UCSF

Rating form completed by:

RUTHERFORD + CHEKENE
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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

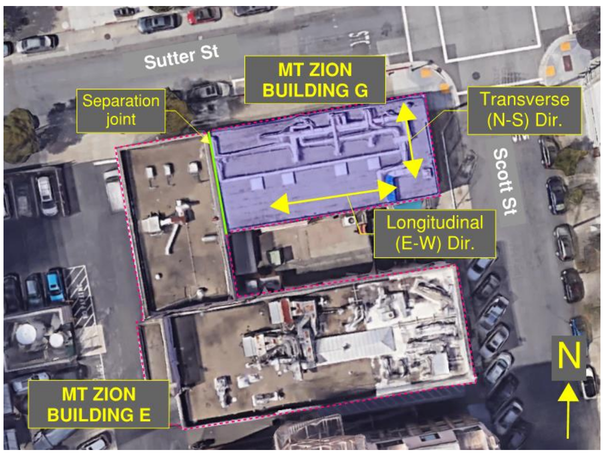
Mount Zion Building G

CAAN #2026
1675 Scott Street, San Francisco, CA 94115
UCSF Campus: Mount Zion

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

10-10-19

Plan



East elevation (looking west)

Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	IV	Findings based on drawing review and ASCE 41-17 Tier 1 evaluation[1]
Rating basis	Tier 1	ASCE 41-17
Date of rating	2019	
Recommended UCSF priority category for retrofit	None	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application for modification
Ballpark total project cost to retrofit to IV rating	N/A	See recommendations on further evaluation and retrofit
Is 2018-2019 rating required by UCOP?	Yes	Does not have a documented previous review
Further evaluation recommended?	No	

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

Building information used in this evaluation

- Architectural drawings by Schubart and Friedman Architects, “New Maintenance Shop for Mt. Zion Hospital and Medical Center, San Francisco, California,” dated 20 October 1961, Sheets A-1 to A-4.
- Structural drawings by I. Thompson Structural Engineer, “New Maintenance Shop for Mt. Zion Hospital and Medical Center, San Francisco, California,” dated 20 October 1961, Sheets 1 to 3.
- Architectural drawings by Howard A. Friedman and Associates Architects and Planners, “Day Care Center for Mt. Zion Hospital and Medical Center,” dated 20 March 1978, Sheets A1 to A4.
- Structural drawings by I. Thompson and Associates, “Day Care Center for Mt. Zion Hospital and Medical Center,” dated 20 March 1978, Sheets S1 to S3.
- Architectural drawings by W. Lee Pollard & Associates Architecture, “Dialysis Relocation Building G,” dated 23 October 1992, Sheets A0.1, A1.1 to A1.3, A2.1, A2.2, A3.1, A3.2, A5.1 to A5.3, A6.1, A6.2, A7.1, A9.1 to A9.4.
- Structural drawings by Rudolf Fehr Consulting Structural Engineer, “Dialysis Relocation Building G,” dated 23 October 1992, Sheets S-1 to S-5.

Additional building information known to exist

None

Scope for completing this form

The architectural and structural drawings for the original 1961 construction and the subsequent 1978 and 1992 renovations are 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 G is a patient dialysis clinic located at the corner of Scott Street and Sutter Street in San Francisco, California on the UCSF Mt. Zion campus. It is a one-story reinforced masonry structure that was constructed in 1961. It contains a rectangular floor plan that measures 87’-6” in the east-west direction by approximately 40’-0” in the north-south direction. It was constructed on a primarily flat site and is adjacent to Building E which was existing at the time of construction. On its first floor, Building G originally housed a machine, paint, and carpentry shop. On its second floor, it contained a 17’-0” wide mezzanine located along its east elevation. In 1978, Building G was converted to a geriatric day care center. At that time, a new 12’-8” wide mezzanine was added to the structure along its north elevation. This space contained a restroom, a conference room, and office space. At that time, the existing east mezzanine was fully enclosed and converted into a mechanical room. In 1992, the structure was renovated a second time and converted to a patient dialysis clinic. Patient care is located on the first floor while the north mezzanine is utilized as open office space, and the east mezzanine remained as a mechanical room. The clinic offers extended hours to patients and is open 6 days a week from 5:00 am to 9:00 pm. During these hours, there are approximately 10 employees and 25 patients inside the building.

<u>Identification of levels</u>: The building levels are designated as the first floor (EL. 134.00), the second floor (EL. 141.75 for north mezzanine and EL. 143 ft for east mezzanine), and the roof (EL. 149.54 ft at the high point and EL. 149.45 ft at the low point). The exterior grade is relatively flat with a low point at the southeast corner of the structure and a high point at the northwest corner of the structure. An entry ramp is located at the main entrance at the southeast corner of the building.

<u>Foundation system</u>: The slab-on-grade is comprised of a 5” thick concrete slab that is reinforced with #3 bars spaced at 15” each way. Perimeter masonry walls are supported by concrete grade beams that are 8 ½”, 9”, and 9 ¾” wide by 2’-0” deep. The beams are centered below the walls and contain 4-#6 longitudinal bars at the top and bottom with #3 ties spaced at 16” o.c. The grade beams span to reinforced concrete piers that are 20” in diameter and range in depth from 14 ft to 23 ft. They are reinforced with 4-#6 longitudinal bars and #3 ties spaced at 18” o.c. The piers are typically centered below the walls, except along the east and north elevation where they are set back by 1’-9” and 1’-0” from the outside face of wall respectively. Along these elevations, the exterior walls are supported by perpendicular grade beams which cantilever from the setback piers. This foundation configuration was likely utilized in order to avoid conflict with the adjacent Building E foundations.

