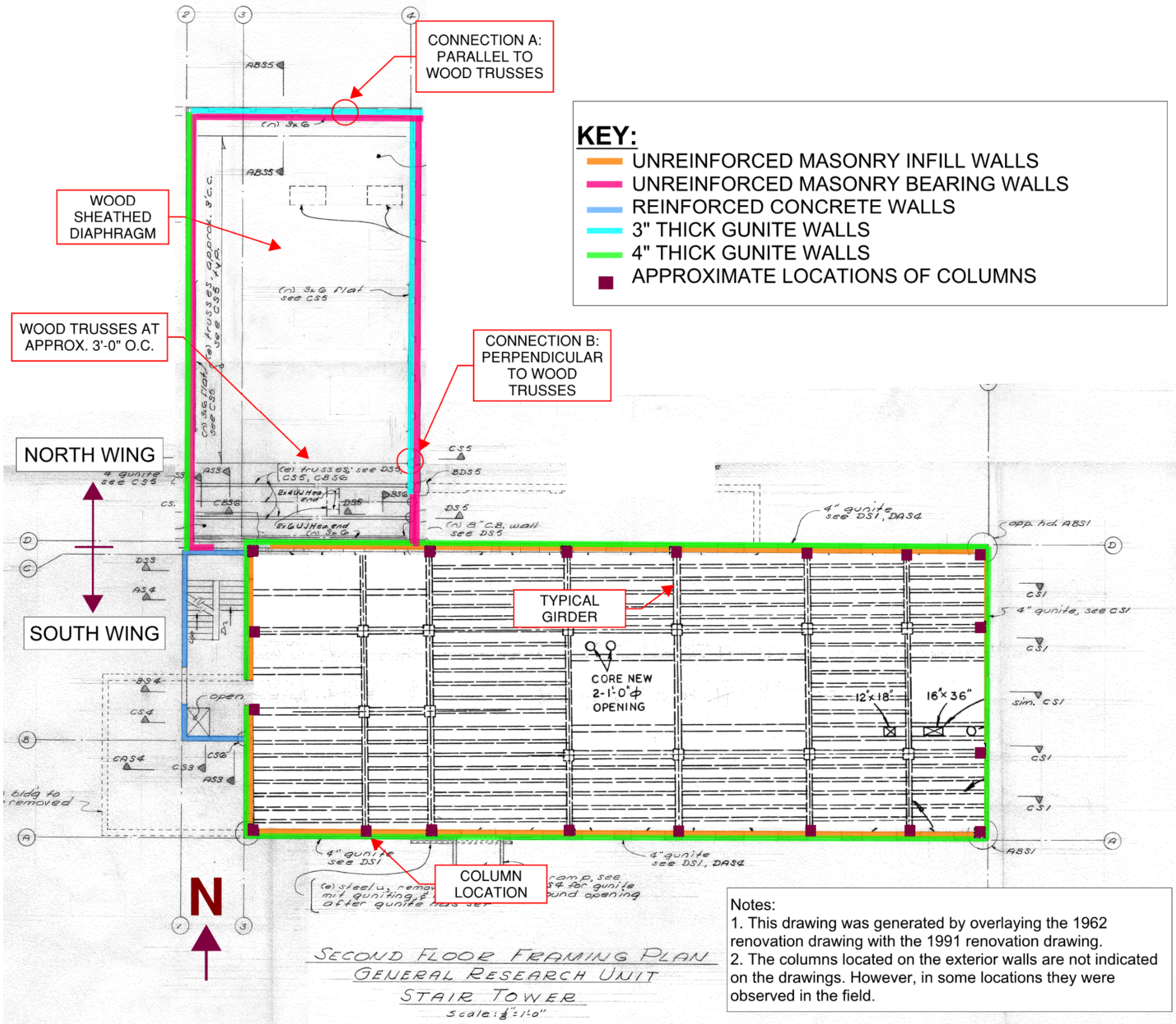
| Additional building data | Entry | Notes |
|---|---|---|
| Latitude | 37.78509 | |
| Longitude | -122.43847 | |
| Are there other structures besides this one under the same CAAN# | No | |
| Number of stories above lowest perimeter grade | 2 | North wing of L-shape building contains only one story. South wing contains two stories. |
| Number of stories (basements) below lowest perimeter grade | 0 | |
| Building occupiable area (OGSF) | 13,500 | |
| Risk Category per 2016 CBC 1604.5 | II | |
| Building structural height, $h_n$ | 25.58 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, $\beta$ | 0.75 | Estimated using ASCE 41-17 equation 4-4 and 7-18 |
| Estimated fundamental period | 0.23 sec | Estimated using ASCE 41-17 equation 4-4 and 7-18 |
| **Site data** | | |
| 975-year hazard parameters $S_s$, $S_1$ | 1.431g, 0.557g | 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, 1.743 | UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019) |

| | | |
|---|---|---|
| Ground motion parameters $S_{cs}$, $S_{c1}$ | 1.431g, 0.971g | UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards, Egan (2019) |
| $S_a$ at building period | 1.43g | W = 3,419 kips, V base = 5,870 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: 1930<br>Code: 1927 UBC | Applicable code assumed |
| Applicable code for partial retrofit | Renovation drawings dated 1962 and 1991<br>Codes: 1961 UBC and 1989 UBC | Applicable codes assumed |
| Applicable code for full retrofit | None | No full retrofit known |
| **Model building data** | | |
| Model building type north-south | <u>Original north wing:</u> URM Unreinforced Masonry Bearing Walls<br><u>Original south wing:</u> C3 Concrete Frames with Infill Masonry Shear Walls<br><u>Gunite retrofit</u>: C2 and C2a Concrete Shear Walls | Use Model Building Type C3 for UCOP spreadsheet. |

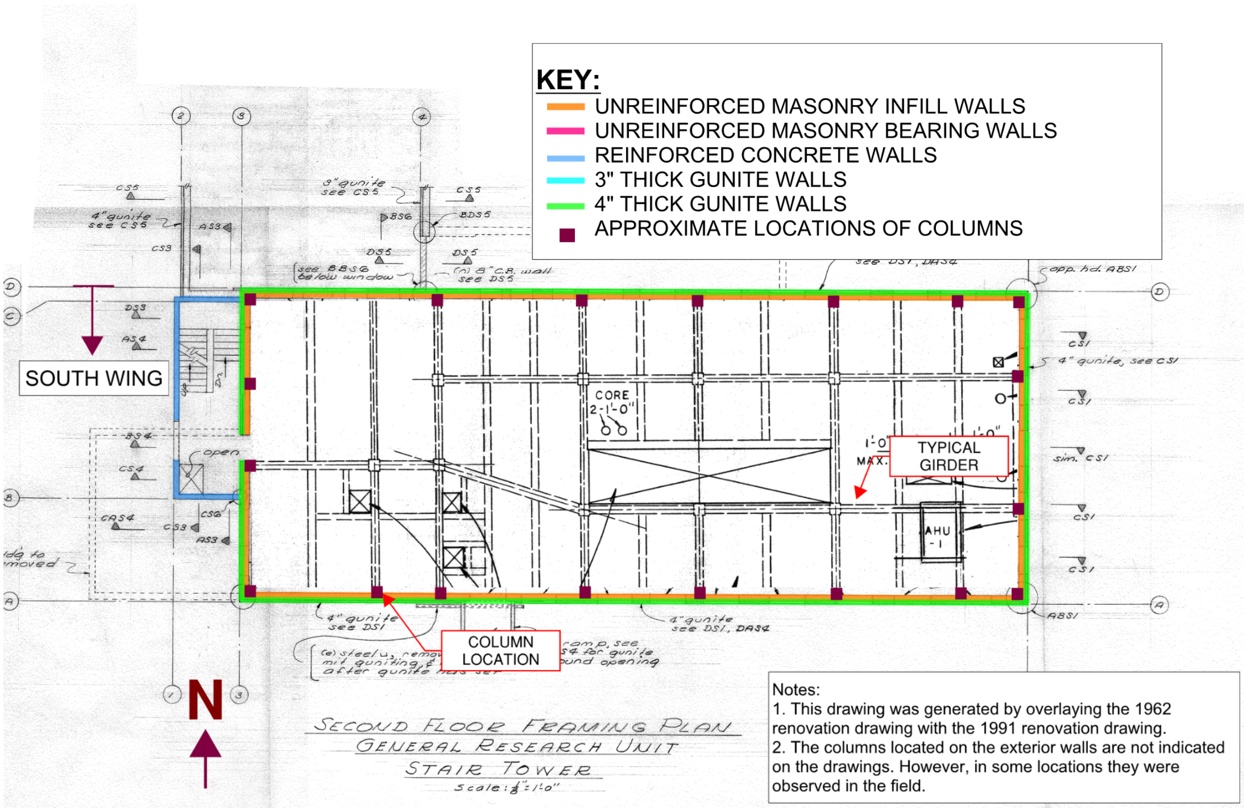
| | | |
|---|---|---|
| Model building type east-west | Original north wing: URM Unreinforced Masonry Bearing Walls<br>Original south wing: C3 Concrete Frames with Infill Masonry Shear Walls<br>Gunite retrofit: C2 and C2a Concrete Shear Walls | Use Model Building Type C3 for UCOP spreadsheet. |
| FEMA P-154 score | N/A | Not applicable as an ASCE 41 Tier 1 evaluation was performed |
| **Previous ratings** | | |
| Most recent rating | V | |
| Date of most recent rating | 2013 | Per the 2013 report, "Weak and nonductile building due to presence of unreinforced masonry infill and old concrete frame." |
| 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 |