In 1978, additional 20” diameter concrete piers were installed below the steel posts that were added to support the north mezzanine. The piers range in depth from 10 ft to 15 ft, and the reinforcing matched the detail originally used in 1961.

In 1992, 15” diameter by 15 ft deep concrete piers were added below a new shear wall located in the center of the structure. Grade beams that measure 12” wide by 1’-6” deep were also added between the existing piers that support the north mezzanine.

Structural system for vertical (gravity) load: Building G contains gravity load-bearing concrete masonry walls around its perimeter on four sides. They are comprised of 4” x 16” stacked bond units. The vertical wall reinforcing consists of a single layer of #5 bars spaced at 16” o.c. while the horizontal reinforcing consists of 2-#3 bars spaced at 24” o.c. A reinforced concrete beam that measures 9 ¼” x 1’- 2 3/4” sits on top of the masonry walls. Two interior load-bearing masonry walls are oriented in the north-south direction and support portions of the mezzanine slabs.

The roof framing consists of 2” unfilled metal deck that spans between 10B11.5 steel beams. The deck profile appears similar to N-deck, and the gage is unknown. It is supported at the perimeter by a 3”x 2” steel ledger angle that is bolted into the face of the concrete beam with 5/8” diameter bolts spaced at 2’-8” o.c. The deck profile and the connection of the deck to the steel framing are unknown as the available drawings refer to the metal deck specifications which are not currently available for review. The 10B11.5 steel beams are oriented in the north-south direction and form diaphragm crossties in the transverse direction. They are spaced between 6’-3” to 6’-8” o.c. and span approximately 20’-0” from the exterior walls to 16W36 girders oriented in the east-west direction. The girders are located along the center longitudinal axis of the structure at the roof high point. They span between the exterior masonry walls and an interior shear wall.

The east mezzanine slab from the 1961 original construction is an 8” thick concrete slab that is reinforced with #4 bars spaced at 16” o.c. at the top and bottom in both directions. It is supported by walls around its perimeter as well as one central interior wall that is oriented in the east-west direction. The masonry wall construction typically stops at the underside of the slab and restarts at the top of the slab. As such, the slab bears on the wall and is connected to the wall with #5 vertical dowels spaced at 16” o.c. which run through the slab from the wall below and into the wall above.

The north mezzanine slab was added to Building G in 1978. It is comprised of a 6 1/2” thick reinforced concrete slab that contains #5 bars at 16” o.c. at the bottom. It spans in the north-south direction and is supported on its southern edge by a row of 2 ½” x 2 ½” steel posts. On its northern edge, it is supported by the existing masonry wall, and it is connected to this wall by ¾” diameter “Parabolts” spaced at 12” o.c. that are located at the mid-depth of the slab. The Parabolt is a proprietary masonry insert that was used during construction at that time. A wall anchor is inserted into the existing wall, and an accompanying threaded rod was nested into the anchor. During the 1992 renovation, a ledger angle was added to further reinforce the connection of the mezzanine to the wall. This 4”x 4” x 3/8” steel angle is bolted to the underside of the slab with ½” diameter bolts spaced at 12” o.c. and is bolted to the wall with 5/8” diameter bolts spaced at 16” o.c. This angle was observed in the field.

Structural system for lateral forces: The lateral force-resisting system is comprised of bearing reinforced concrete masonry shear walls located around the building perimeter and at the interior. In the transverse direction, three walls resist forces at the roof level, and four walls resist forces at the second floor below the mezzanine slabs. The walls are well spaced apart and limit the span of the diaphragm to a maximum of 38’-6”. In the longitudinal direction, two walls resist forces at the roof level, and three walls resist forces at the second floor. In this direction, the diaphragm spans approximately 40’-0”. The original walls contained a number of window and door openings. During both the 1978 and 1992 renovation, these openings were reconfigured with new openings added and existing openings infilled. In 1992, a new centrally located reinforced concrete shear wall was built in the transverse direction. It is 16” thick and reinforced with #5 bars spaced at 12” o.c. on each face in the vertical direction. The horizontal reinforcing consists of #4 bars spaced at 12” o.c. on each face. At the wall ends, these bars are hooked around 3-#6 vertical boundary bars with 90-degree hooks. The wall is connected to the roof with a steel truss constructed from WT 3x10 chord members and 3” x 2” rectangular tube diagonal members. At the underside of the roof framing, steel channels were added to both sides of an existing reinforced concrete beam and this built-up beam assembly serves as a collector element to deliver load from the roof diaphragm to the vertical truss and down into the wall.

The roof diaphragm consists of unfilled metal deck, and its connection to the roof framing is unknown. An in-plane steel truss that serves as diaphragm bracing is located in the center of the roof and spans in the east-west direction. The truss is 6'-3" deep and is comprised of diagonal 5B5.75 members. The east and west walls are braced out-of-plane by the flutes of the metal deck which are oriented perpendicular to these walls. The deck is connected to the walls with a steel ledger angle. The walls are braced out-of-plane in the north-south direction by the steel roof framing. These beams bear on an 8" x 4" steel angle that is connected to the wall with 4- 5/8" diameter bolts. The angle is then connected to the beam bottom flange with 2-5/8" diameter bolts.

Building condition: Good. No on-going maintenance problems were noted by the building engineer.

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 and potential seismic deficiencies of the building include the following:

- The connection of the metal deck to the steel framing is unknown. The metal deck specifications are referenced on the structural drawings, however; these documents are not currently available for review. Given the building vintage and the attention to detail present in the available construction documents, it is assumed that a nominal connection of the metal deck to the steel roof framing was provided.
- The mezzanine diaphragm consists of split levels as the east and north slabs are located at different elevations.
- The north mezzanine is connected to walls on three sides and does not contain lateral support along its southern edge.
- The foundation piers are typically connected together with grade beams in one direction only. The slab-on-grade may act as foundation ties.

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

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

There are two nonstructural components of interest in this structure. The first is a small canvas sheathed canopy that was added above the main entrance in 1992. It is supported by tube steel framing that spans between a steel post and the exterior wall of the structure. The second is a 4'-4" wide soffit was constructed along the south interior wall to offer privacy over the patient vestibules. It is framed with metal stud framing clad with gypsum board on all sides. The anchorage of these items is beyond the scope of the Tier 1 assessment. However, it is noted that they are

[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.

both potential falling hazards as the canopy is located over the main egress and the soffit is located over the patient vestibules.

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.	The building engineer notes that natural gas is not supplied to Building G. It is however, supplied to the vacant adjacent structure, Building E.

Basis of Seismic Performance Level rating

Building G is a rectangular structure with a plan aspect ratio of approximately 1W:2L. The walls are optimally located around its entire perimeter and are well-spaced on the interior. The structure is regular, located on a flat site, and does not contain discontinuous shear walls or geometric irregularities. The number of walls in each direction increase from the roof down to the first floor. The overturning forces are likely low given the shear wall aspect ratio of 1V:2.5H in the transverse direction and 1V:5.7H in the longitudinal direction.

Building G was constructed in close proximity to the adjacent Building E which is located on its west elevation. The two structures are separated by a 3” wide gap which is larger than the 2.8” gap required by the Tier 1 assessment.

In the longitudinal direction, the wall stresses are 12 psi between the roof to second floor and 37 psi the second floor to first floor. In the transverse direction, the walls stresses are 29 psi between the roof to second floor and 61 psi between the second floor to first floor. These stresses are below the Tier 1 acceptance limit of 70 psi.

The roof contains a flexible metal deck diaphragm that contains steel cross bracing in the direction parallel to the deck flutes. The connection of the metal deck to the steel framing is unknown as this detail references the specifications which are not currently available for review. For this assessment, it is assumed that a nominal connection, such as puddle welding of the deck down flutes, was provided. The second floor is comprised of two reinforced concrete mezzanine slabs. These slabs are located at different elevations and are therefore not likely to share load. However, each slab is laterally braced and connected to shear walls on at least three sides.

The steel bolts in the anchorage connections at the roof and second floor slabs were checked for out-of-plane forces and were found to be adequate.

The building is assigned a Seismic Performance Level Rating of IV because the structure does not contain any significant deficiencies. Diaphragm spans and aspect ratios are low. In addition, the walls are well configured with no significant openings or discontinuities, and the wall stresses are low.

Recommendations for further evaluation or retrofit

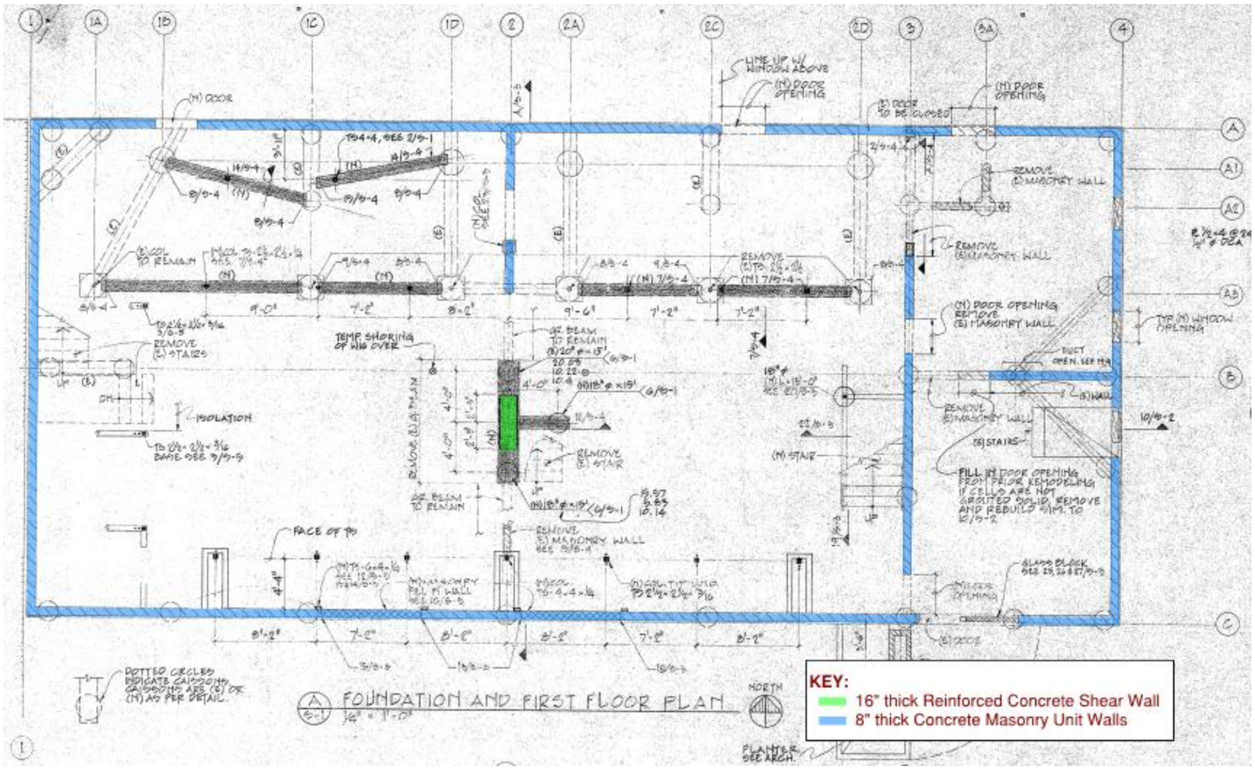
No additional assessment is required.