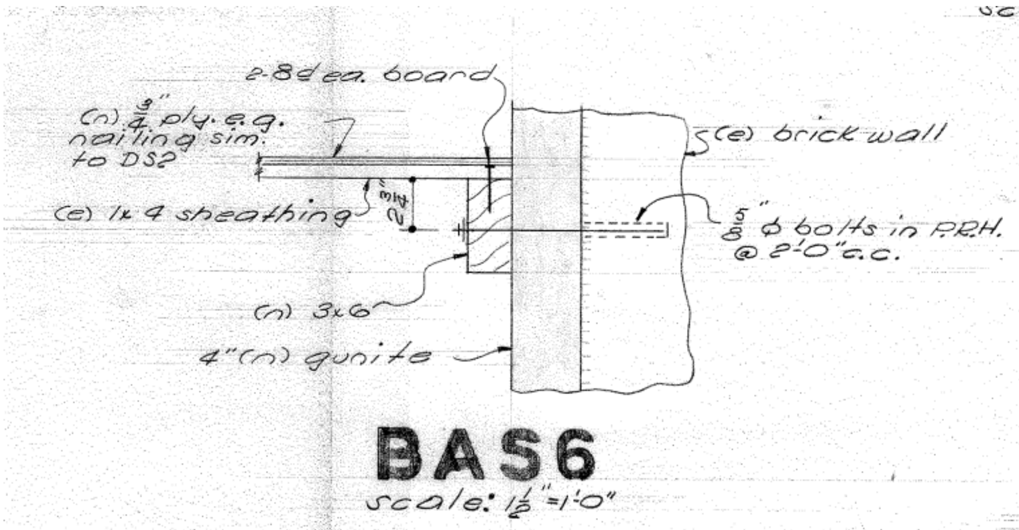

Lateral force-resisting system at 1st floor



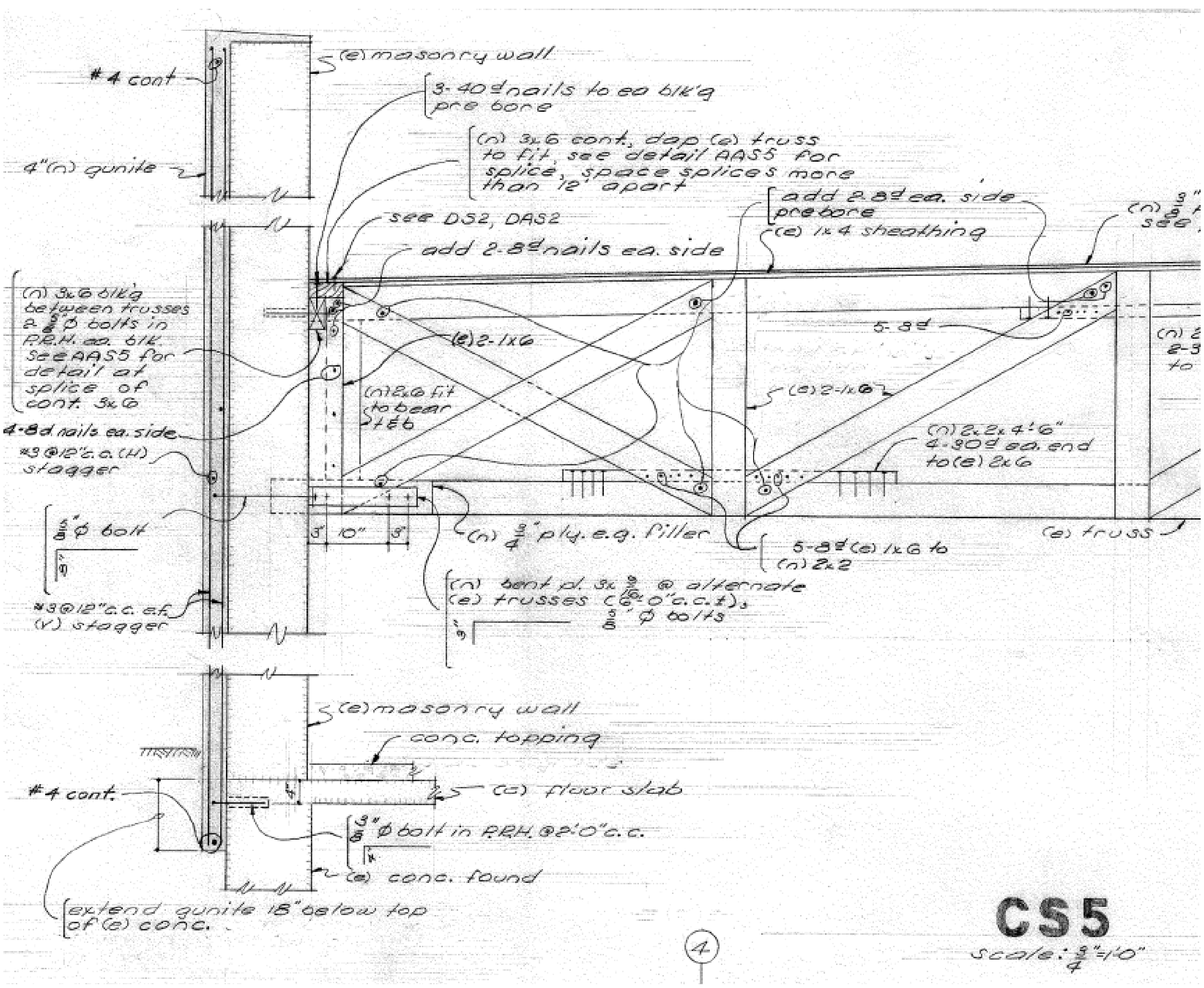

Lateral force-resisting system at 2nd floor and low roof




Lateral force-resisting system at high roof




Connection A: Typical diaphragm-to-wall connection parallel to the wood trusses

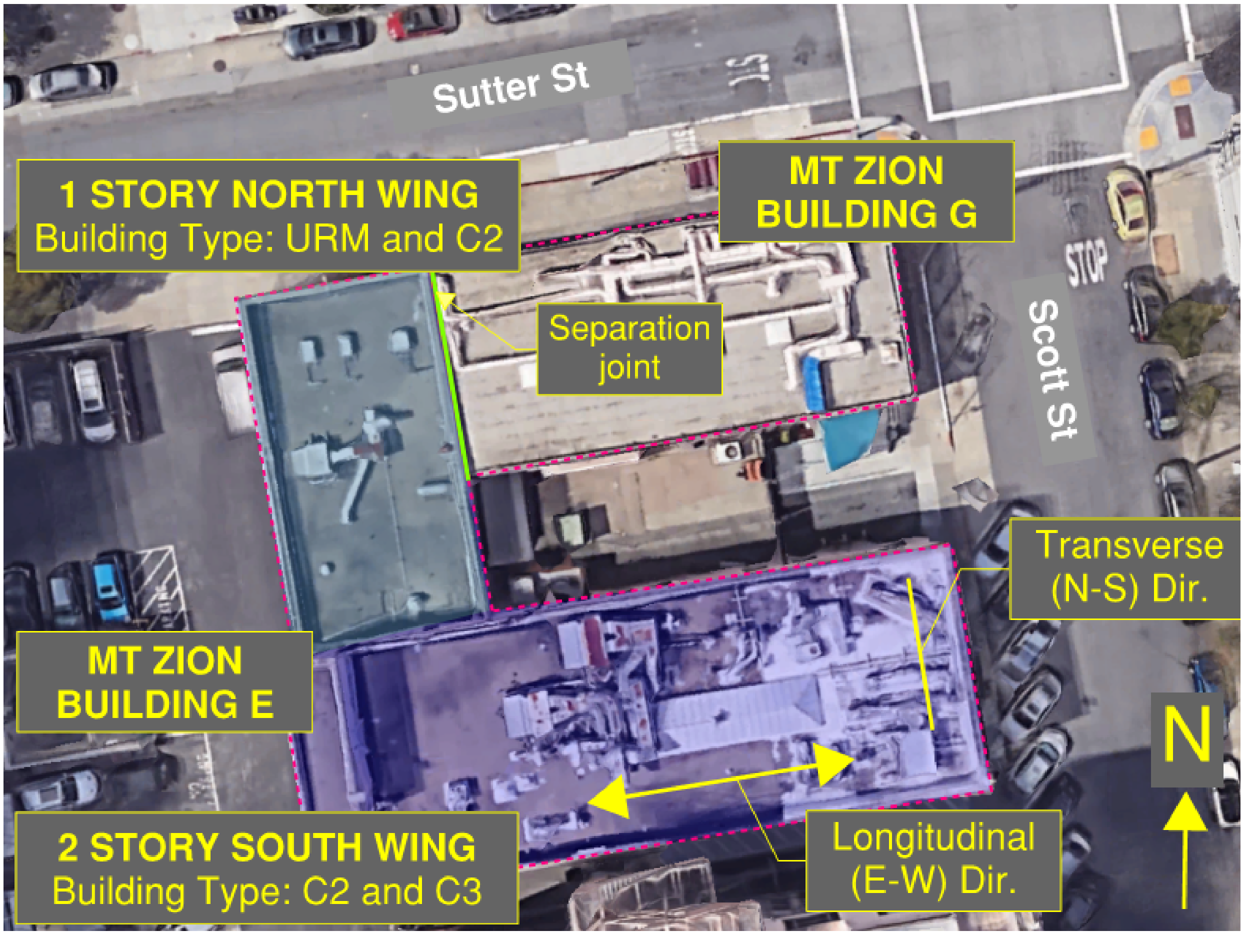



Connection B: Typical diaphragm-to-wall connection perpendicular to the wood trusses

# APPENDIX A

# Additional Images


Overview of Mt. Zion campus


Sutter St
1 STORY NORTH WING
Building Type: URM and C2
MT ZION
BUILDING G
Separation
joint
Scott St
Transverse
(N-S) Dir.
MT ZION
BUILDING E
N
2 STORY SOUTH WING
Building Type: C2 and C3
Longitudinal
(E-W) Dir.