Peer review comments on rating

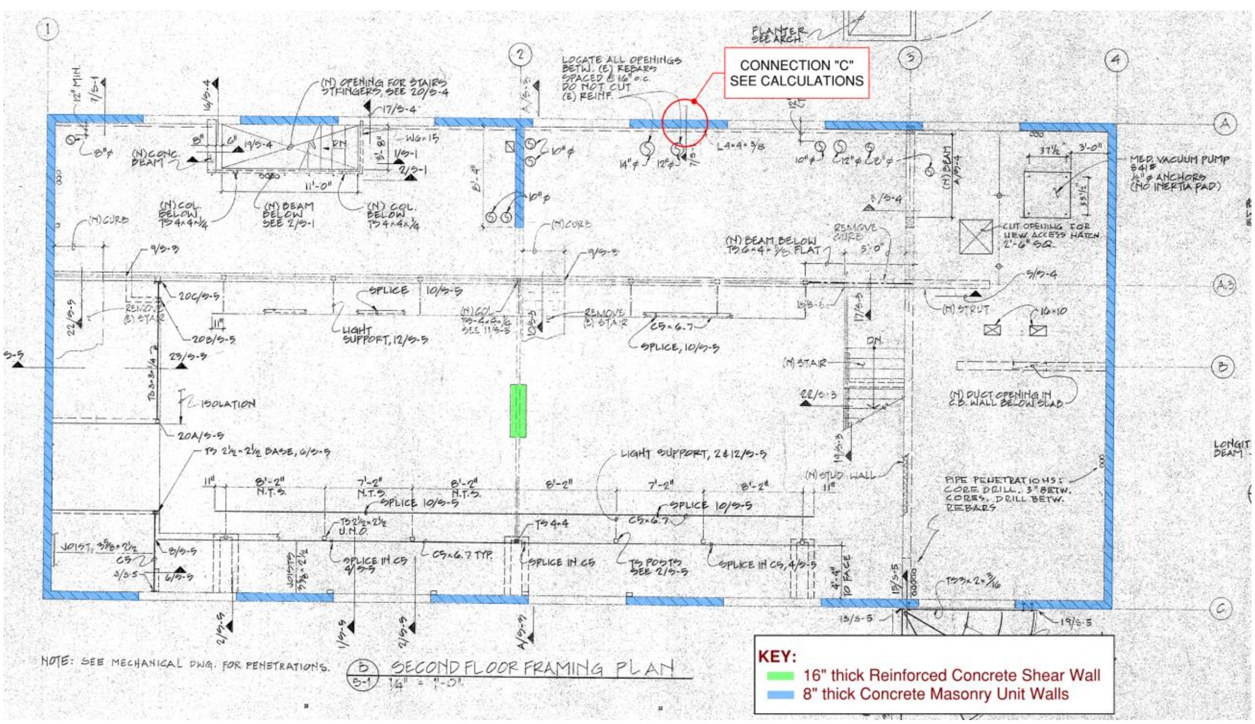
The structural members of the UCSF Seismic Review Committee (SRC) reviewed the evaluation on 10 October 2019 and were unanimous that the Seismic Performance Level Rating is Level IV. No additional assessment is required.

Additional building data	Entry	Notes
Latitude	37.78528	
Longitude	-122.43846	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	2	Office space area and mechanical room does not classify as story
Number of stories (basements) below lowest perimeter grade	0	
Building occupiable area (OGSF)	5,300	
Risk Category per 2016 CBC 1604.5	II	
Building structural height, h_n	15.5 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	0.16 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 = 550 kips, V base = 787 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	