Plan

East elevation (looking west)


North elevation (looking southeast on Sutter Street)

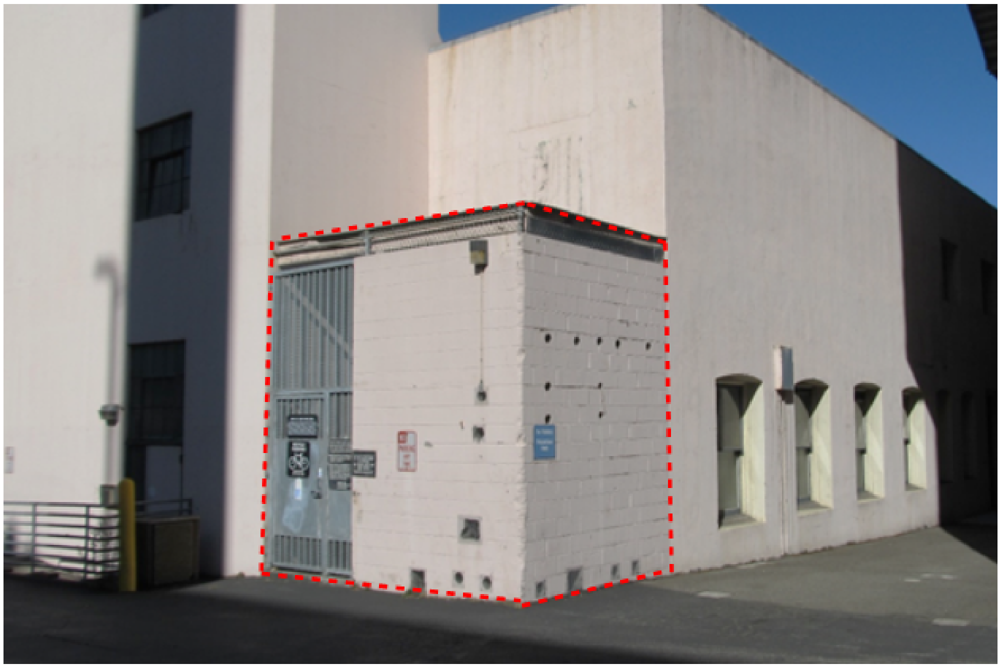

North elevation (looking southeast from roof of the north wing of Building E)

Re-entrant corner at north elevation of the south wing and east elevation of the north wing (looking southwest from Building G)

Exposed solid clay URM wall at east elevation of north wing (looking west)


West elevation (looking east)

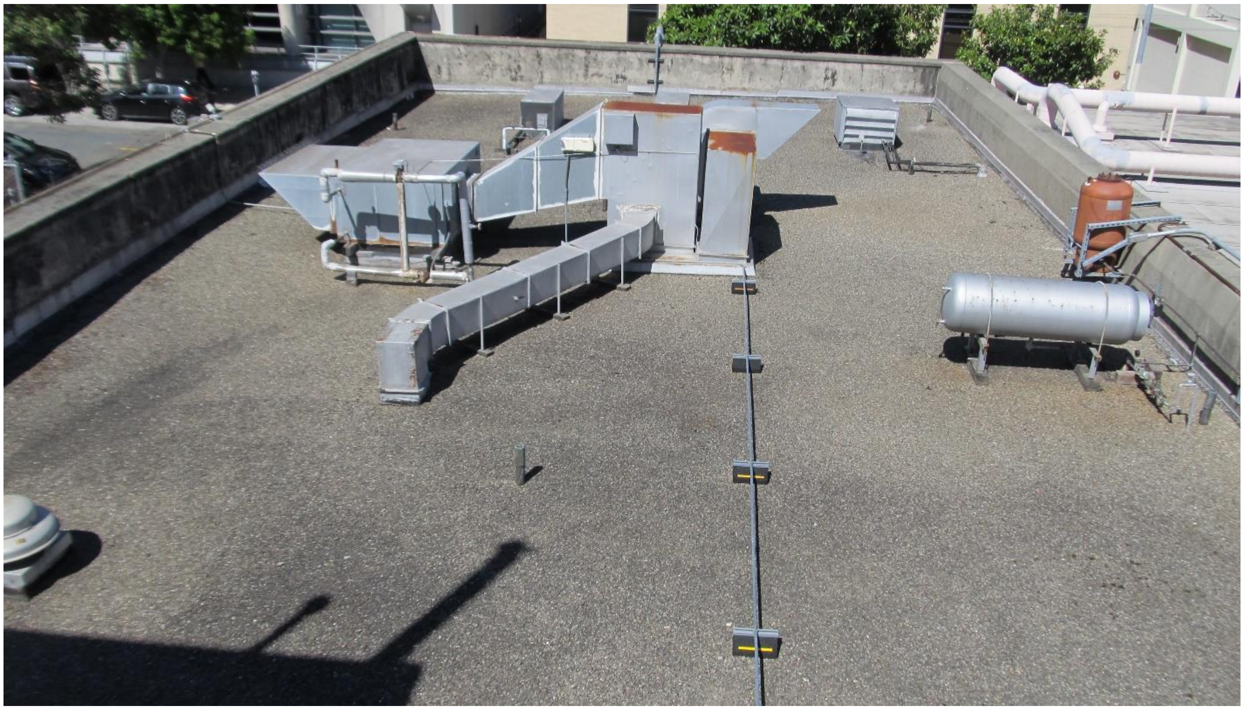

Stair tower within dashed redlines on west elevation added after the 1962 alterations (looking northeast)



South elevation (looking northeast) with CMU bike shed in the foreground


Seismic joint between Building G and Building E
(looking south, Building G on the left and Building E on the right)

Roof at south wing of Building E (looking southwest)



Roof at north wing of Building E (looking north)

Concrete slab over beams with URM infill wall at second floor


Typical concrete framing on the underside of second floor


Original URM masonry walls with CMU infill after the 1962 alterations at second story

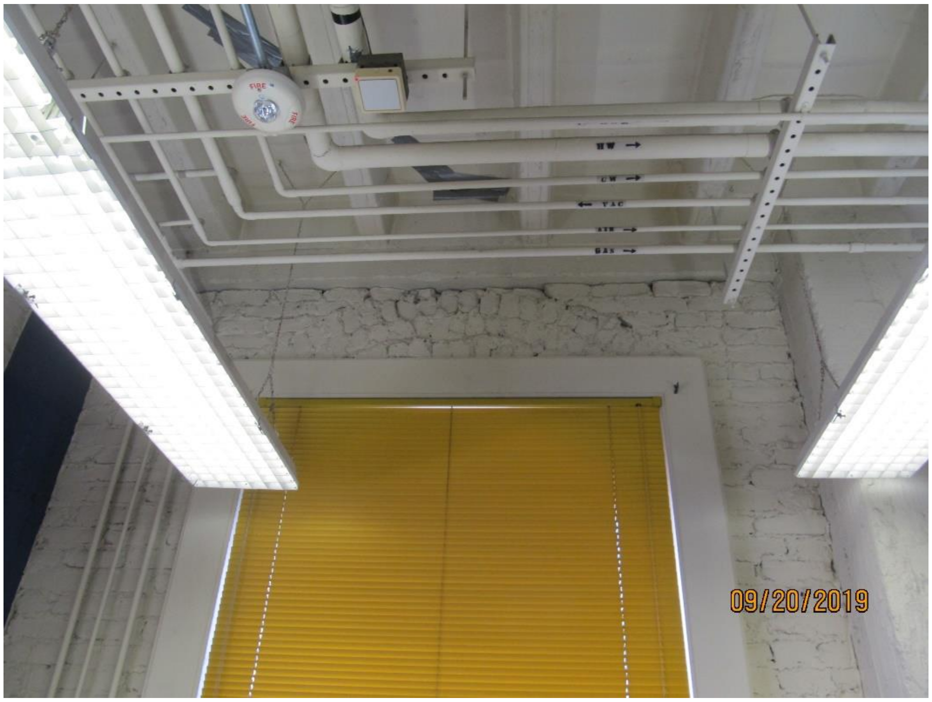
Water damage due to deficient waterproofing on south wing


Hollow clay tile partitions at second story (looking north)


Corridor at second story with hollow clay tile partitions on the right side (looking east)


Pipe system braced to the hollow clay tile partitions at second story

Fixed laboratory furniture with gas supply at second story (looking northwest)



Gap between bottom of concrete framing and top of URM infill (looking east)

Scraped mortar of URM wall using flat-head screwdriver on southeast corner of building at the first story


Unbraced refrigerators in laboratory with biohazard tag at second story (looking northeast)

MEP bracing at first story above the lay-in ceiling (looking west)

South wing of Building E (looking northeast)

# APPENDIX B

# ASCE 41-17 Tier 1 Checklists (Structural)

| UC Campus: | San Francisco | | | Date: | 10/10/2019 | | |
|---|---|---|---|---|---|---|---|
| Building CAAN: | 2024 | Auxiliary CAAN: | | By Firm: | RUTHERFORD + CHEKENE | | |
| Building Name: | UCSF Mt. Zion Building E | | | Initials: | EGM | Checked: | BL |
| Building Address: | 1657 Scott 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 |
|---|---|
| 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)<br><br>**Comments:** Concrete diaphragms are located at the high roof and second floor. A wood-framed diaphragm is located at the low roof. These diaphragms deliver lateral load to the exterior shear walls situated around the building perimeter. The walls consist of unreinforced clay masonry brick walls that were retrofit with gunite (shotcrete). The shear walls are continuous to the foundation and are founded on concrete strip footings. |
| 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)<br><br>**Comments:** Mt. Zion Building G is located in close proximity to the east elevation of Building E. The clear distance between these structures as shown on the drawings is 3”. This measurement was confirmed in the field. Based upon the Building G height of 15’-6” , the required gap is 2.8”. |
| 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)<br><br>**Comments:** There are no mezzanines in the building. |

### 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)<br><br>**Comments:** The total wall area increases from the roof to the first floor. |
| 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)<br><br>**Comments:** The total wall area increases from the roof to the first floor, and the story heights in the south wing do not have significant variations. |

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# ASCE 41-17
# Collapse Prevention Basic Configuration Checklist

| | |
|---|---|
| **C** (selected) **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)<br><br>**Comments:** All walls are continuous to the foundation. |
| **C NC** (selected) **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)<br><br>**Comments:** In the east-west direction, the net horizontal of the force-resisting system decreases by 60% in the story between the low roof of the north wing and the high roof of the south wing. |
| **C** (selected) **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)<br><br>**Comments:** Effective mass has a slight increase from the high roof to the 2[nd] floor, however, this increase is approximately 3%. |
| **C** (selected) **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)<br><br>**Comments:** Building has an L-shaped plan configuration; however, the seismic-force-resisting system is mainly located around the perimeter of the structure. |

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

### GEOLOGIC SITE HAZARD

| | **Description** |
|---|---|
| **C** (selected) **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)<br><br>**Comments:** Per "Table 1 - UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards" by Egan (2019), the liquefaction potential is very low. |
| **C** (selected) **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)<br><br>**Comments:** Per "Table 1 - UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards" by Egan (2019), the site is located on a gentle slope (approximately 1-degree), and it not susceptible to landslide. |

**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 ( ) 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)<br><br>**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 | 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)<br><br>**Comments:**<br>The building width is B = 45’-0” from Grid A to D. The building height from the 1st floor to the high roof is H = 25”-7”,<br>B/H = 1.76<br>Sa = 1.43g for at BSE-2E<br>0.6x Sa = 0.86<br>B/H > 0.6 Sa. |
| ( ) 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)<br><br>**Comments:** Per “Table 1 - UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards” by Egan (2019), the soil is classified as Site Class D. Per details on Sheet S1 in 1962 structural drawings, concrete strip footings appear to be restrained by a concrete slab-on-grade. The slab reinforcing and its connection to the footings are unknown. |

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# ASCE 41-17
# Collapse Prevention Structural Checklist For Building Type URM-URMA

## Low And Moderate Seismicity

**Note: This checklist is being used to evaluate the original north wing.**

## SEISMIC-FORCE-RESISTING SYSTEM

| C NC N/A U | Description |
|---|---|
| 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)<br><br>**Comments:** In both the north and south wing, considering the masonry walls only, at the first story, there are 3 lines of walls in the longitudinal (E-W) direction and 4 lines of walls in the transverse (N-S) direction. At the second story, there are 2 lines of walls in the longitudinal (E-W) direction and 2 lines of walls in the transverse (N-S) direction. |
| NC (selected) | SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 30 lb/in.$^2$ (0.21 MPa) for clay units and 70 lb/in.$^2$ (0.48 MPa) for concrete units. (Commentary: Sec. A.3.2.5.1. Tier 2: Sec. 5.5.3.1.1)<br><br>**Comments:** The story shear is proportioned based upon relative rigidity of masonry walls and the combination of the concrete and gunite walls. The average shear stresses in the longitudinal (E-W) direction are 18 psi (first floor to second floor) and 27 psi (second floor to high roof). The average shear stresses in the transverse (N-S) direction are 21 psi (first floor to second floor) and 51 psi (second floor to high roof).The calculated wall stresses in the masonry between the second floor to the high roof exceed the ASCE 41 limit of 30 psi for clay units in the N-S direction. |

## CONNECTIONS

| C NC N/A U | Description |
|---|---|
| NC (selected) | WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections 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)<br><br>**Comments:** The connection details utilize wood members in cross-grain bending. In addition, there are no crossties developed into the diaphragm for the north wall. |
| NC (selected) | WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)<br><br>**Comments:** The roof connection located at the north wall of the structure is reliant upon cross-grain bending. |

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

| | | | | | | | |
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## Collapse Prevention Structural Checklist For Building Type URM-URMA

| 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)<br><br>**Comments:** In the north wing, plywood was added over the existing roof sheathing; it was nailed to new blocking which was bolted through the existing masonry to the new gunite walls. |
| ( ) ( ) (•) ( ) | GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)<br><br>**Comments:** There are no columns in the north wing. |

## 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
( ) (•) ( ) ( )

PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than the following: (Commentary: Sec. A.3.2.5.2. Tier 2: Sec. 5.5.3.1.2):

| | |
|---|---|
| Top story of multi-story building | 9 |
| First story of multi-story building | 15 |
| All other conditions | 13 |

**Comments:**

The following table applies to original unreinforced masonry walls

| Condition | Height (ft) | Wall thickness (in) | Height/Thickness | Limit |
|---|---|---|---|---|
| (a) Top story of multi-story building | 12.5 ($2^{nd}$ floor to high roof) | 13 | 11.5 | 9 |
| (b) First story of multi-story building | 13.08 ($1^{st}$ floor to $2^{nd}$ floor) | 13 | 12.1 | 15 |
| (c ) All other conditions | 18.25 ($1^{st}$ floor to low roof) | 13 | 16.8 | 13 |

The height to thickness ratio is exceeded for (a) Top story of multi-story building and for (c) all other conditions.

C NC N/A U
( ) ( ) ( ) (•)

MASONRY LAYUP: Filled collar joints of multi-wythe masonry walls have negligible voids. (Commentary: Sec. A.3.2.5.3. Tier 2: Sec. 5.5.3.4.1)

**Comments:** The condition at the collar joints is unknown.

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# ASCE 41-17
# Collapse Prevention Structural Checklist For Building Type URM-URMA

## DIAPHRAGMS (STIFF OR FLEXIBLE)

| | Description |
|---|---|
| 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)<br><br>**Comments:** There are no large openings immediately adjacent to masonry shear walls. |
| C: ◉ NC: ○ N/A: ○ U: ○ | OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)<br><br>**Comments:** There are no large openings immediately adjacent to exterior masonry shear walls. |

## 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)<br><br>**Comments:** The wood trusses spaced at ±3'-0" o.c. function as cross ties in the (E-W) direction. There are no cross ties in the N-S direction. |
| 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)<br><br>**Comments:** It is unknown if the original roof contains straight sheathing. However, the north wing roof aspect ratio is 1W:1.97L which is compliant, and plywood was added to the roof in 1962. |
| 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)<br><br>**Comments:** The flexible diaphragm on the north wing was improved during the 1962 alterations by the addition of 3/8" plywood sheathing placed over the existing 1x4 sheathing. |
| 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)<br><br>**Comments**: It is unknown if the original roof contains diagonal sheathing. However, the north wing roof was strengthened with plywood in 1962, and it contains an aspect ratio of 1W:1.97L which is compliant. |
| C: ◉ NC: ○ N/A: ○ U: ○ | OTHER DIAPHRAGMS: The 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)<br><br>**Comments:** The north wing contains a wood diaphragm. |

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

# ASCE 41-17
# Collapse Prevention Structural Checklist For Building Type URM-URMA

## CONNECTIONS

| C / NC / N/A / U | Description |
|---|---|
| U | STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)<br><br>**Comments:** The stiffness of the wall anchors is unknown. |
| NC | BEAM, GIRDER, AND TRUSS SUPPORTS: Beams, girders, and trusses supported by unreinforced masonry walls or pilasters have independent secondary columns for support of vertical loads. (Commentary: Sec. A.5.4.5. Tier 2: Sec. 5.7.4.4)<br><br>**Comments:** Wood trusses are supported by exterior masonry walls with no independent support. |

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# ASCE 41-17
# Collapse Prevention Structural Checklist For Building Type C3-C3A

## Low And Moderate Seismicity

**Note: This checklist is being used to evaluate the original south wing.**

## SEISMIC-FORCE-RESISTING SYSTEM

| C NC N/A U | Description |
|---|---|
| **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)<br><br>**Comments:** In both the north and south wing, there are 4 lines of walls in the longitudinal (E-W) direction and 4 lines of walls in the transverse (N-S) direction between the first and second floor. There are 3 lines of walls in the longitudinal (E-W) direction and 3 lines of walls in the transverse (N-S) direction between the second floor and the roof. |
| **N/A** (selected) | SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.$^2$ (0.48 MPa). (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1)<br><br>**Comments:** There are no reinforced masonry walls. |
| **NC** (selected) | SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 30 lb/in.$^2$ (0.21 MPa) for clay units and 70 lb/in.$^2$ (0.48 MPa) for concrete units. Bays with openings greater than 25% of the wall area shall not be included in $A_w$ of Eq. (4-8). (Commentary: Sec. A.3.2.5.1. Tier 2: Sec. 5.5.3.1.1)<br><br>**Comments:** The story shear is proportioned based upon relative rigidity of masonry walls and the combination of cast-in-place concrete and gunite walls. The average shear stresses in the longitudinal (E-W) direction are 18 psi (first floor to second floor) and 27 psi (second floor to high roof). The average shear stresses in the transverse (N-S) direction are 21 psi (first floor to second floor) and 51 psi (second floor to high roof).The calculated wall stresses in the masonry between the second floor to the high roof exceed the ASCE 41 limit of 30 psi for clay units in the N-S direction. |
| **NC** (selected) | INFILL WALL CONNECTIONS: Masonry is in full contact with frame. (Commentary: A.3.2.6.1. Tier 2: Secs. 5.5.3.5.1 and 5.5.3.5.3)<br><br>**Comments:** Gaps between the infill walls and the masonry frame were observed in the field. |

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# ASCE 41-17
# Collapse Prevention Structural Checklist For Building Type C3-C3A

## CONNECTIONS

| | Description |
|---|---|
| **C NC N/A U**<br>( ) (•) ( ) ( ) | TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)<br><br>**Comments:** During the 1962 renovation, 3/8” diameter bolts spaced at 3’-0” o.c. were provided in the south wing at each floor level to transfer loads from the original concrete floor beams into the new gunite walls. There does not appear to be a connection of the gunite directly to the masonry, and it is assumed there is no connection of the masonry infill to the concrete frame. |
| **C NC N/A U**<br>( ) ( ) ( ) (•) | CONCRETE COLUMNS: All concrete columns are doweled into the foundation with a minimum of four bars. (Commentary: Sec. A.5.3.2. Tier 2: Sec. 5.7.3.1)<br><br>**Comments:** The connection between the foundations and the concrete columns is unknown as the original drawings are not available for review. |