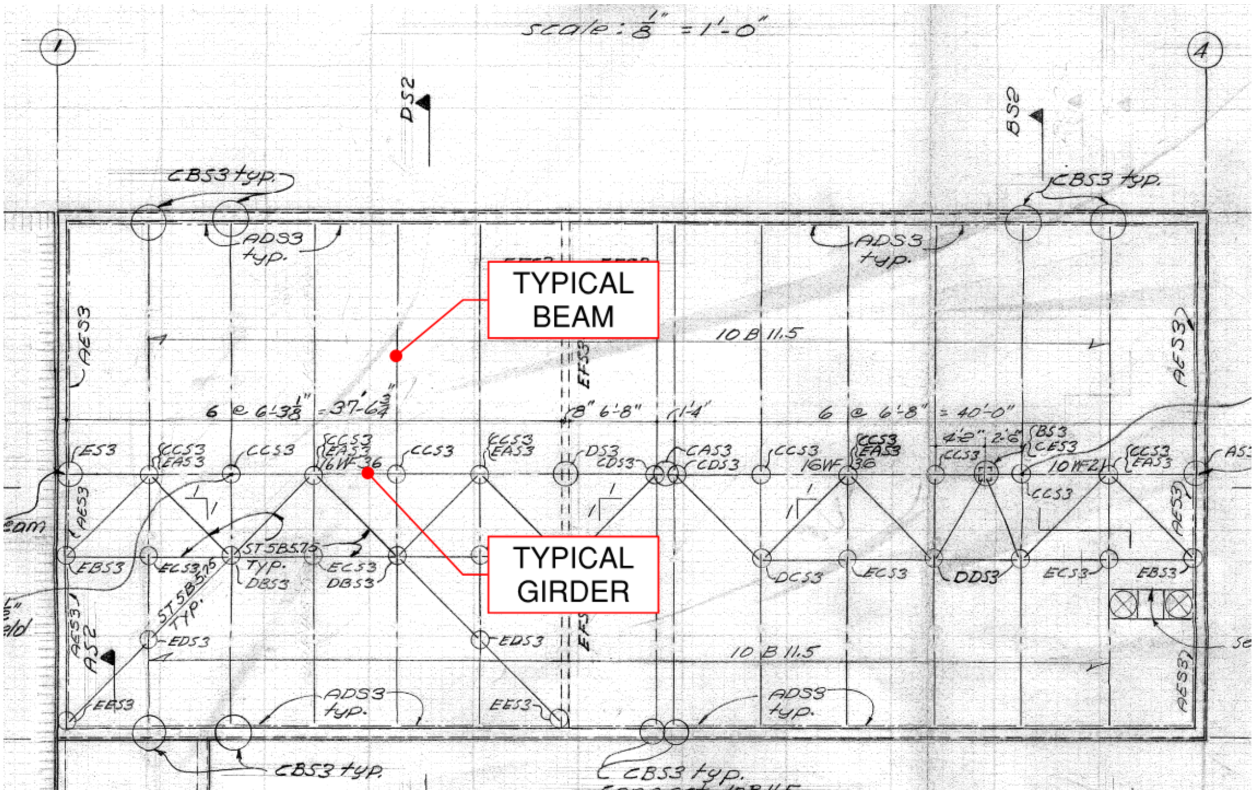
Applicable code		
Applicable code or approx. date of original construction	Built: 1962 Code: 1958 UBC assumed	Applicable code assumed
Applicable code for partial retrofit	Renovation drawings dated 1978 and 1992 Codes: 1976 and 1991 UBC are assumed	
Applicable code for full retrofit	None	No full retrofit known
Model building data		
Model building type north-south	RM1-RM2 Reinforced Masonry Bearing Walls w/ Flexible and Rigid Diaphragms	
Model building type east-west	RM1-RM2 Reinforced Masonry Bearing Walls w/ Flexible and Rigid Diaphragms	
FEMA P-154 score	N/A	Not applicable as an ASCE 41 Tier 1 evaluation was performed
Previous ratings		
Most recent rating	IV	
Date of most recent rating	2013	
2nd most recent rating	-	
Date of 2nd most recent rating	-	
3rd most recent rating	-	
Date of 3rd most recent rating	-	
Appendices		
ASCE 41 Tier 1 checklist included here?	Yes	Refer to attached checklist file



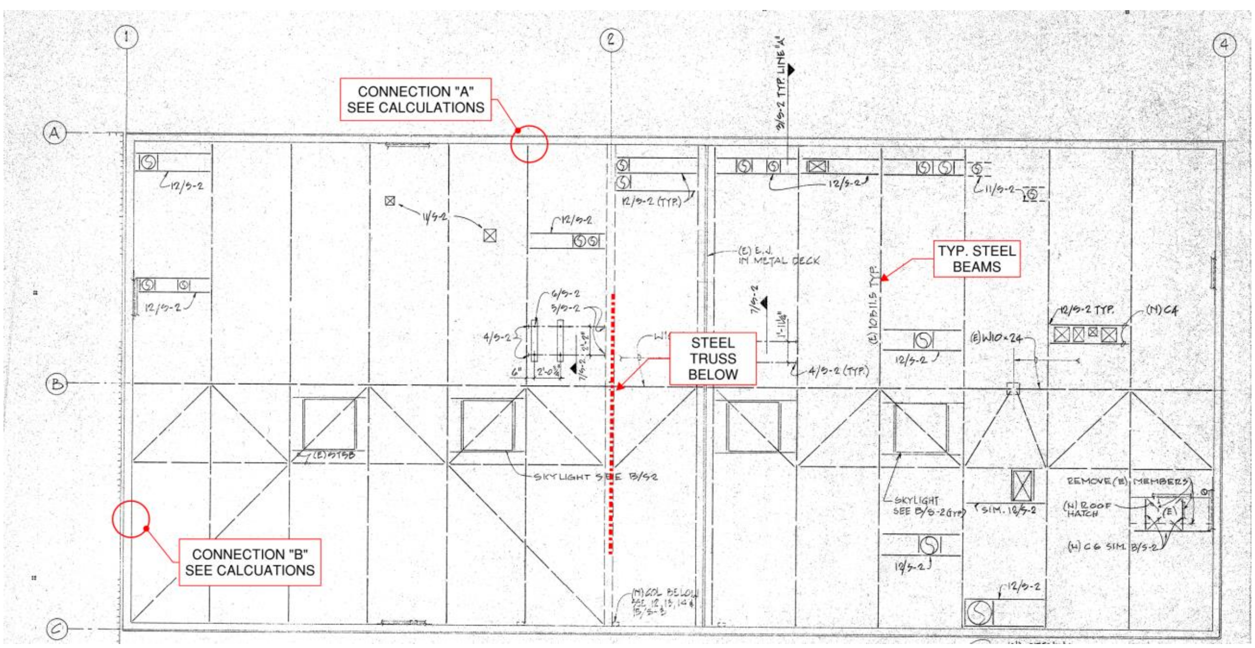
Lateral force-resisting system at the first floor