## 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**<br>( ) (•) ( ) ( ) | 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)<br><br>**Comments:** The original construction details are not available for review. Per the list of “UCSF owned, leased buildings for seismic evaluation,” the structure was built in 1930. Given this vintage, it is likely that the gravity columns contain non-ductile detailing and are shear controlled. |
| **C NC N/A U**<br>( ) ( ) (•) ( ) | 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)<br><br>**Comments:** There are no flat slabs in this building. |

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 C3-C3A

| C NC N/A U | Description |
|---|---|
| NC | PROPORTIONS: The height-to-thickness ratio of the unreinforced infill walls at each story is less than 9. (Commentary: A.3.2.6.2. Tier 2: Sec. 5.5.3.1.2)<br><br>**Comments:** The height to thickness ratios are as follows: |

| Condition | Height (ft) | Wall thickness (in) | Height/Thickness | Limit |
|---|---|---|---|---|
| (a) Top story of multi-story building | 12.5 (2nd floor to high roof) | 13 | 11.5 | 9 |
| (b) First story of multi-story building | 13.08 (1st floor to 2nd floor) | 13 | 12.1 | 15 |
| (c ) All other conditions | 18.25 (1st floor to low roof) | 13 | 16.8 | 13 |

| C NC N/A U | Description |
|---|---|
| C | CAVITY WALLS: The infill walls are not of cavity construction. (Commentary: Sec. A.3.2.6.3. Tier 2: Sec. 5.5.3.5.2)<br><br>**Comments:** Header courses were observed in the field which indicates that the walls are not of cavity construction. |
| C | INFILL WALLS: The infill walls are continuous to the soffits of the frame beams and to the columns to either side. (Commentary: Sec. A.3.2.6.4. Tier 2: Sec. 5.5.3.5.3)<br><br>**Comments:** Where observed, the masonry was constructed to the edges of the concrete beams and columns. |

## DIAPHRAGMS (STIFF OR FLEXIBLE)

| C NC N/A U | Description |
|---|---|
| NC | 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)<br><br>**Comments:** The roof of the north wing and south wing are located at different elevations. |
| NC | OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)<br><br>**Comments:** The stair tower added during the 1962 alteration contains openings on the west elevation of the building with the same dimensions as the concrete shear walls. |

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 C3-C3A

| | |
|---|---|
| **C** (selected) **NC N/A U** | OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)<br><br>**Comments:** There are no large openings immediately adjacent to exterior masonry shear walls. |

## 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)<br><br>**Comments:** The south wing contains concrete diaphragms. |
| 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)<br><br>**Comments:** The south wing contains concrete diaphragms. |
| 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)<br><br>**Comments:** The south wing contains concrete diaphragms. |
| 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)<br><br>**Comments:** The south wing contains concrete diaphragms. |
| **C** (selected) 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)<br><br>**Comments:** The south wing contains concrete diaphragms. |

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

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| UC Campus: | San Francisco | | | Date: | 10/10/2019 | | |
| Building CAAN: | 2024 | Auxiliary CAAN: | | By Firm: | RUTHERFORD + CHEKENE | | |
| Building Name: | Mount Zion, Building E | | | Initials: | EGM | Checked: | BL |
| Building Address: | 1657 Scott St, San Francisco, CA 94115 | | | Page: | 5 | of | 5 |

# ASCE 41-17
# Collapse Prevention Structural Checklist For Building Type C3-C3A

## CONNECTIONS

| | Description |
|---|---|
| C NC **N/A** U (N/A selected) | 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)<br><br>**Comments:** The building has strip footings. |
| C NC **N/A** U (N/A selected) | STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)<br><br>**Comments:** The south wing does not contain wood elements. |

| UC Campus: | San Francisco | | | Date: | 10/10/2019 | | |
|---|---|---|---|---|---|---|---|
| Building CAAN: | 2024 | Auxiliary CAAN: | | By Firm: | RUTHERFORD + CHEKENE | | |
| Building Name: | Mount Zion, Building E | | | Initials: | EGM | Checked: | BL |
| Building Address: | 1657 Scott 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

**Note: This checklist is being used to evaluate the 1962 gunite retrofit.**

## Seismic-Force-Resisting System

| | Description |
|---|---|
| C NC N/A U<br>○ ● ○ ○ | 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)<br><br>**Comments:** In the south wing of the building, the building has interior gravity columns, as shown in the floor plans in the 1991 structural drawings. At the building perimeter, it contains concrete beams and columns with infilled masonry walls. The exterior columns are not shown on the drawings.<br><br>In the north wing of the building, the low wood roof is directly supported by the exterior masonry walls. |
| C NC N/A U<br>● ○ ○ ○ | 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)<br><br>**Comments:** There are 4 lines of walls in the longitudinal (E-W) direction and 4 lines of walls in the transverse (N-S) direction between the first and second floor. There are 3 lines of walls in the longitudinal (E-W) direction and 3 lines of walls in the transverse (N-S) direction between the second floor and the roof. |
| C NC N/A U<br>○ ● ○ ○ | 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)<br><br>**Comments:**<br>Concrete compressive strength of f'c = 2,500 psi is assumed according to the building vintage and Table 4-2 in ASCE 41-17.<br><br><u>Shear stress in the concrete (gunite + cast-in-place concrete) when the load is shared with the URM walls based upon relative rigidity:</u><br>In the E-W direction, the stresses are 49 psi and 77 psi in the first and second story, respectively.<br>In the N-S direction, the stresses are 60 psi and 144 psi in the first and second story, respectively. The limit of 100 psi is exceeded in the second story in the N-S direction.<br><br><u>Shear stress in the concrete (gunite + cast-in-place concrete) when the concrete is assumed to resist the entire base shear:</u><br>In the E-W direction, the stresses are 63 psi and 99 psi in the first and second story, respectively.<br>In the N-S direction, the stresses are 77 psi and 172 psi in the first and second story, respectively. The limit of 100 psi is exceeded in the second story in the N-S direction. |

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

| | | | | | | |
|---|---|---|---|---|---|---|
| UC Campus: | **San Francisco** | | Date: | **10/10/2019** | | |
| Building CAAN: | **2024** | Auxiliary CAAN: | By Firm: | **RUTHERFORD + CHEKENE** | | |
| Building Name: | **Mount Zion, Building E** | | Initials: | **EGM** | Checked: | **BL** |
| Building Address: | **1657 Scott St, San Francisco, CA 94115** | | Page: | **2** | of | **4** |

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

| 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)<br><br>**Comments:** Per Detail CS1 & DS1 in the 1962 drawings, 4" thick gunite walls are typically reinforced with #3 at 12" o.c. e.w. (ρ = 0.00229). Per Detail AS3 in the 1962 drawings, the 8" thick concrete walls added for the new stair tower are vertically reinforced with #4 at 16" o.c. e.f. (ρ = 0.00313) and are horizontally reinforced with #3 at 11" o.c. e.f. (ρ = 0.0025). |

## Connections

| C NC N/A U | Description |
|---|---|
| NC (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)<br><br>**Comments:** The north wing contains a flexible wood diaphragm. The connection details of the walls to the exterior walls utilizes wood members in cross-grain bending. |
| NC (selected) | 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)<br><br>**Comments:** During the 1962 renovation, 3/8" diameter bolts spaced at 3'-0" o.c. were provided in the south wing at each floor level to transfer loads from the original concrete floor beams into the new gunite walls. There does not appear to be a connection of the gunite directly to the masonry, and it is assumed there is no connection of the masonry infill to the concrete frame.<br><br>In the north wing, plywood was added over the existing roof sheathing, and it was nailed to new blocking which was bolted through the existing masonry to the new gunite walls. |
| NC (selected) | 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)<br><br>**Comments:** The original URM walls are not positively connected to the foundations, as is typical of URM buildings. Gunite walls added in 1962 do not extend to the top of the foundations and have no bearing capacity. They were bolted into the sides of the existing foundation stem walls with 3/8" diameter bolts spaced at 2'-6", 3'-0", and 4'-0" o.c. The new reinforced concrete walls that were added in 1962 are doweled into new strip footings with vertical dowels that match the size and spacing of the wall reinforcing. |

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

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| UC Campus: | **San Francisco** | | | Date: | **10/10/2019** | | |
| Building CAAN: | **2024** | Auxiliary CAAN: | | By Firm: | **RUTHERFORD + CHEKENE** | | |
| Building Name: | **Mount Zion, Building E** | | | Initials: | **EGM** | Checked: | **BL** |
| Building Address: | **1657 Scott St, San Francisco, CA 94115** | | | Page: | **3** | of | **4** |

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

## 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 |
|---|---|
| N/A | 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)<br><br>**Comments:** This checklist is used to evaluate the gunite retrofit only. Refer to the C3 checklist used to evaluate the concrete components in the south wing. |
| N/A | 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)<br><br>**Comments:** There are no flat slabs in this building. |
| N/A | COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)<br><br>**Comments:** There are no coupling beams in this building. |

### Diaphragms (Stiff Or Flexible)

| C NC N/A U | Description |
|---|---|
| NC | 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)<br><br>**Comments:** The roof of the north wing and south wing are located at different elevations. |
| NC | OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)<br><br>**Comments:** The stair tower added during the 1962 alteration contains openings on the west elevation of the building with the same dimensions as the concrete shear walls. |

### Flexible Diaphragms

| C NC N/A U | Description |
|---|---|
| NC | CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)<br><br>**Comments:** The wood trusses spaced at ±3’-0” o.c. function as cross ties in the E-W direction of the north wing. There are no cross ties in the N-S direction. |

| | | | | | | |
|---|---|---|---|---|---|---|
| UC Campus: | San Francisco | | Date: | 10/10/2019 | | |
| Building CAAN: | 2024 | Auxiliary CAAN: | By Firm: | RUTHERFORD + CHEKENE | | |
| Building Name: | Mount Zion, Building E | | Initials: | EGM | Checked: | BL |
| Building Address: | 1657 Scott St, San Francisco, CA 94115 | | Page: | 4 | of | 4 |

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

| C NC N/A U | |
|---|---|
| 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)<br><br>**Comments:** It is unknown if the original roof contains straight sheathing. However, the north wing roof aspect ratio is 1W:1.97L which is compliant and 3/8" plywood sheathing was placed over the existing 1x4 sheathing. |
| 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)<br><br>**Comments:** The flexible diaphragm on the north wing was improved during the 1962 alterations by the addition of 3/8" plywood sheathing placed over the existing 1x4 sheathing. |
| 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)<br><br>**Comments:** It is unknown if the original roof contains diagonal sheathing. However, the north wing roof was strengthened with plywood in 1962, and it contains an aspect ratio of 1W:1.97L which is compliant. |
| 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)<br><br>**Comments:** The building contains wood and concrete diaphragms. |

### Connections

| | Description |
|---|---|
| C: ○ NC: ○ N/A: ◉ U: ○ | UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)<br><br>**Comments:** The building has strip footings. |

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

# APPENDIX C

# UCOP Seismic Safety Policy Falling Hazards Assessment Summary

| UC Campus: | San Francisco | | | Date: | 10/10/2019 | | |
|---|---|---|---|---|---|---|---|
| Building CAAN: | 2024 | Auxiliary CAAN: | | By Firm: | Rutherford+Chekene | | |
| Building Name: | UCSF Mt. Zion Building E | | | Initials: | EGM | Checked: | BL |
| Building Address: | 1657 Scott 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)**<br><br>**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**<br><br>**Comments:** No heavy masonry or stone veneer is located near exit ways or public access areas because it is unlikely the original URM parapets contain a veneer wythe and are faced with gunite. |
| P ☒ N/A ☐ | **Unbraced masonry parapets, cornices, or other ornamentation above exit ways or public access areas**<br><br>**Comments:** Masonry parapets are located around the perimeter of the structure, but are faced with gunite. |
| P ☒ N/A ☐ | **Unrestrained hazardous material storage**<br><br>**Comments:** Items marked as biohazards are being stored in unbraced refrigerators. In addition, the facility manager indicates that chemical waste from the adjacent hospital is stored inside the structure. |
| P ☐ N/A ☒ | **Masonry chimneys**<br><br>**Comments:** No masonry chimneys are in the building. |
| P ☒ N/A ☐ | **Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.**<br><br>**Comments:** The UCSF Mt. Zion campus assistant engineer indicates that gas is supplied to the lab benches in Building E. Since Building E is vacant, it is unknown if the gas supply was capped when the occupants relocated. It is unknown is the gas lines are braced. |
| P ☒ N/A ☐ | **Other:**<br><br>**Comments:** The structure contains hollow clay tile partitions along the interior corridors. In addition, in some locations the lateral bracing for the MEP piping that runs above the corridor is kicked to these partitions and will load them out-of-plane. |
| P ☐ N/A ☐ | **Other:**<br><br>**Comments:** |
| P ☐ N/A ☐ | **Other:**<br><br>**Comments:** |

Falling Hazards Risk: Medium

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

# APPENDIX D

# Quick Check Calculations

# Flat Load Tables

| STAIR TOWER ROOF | Seismic Weight psf | Dead Load psf | Remarks |
|---|---|---|---|
| Roofing and waterproofing | 5 | 5 | |
| Slab | 81 | 81 | 6.5" NWC slab |
| Beams/girders | 0 | 0 | No beams below concrete slab |
| MEP | 3 | 3 | MEP hung from underside of roof slab |
| Lighting and misc. | 2 | 2 | Lighting and misc. hung from underside of roof slab |
| Columns | 0 | 0 | |
| Partitions | 0 | 0 | |
| Total | 91 | 91 | |

1 - The flat load is a reinforced concrete slab assembly that is located above the stair tower roof, 8'-6" above roof between Grids B-C/1-3.

2 - Per Det. AS3 & CS3, the concrete slab is 6.5" thick.

3 - The concrete slab is directly supported by exterior concrete walls. No columns extend to the roof.

| STAIR TOWER FLOOR | Seismic Weight psf | Dead Load psf | Remarks |
|---|---|---|---|
| Slab | 88 | 88 | 7" NWC slab |
| Beams/girders | 0 | 0 | No beams below concrete 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 | 0 | 0 | No columns supporting slab |
| Partitions | 0 | 0 | |
| Total | 93 | 93 | |

1 - The flat load is a reinforced concrete slab assembly that is located at the roof and second floor between Grids B-C/1-3.

2 - Per Det. AS3 & CS3, the concrete slab is 7" thick.

3 - The concrete slab is directly supported by exterior concrete walls. No columns extend to the roof.

| HIGH ROOF | Seismic Weight psf | Dead Load psf | Remarks |
|---|---|---|---|
| Mechanical equipment | 5 | 10 | Rooftop equipment consists of duct work |
| Roofing, waterproofing, and insulation | 10 | 10 | |
| Slab | 75 | 75 | 6" NWC slab |
| Beams/girders | 24 | 24 | Concrete beams below slab |
| MEP | 5 | 5 | MEP hung from underside of roof slab |
| Ceiling, lighting and misc. | 2 | 2 | Lay-in ceiling, lighting, and misc. hung from underside of floor slab |
| Columns | 6 | 0 | Reinforced concrete columns |
| Partitions | 8.5 | 0 | Includes hollow clay tile partition along one side of the corridor |
| Total | 136 | 126 | |

1 - The flat load is a reinforced concrete slab assembly that is located at the high roof between Grids A-D/3-5.

2 - The equipment is assumed to weigh 10 psf where it is located. The equipment is located on approximately 1/2 of the roof area and therefore, 5 psf is assumed for seismic mass.

3 - The slab thickness is inferred from Det. CS1, DS1, and CS4 in 1962 drawings.

4 - The concrete beam dimensions are unknown. Their geometry is inferred from elevations on A-9 in the 1962 drawings and plan views in the 1991 structural drawings.

5 - The flat load includes weight of (12) 20" square concrete columns below roof in a 5,298 ft $^2$ area. Column trib. height is 6'-3".

6 - Hollow clay tile partitions are located on one side of the corridor in the south wing. It is assumed to weigh 25 psf on its vertical face and its total weight is smeared over the plan area of the north wing floor. This weight is added to a typical interior partioins weight of 10 psf.

| LOW ROOF | Seismic Weight psf | Dead Load psf | Remarks |
|---|---|---|---|
| Mechanical equipment | 5 | 10 | Rooftop equipment consists of duct work |
| Roofing | 2 | 2 | Built-up roofing system, 3-ply and smooth-surfaced assumed |
| Waterproofing and insulation | 2 | 2 | 2" batt insulation and waterproofing membrane assumed |
| Sheathing | 4 | 4 | 3/8" plywood over 1" sheathing |
| Wood framing | 4 | 4 | Wood trusses below sheathing |
| MEP | 5 | 5 | MEP hung from underside of roof slab |
| Ceiling, lighting and misc. | 2 | 2 | Lay-in ceiling, lighting, and misc. hung from underside of floor slab |
| Columns | 0 | 0 | |
| Partitions | 5 | 0 | |
| Total | 29 | 29 | |

1 - The flat load is a wood-framed assembly supported by wood trusses that takes place at the low roof between Grids D-E/2-4.

2 - The equipment is assumed to weigh 10 psf where it is located. The equipment is located on approximately 1/2 of the room area and therefore, 5 psf is assumed for seismic mass.

3 - In the 1962 alterations, 3/8" plywood sheathing was placed over existing 1" sheathing.

4 - The wood framing consists of wood trusses at ±3'-0" o.c. that span in the east-west direction as specified on Det. CS5 & DS5 in the 1962 drawings.

5 - There is no evidence of an adequate connection of the interior masonry walls to the low roof. Therefore, the CMU partition walls are not included as seismic mass tributary to the low roof.

6 - The low roof is directly supported by exterior walls. No columns extend to the roof.

| | Seismic Weight | Dead Load | |
|---|---|---|---|
| **SECOND FLOOR** | **psf** | **psf** | **Remarks** |
| Flooring | 5 | 5 | Vinyl asbestos tiling (VAT) |
| Slab | 81 | 81 | 6.5" NWC slab |
| Beams/girders | 35 | 35 | Concrete beams below slab |
| MEP | 10 | 10 | MEP hung from underside of roof slab |
| Ceiling, lighting and misc. | 2 | 2 | Lay-in ceiling, lighting, and misc. hung from underside of floor slab |
| Columns | 13 | 0 | Reinforced concrete columns |
| Laboratory furniture | 15 | 15 | |
| Partitions | 17 | 17 | Includes hollow clay tile partition along one side of the corridor |
| Total | 178 | 165 | |

1 - The flat load is a reinforced concrete slab assembly located at the second floor between Grids A-D/3-5.

2 - The slab thickness is inferred from Det. CS1, DS1, and CS4 in the 1962 drawings.

3 - The flat load includes weight of (12) 20" square concrete columns above below second floor in a 5,298 $ft^2$ area. Column trib. height is 12'-9.5".

4 - The concrete beam dimensions are unknown. Their geometry is inferred from elevations on A-9 in the 1962 drawings and plan views and Det. 18 in the 1991 structural drawings.

5 - The building contains fixed lab benches at the second floor.

6 - Hollow clay tile partitions are located on one side of the corridor in the south wing. It is assumed to weigh 25 psf on its vertical face, and its total weight is smeared over the plan area of the north wing floor. This weight is added to a typical interior portions weight of 10 psf.