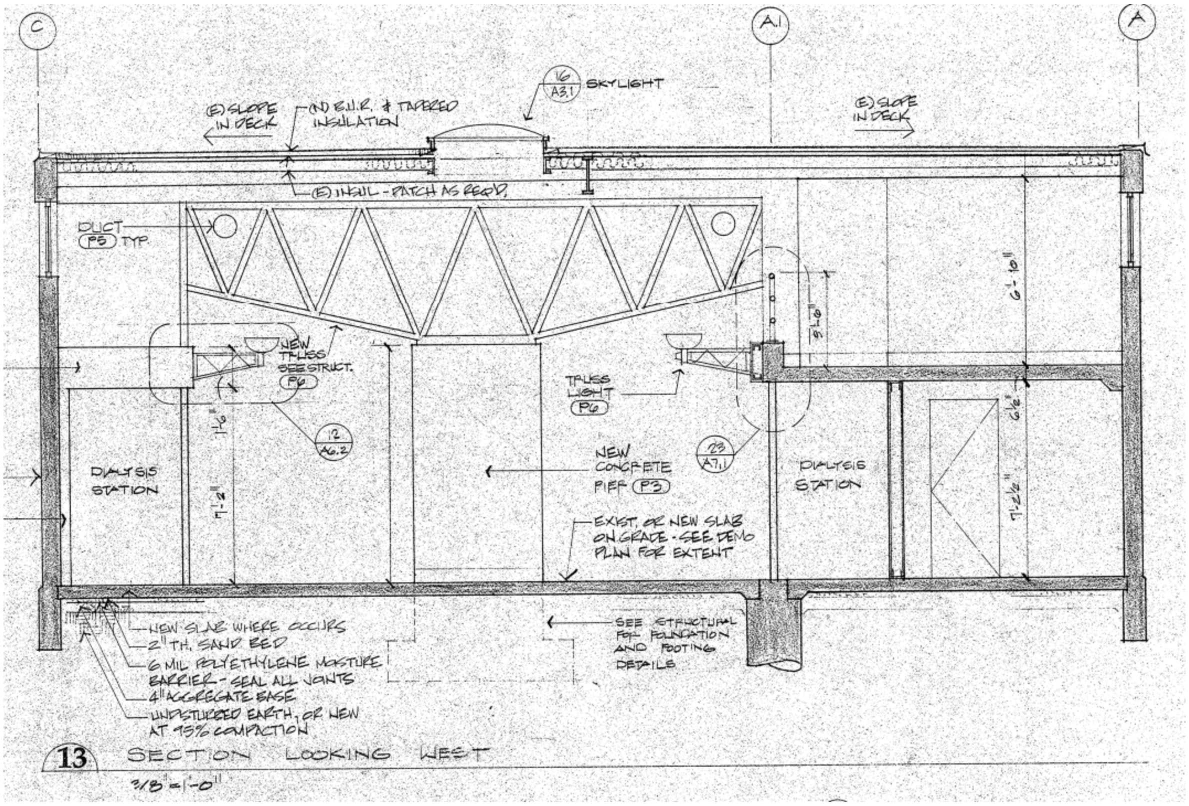
Lateral force-resisting system at the second floor



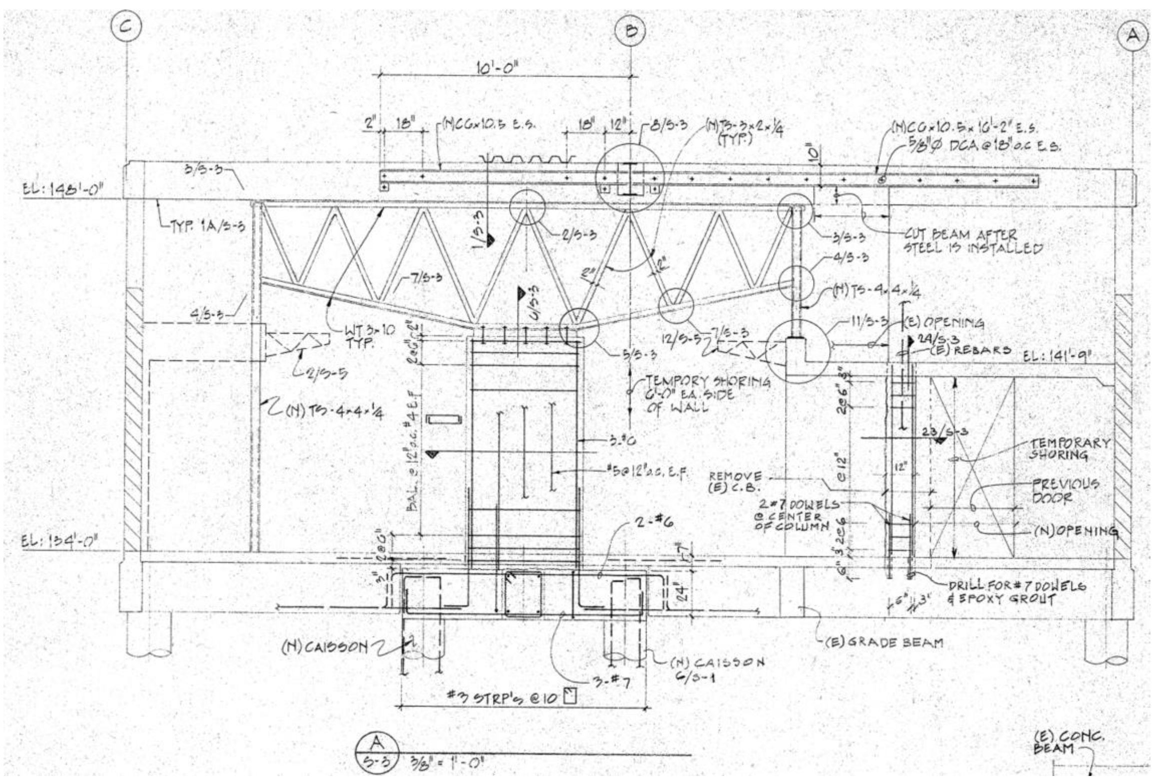
Gravity force-resisting system at the roof (as shown on 1961 drawing)