## Story Weight

wglass = 15 psf

| | Floor Area (ft²)[1,2] | | | | | Floor Weight (psf) | | | | | Wall Weight [3,4,5] | | Glass Weight [6] | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Floor Levels | STAIR TOWER ROOF | STAIR TOWER FLOOR | HIGH ROOF | LOW ROOF | SECOND FLOOR | STAIR TOWER ROOF | STAIR TOWER FLOOR | HIGH ROOF | LOW ROOF | SECOND FLOOR | Height below floor level (ft) | Wall Seismic Weight (kips) | Length (ft) | Trib. Wall Height [above & below] (ft) | Glass Seismic Weight (kips) | Total Seismic Weight (kips) |
| | | | | | | | | | | | | | | | | |
| High Roof | 294 | 294 | 5,298 | 1,218 | 0 | 91 | 93 | 136 | 29 | 178 | 12.50 | 757 | 52 | 6.25 | 5 | 1,571 |
| Second Floor | 0 | 0 | 0 | 1,218 | 5,298 | 91 | 93 | 136 | 29 | 178 | 13.08 | 860 | 59 | 12.79 | 11 | 1,847 |
| First Floor | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |

Notes: **3,419** **kips**

1 - The seismic base is set at the first floor.

2 -The weight of the low roof is distributed equally between the second floor and the high roof.

3 - Wall weight includes the following contributions and is summarized in the following table:

- 3.1 - Reinforced concrete walls: Structural 8" thick elements constructed after the 1962 alterations enclosing the stair tower from the first floor up to the high roof and concrete wall between First and Second floor on Line B.5 which construction year is unknown
- 3.2 - Masonry walls: Original unreinforced masonry (URM) exterior walls, including concrete masonry unit (CMU) infill of openings after the 1962 alterations.
- 3.3 - Gunite walls: 3" and 4" thick gunite walls on interior or exterior face of perimeter masonry walls added during the 1962 renovation
- 3.4 - Stair tower above roof: 8" thick walls extending 11'-0" from the high roof to the stair tower roof.
- 3.5 - Parapet: masonry and gunite parapets extending above the low and high roof.

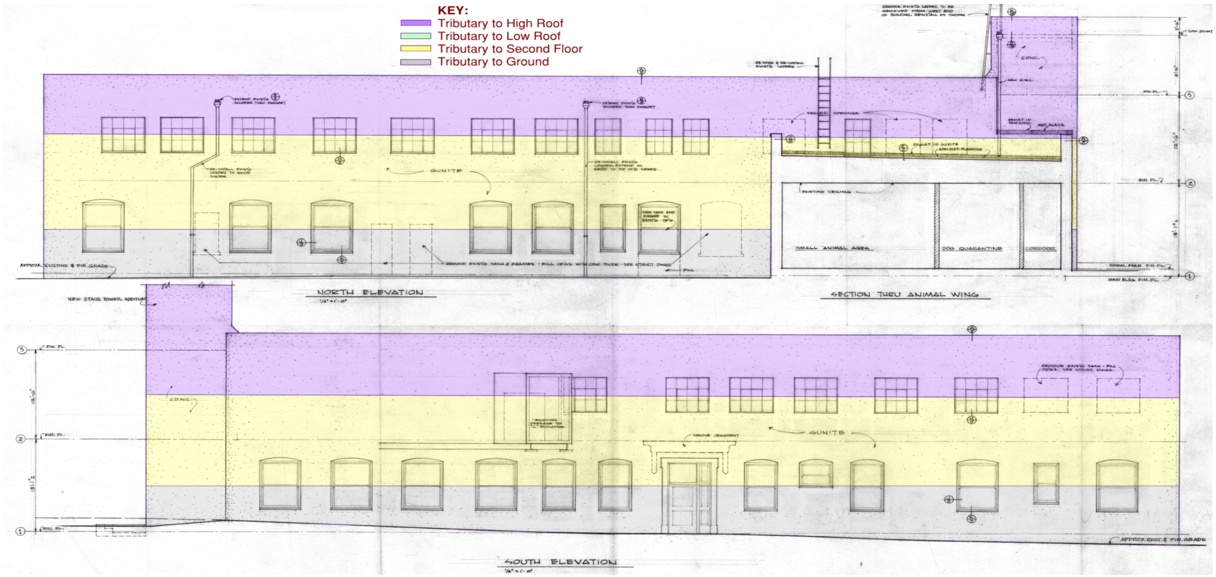
| Level | Structural Wall Weight (kips) | | | Additional Weight (kips) | | Wall Seismic Weight (kips) |
|---|---|---|---|---|---|---|
| | Reinforced Concrete | Masonry | Gunite | Stair Tower above high roof | Parapet | |
| High Roof | 30 | 216 | 83 | 88 | 159 | 577 |
| Low Roof | 0 | 198 | 62 | 0 | 101 | 361 |
| Second Floor | 75 | 436 | 168 | 0 | 0 | 679 |

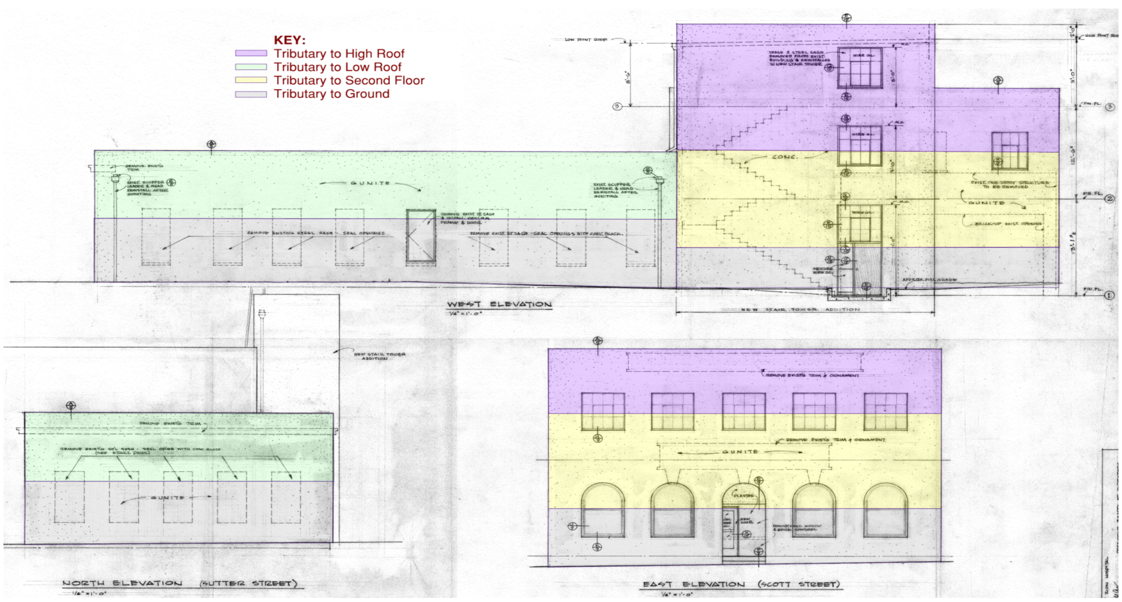
4 - There is no evidence of an adequate connection to transfer loads from the low roof diaphragm to the interior masonry walls; thus, the CMU partition walls on the north wing of the building are not considered for the shear wall stress check

5 - Out-of-plane bracing of the wall with the diaphragms determines the tributary height at each level. Exterior wall elevations with color-coded tributary areas are shown in the next page. Similar to the floor area, wall weight tributary to low roof is distributed equally between the second floor and the high roof.

6 - The glass weight includes area exterior windows with and assumed weight of 15 psf.

## Building "E" Exterior Wall Elevations



# Period

| | |
|---|---|
| $C_t$= | 0.02 |
| $h_n$ (ft)= | 25.58 |
| B= | 0.75 |

| | | |
|---|---|---|
| T= | 0.23 | 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 first floor to the high 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;

$\beta$ = 0.80 for moment-resisting frame systems of steel (Building Types S1 and S1a);

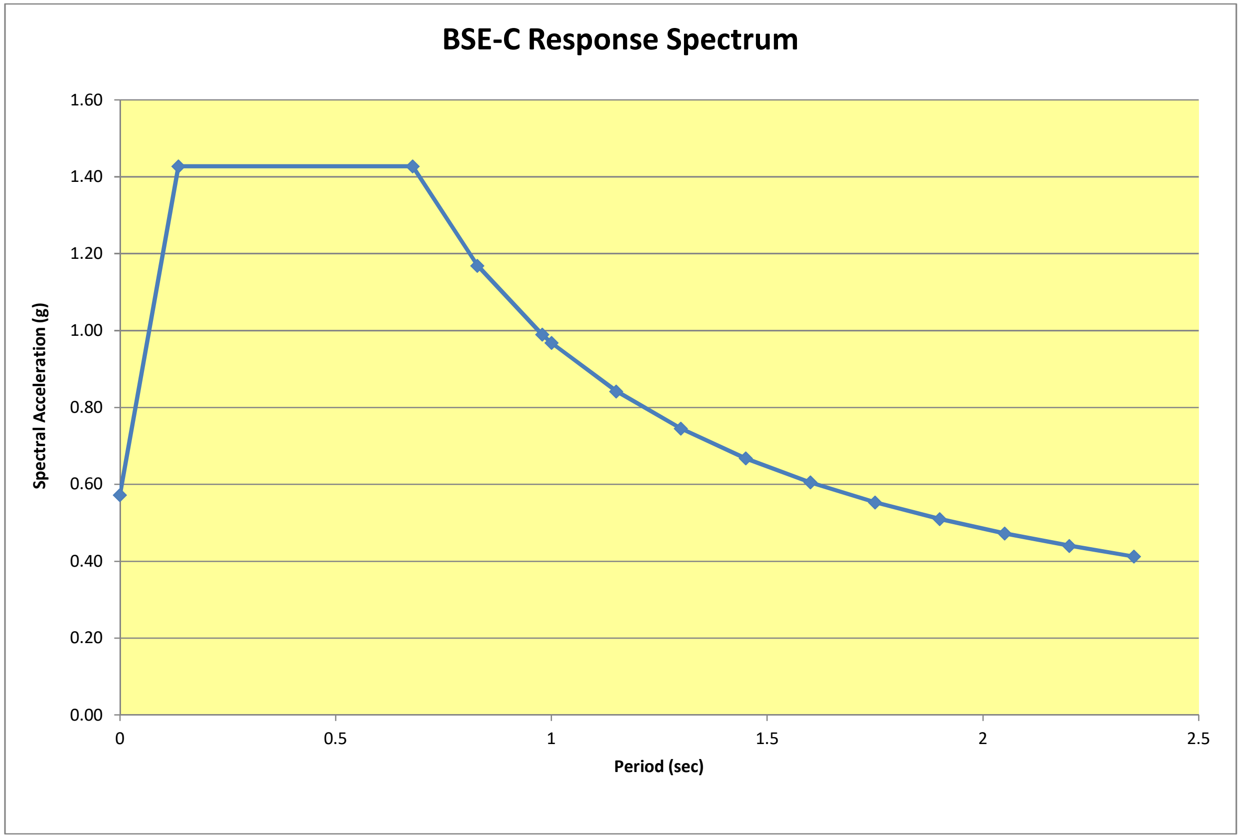
= 0.90 for moment-resisting frame systems of reinforced concrete (Building Type C1); and

= 0.75 for all other framing systems.

# Site Parameters

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

| | | |
|---|---|---|
| BSE-C | | |
| $\beta$ = | 0.05 | |
| $B_1$ = | 1.00 | |
| $S_S$ = | 1.431 | g |
| $S_1$ = | 0.557 | g |
| $F_a$ = | 1.000 | g |
| $F_v$ = | 1.743 | g |
| Site Class = | D | |
| $S_{CS}$ = | 1.431 | g |
| $S_{C1}$ = | 0.971 | g |
| $T_0$ = | 0.14 | s |
| $T_s$ = | 0.68 | s |
| T = | 0.23 | s |
| $S_a$ = | 1.43 | g (See Note 2) |
| **Tier 1 $S_a$ =** | **1.43** | **g** (See Note 3) |




Notes:

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

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

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

# Seismic Force Distribution

| Horizontal Response Spectrum Seismic Parameters | | |
|---|---|---|
| Hazard Level | BSE-C | |
| Site Class | D | |
| $S_{CS}$= | 1.431 g | (See Note 2) |
| $S_{C1}$= | 0.971 g | (See Note 2) |

| | | | |
|---|---|---|---|
| T= | 0.23 | s | |
| Sa= | 1.43 | g | (See Note 3) |
| W= | 3,419 | kips | |
| C= | 1.2 | Per ASCE 41-17 Table 4-7 | |

| | | |
|---|---|---|
| V= | 5,870 | 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 (ft) | Total Height, H (ft) | Weight, W (kips) | W x $H^k$ | coeff | Fx (kips) | Story Shear, V (kips) |
|---|---|---|---|---|---|---|---|
| | | | | | | | |
| | | | | | | | |
| High Roof | 12.50 | 25.58 | 1,571 | 40,196 | 0.62 | 3,666 | 3,666 |
| Second Floor | 13.08 | 13.08 | 1,847 | 24,170 | 0.38 | 2,204 | 5,870 |
| First Floor | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| Σ = | 25.6 | | 3,419 | 64,366 | 1 | 5,870 | |

Notes:

1- The seismic base of building is set at the first 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)<br>Moment frame (S1, S3, C1, PC2a) | 1.3 | 1.1 | 1.0 | 1.0 |
| Shear wall (S4, S5, C2, C3, PC1a, PC2, RM2, URMa)<br>Braced frame (S2)<br>Cold-formed steel strap-brace wall (CFS2) | 1.4 | 1.2 | 1.1 | 1.0 |
| Unreinforced masonry (URM)<br>Flexible diaphragms (S1a, S2a, S5a, C2a, C3a, PC1, RM1) | 1.0 | 1.0 | 1.0 | 1.0 |

[a] Defined in Table 3-1.

5- The two-story portion of the structure contains concrete diaphragms, a infilled concrete frame, with a gunite retrofit on URM walls. This is a combination of building type C2 and C3. The one-story building contains a flexible wood diaphragm with a gunite retrofit on URM walls. This is similar to buidlnig type C2a and URM.

# Average Wall Stress Check: Concrete and URM

**Average Stresses**

***Clay URM***

Ms = 1.75

***Reinforced Concrete and Gunite***

Ms = 4.5

f'c = 2500 psi (Assumed per building vintage, ASCE 41-17. See Note 3)

Em = 420 psi (See Note 5)

Ec = 3031 psi (See Note 7)

Ec/Em = 7.2

| Longitudinal (E-W direction) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall Areas ($in^2$) | | | Shear Force Distribution (kips) | | Masonry Walls Check | | | Reinforced Concrete and Gunite Walls Check | | |
| Story | Story Shear | Reinforced Concrete Wall Area | Masonry Wall Area | Gunite Wall Area | Shear Force in Masonry Walls | Shear Force in RC and Gunite Walls | Average Shear Stress, URM Walls (psi) | Tier 1 Shear Stress Limit for Clay Masonry Walls (psi) | Masonry Walls OK? | Average Shear Stress, RC and Gunite Walls (psi) | Tier 1 Shear Stress Limit, RC and Gunite Walls (psi) | RC and Gunite Walls OK? |
| High Roof - Second Floor | 3,666 | 1,800 | 16,760 | 6,446 | 806 | 2860 | 27 | 30 | OK | 77 | 100 | OK |
| Second Floor - First Floor | 5,870 | 5,953 | 40,685 | 14,888 | 1,250 | 4621 | 18 | 30 | OK | 49 | 100 | OK |

| Transverse (N-S direction) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall Areas ($in^2$) | | | Shear Force Distribution (kips) | | Masonry Walls Check | | | Reinforced Concrete and Gunite Walls Check | | |
| Story | Story Shear | Reinforced Concrete Wall Area | Masonry Wall Area | Gunite Wall Area | Shear Force in Masonry Walls | Shear Force in RC and Gunite Walls | Average Shear Stress, URM Walls (psi) | Tier 1 Shear Stress Limit for Clay Masonry Walls (psi) | Masonry Walls OK? | Average Shear Stress, RC and Gunite Walls (psi) | Tier 1 Shear Stress Limit, RC and Gunite Walls (psi) | RC and Gunite Walls OK? |
| High Roof - Second Floor | 3,666 | 2,228 | 6,523 | 2,509 | 587 | 3079 | 51 | 30 | NG | 144 | 100 | NG |
| Second Floor - First Floor | 5,870 | 4,559 | 35,574 | 12,411 | 1,321 | 4549 | 21 | 30 | OK | 60 | 100 | OK |

Notes:

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

2 - The shear stress in shear walls is based upon ASCE 41-17, Equation 4-8. The respective Ms factors for each material are used 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 - The reinforced concrete and gunite wall compressive strength is not specified in available drawings. Wall compressive strength of 2.5 ksi is assumed per ASCE 41-17, Table 4-2.

**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 force distribution between masonry and shotcrete is based upon relative rigidity , Wall Area x Shear modulus = A x 0.4x E.

5 - Em = 700 f'm per ASCE 41-17 Section 11.2.3.4 & TMS 402-16 Table 4.2.2.

6- The unreinforced masonry compressive strenght is f'm = 600 psi for solid clay units per ASCE 41-17, Table 11-2a.

**Table 11-2a. Default Lower-Bound Unreinforced Masonry Strengths**

| Material | Solid Units | Hollow Concrete Units |
|---|---|---|
| Compressive strength[a] | 600 lb/in.$^2$ | 1,000 lb/in.$^2$ |
| Flexural tensile strength[b] | 60 lb/in.$^2$ | 38 lb/in.$^{2c}$ (95 lb/in.$^2$)[d] |
| Shear strength | [e] | [e] |

7 - The elastic modulus of concrete is defined as Ec = $wc^{1.5}$ x 33 x sqrt(f'c) per ACI 318-14 Section 19.2.2.

# Average Wall Stress Check: Concrete Walls Only

**Average Stresses**

***Reinforced Concrete and Gunite***

Ms = 4.5

f'c = 2500 psi (Assumed per building vintage, ASCE 41-17. See Note 3)

| Longitudinal (E-W direction) | | | | | | | |
|---|---|---|---|---|---|---|---|
| | | Wall Areas (in$^2$) | | | Reinforced Concrete and Gunite Walls Check | | |
| Story | Story Shear | Reinforced Concrete Wall Area | Gunite Wall Area | Total Concrete Wall Area | Average Shear Stress, RC and Gunite Walls (psi) | Tier 1 Shear Stress Limit, RC and Gunite Walls (psi) | RC and Gunite Walls OK? |
| High Roof - Second Floor | 3,666 | 1,800 | 6,446 | 8,246 | 99 | 100 | OK |
| Second Floor - First Floor | 5,870 | 5,953 | 14,888 | 20,841 | 63 | 100 | OK |

| Transverse (N-S direction) | | | | | | | |
|---|---|---|---|---|---|---|---|
| | | Wall Areas (in$^2$) | | | Reinforced Concrete and Gunite Walls Check | | |
| Story | Story Shear | Reinforced Concrete Wall Area | Gunite Wall Area | Total Concrete Wall Area | Average Shear Stress, RC and Gunite Walls (psi) | Tier 1 Shear Stress Limit, RC and Gunite Walls (psi) | RC and Gunite Walls OK? |
| High Roof - Second Floor | 3,666 | 2,228 | 2,509 | 4,736 | 172 | 100 | NG |
| Second Floor - First Floor | 5,870 | 4,559 | 12,411 | 16,970 | 77 | 100 | OK |

Notes:

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

2 - The shear stress in shear walls is based upon ASCE 41-17, Equation 4-8. The respective Ms factors for each material are used per ASCE 41-17 Table 4-8.

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| | 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 - The reinforced concrete and gunite wall compressive strength is not specified in available drawings. Wall compressive strength of 2.5 ksi is assumed per ASCE 41-17, Table 4-2.

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| 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 |