Lateral force-resisting system at the roof (as shown on 1992 renovation drawing)



Elevation of interior concrete shear wall and steel truss on architectural drawings

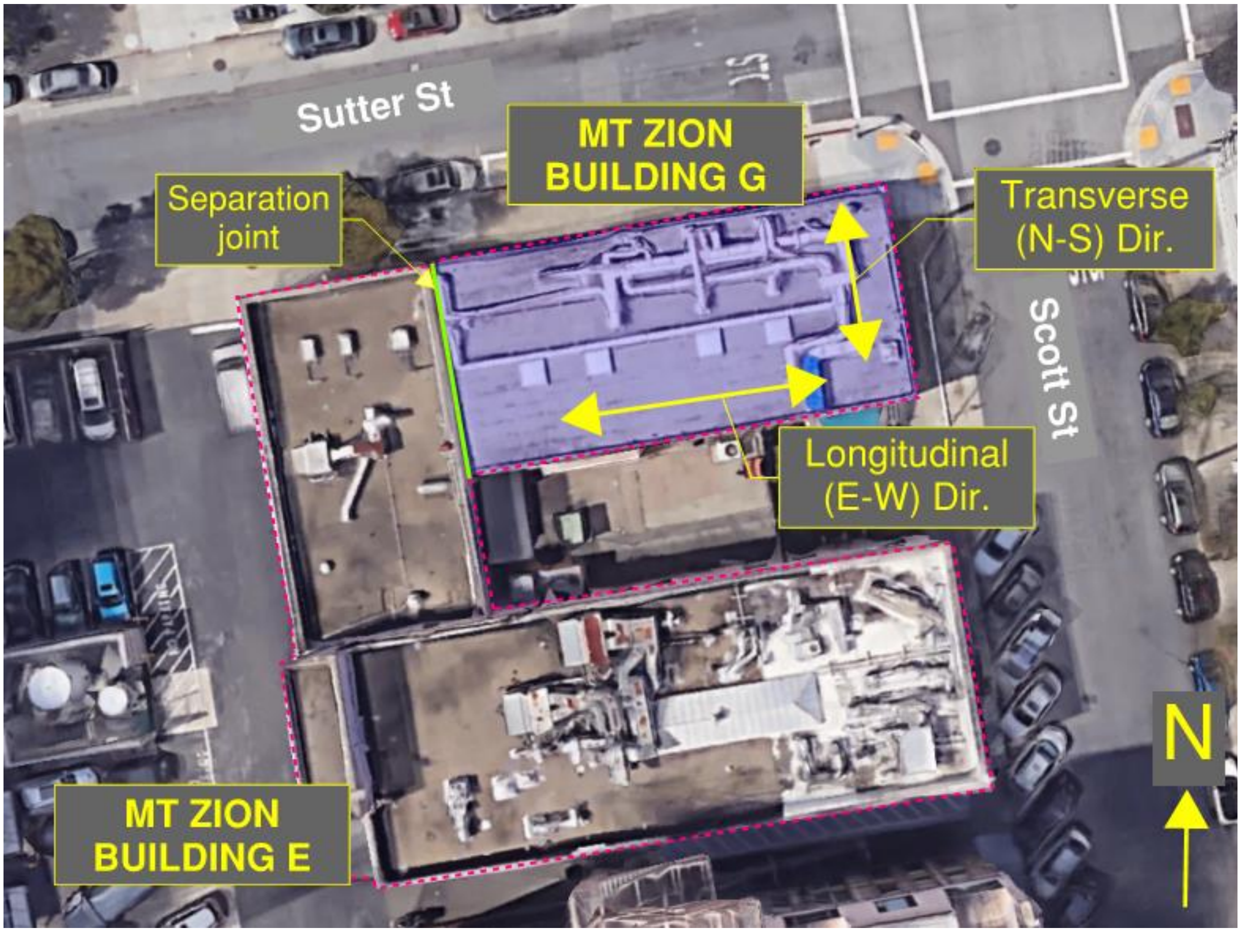


Elevation of interior concrete shear wall and steel truss on structural drawings

APPENDIX A

Additional Images

Overview of Mt. Zion campus


Sutter St
MT ZION BUILDING G
Separation joint
Transverse (N-S) Dir.
Scott St
Longitudinal (E-W) Dir.
MT ZION BUILDING E
N

Plan

East elevation (looking west)

North elevation (looking south)

South elevation (looking northeast)

South elevation and courtyard (looking northwest)

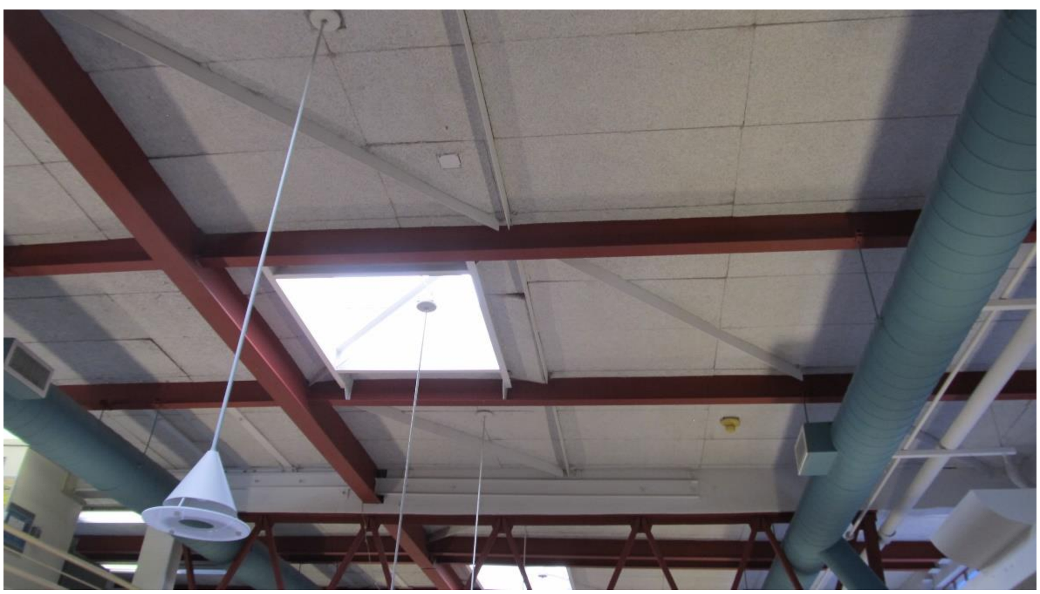
Seismic joint between Building G and Building E looking south
(Building G on the left and Building E on the right)

Main entrance on south elevation (looking north)

Interior looking west
(mezzanine on the right and first floor on the left)

Looking southwest at interior truss at shear wall. The soffit attached to the south wall that is located over patients is on the left and the mezzanine slab attached to the north wall is on the right.

Interior of south elevation with soffit (looking southwest)



Bottom of diaphragm cross-bracing protrudes below the acoustic ceiling

Offices below the east mezzanine (looking north)

Mechanical room on the second-floor east mezzanine (looking south)

APPENDIX B

ASCE 41-17 Tier 1 Checklists (Structural)

UC Campus:	**San Francisco**			Date:	**10/10/2019**		
Building CAAN:	**2026**	Auxiliary CAAN:		By Firm:	**RUTHERFORD + CHEKENE**		
Building Name:	**UCSF Mt. Zion Building G**			Initials:	**EGM**	Checked:	**BL**
Building Address:	**1675 Scott St, San Francisco, CA 94115**			Page:	**1**	of	**3**

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

LOW SEISMICITY

BUILDING SYSTEMS - *GENERAL*

	Description
C (selected) NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1) **Comments:** Unfilled metal deck with steel beam crossties functions as the roof diaphragm to deliver lateral forces to the reinforced masonry shear walls in both directions. Reinforced concrete slabs are located at the second floor and serve as diaphragm elements for the mezzanine levels. These are either doweled or bolted into the shear walls. The shear walls are continuous to the foundation and are supported by reinforced concrete grade beams and concrete piers.
C (selected) NC N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2) **Comments:** Mt. Zion Building E is located in close proximity to the west elevation of Building G. The clear distance between these structures as shown on the drawings is 3”. This measurement was confirmed in the field. Based upon the building height of 15’-6”, the required gap is 2.8”.
C (selected) NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3) **Comments:** The second floor is comprised of two mezzanine slabs. The east mezzanine serves as a mechanical floor and is connected to shear walls on all sides. Detail CS2 in the 1961 drawings shows the slab reinforcement hooked into the CMU walls. The north mezzanine serves as open office space and is connected to the shear walls on three sides. Detail BS2 in the 1978 drawings specifies a threaded rod insert that connects the slab to the CMU walls. At the time of the 1978 renovation, a steel angle was added on the underside of the north mezzanine to improve the load transfer between the slab and the north wall, as depicted in Detail 7/S-1.

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

	Description
C (selected) NC N/A U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1) **Comments:** The total wall area increases from the roof to the first floor.

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

UC Campus:	San Francisco		Date:	10/10/2019		
Building CAAN:	2026	Auxiliary CAAN:	By Firm:	RUTHERFORD + CHEKENE		
Building Name:	UCSF Mt. Zion Building G		Initials:	EGM	Checked:	BL
Building Address:	1675 Scott St, San Francisco, CA 94115		Page:	2	of	3

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

C NC N/A U (C selected)	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2) **Comments:** The total wall area increases from the roof to the first floor.
C NC N/A U (C selected)	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:** All walls are continuous to the foundation.
C NC N/A U (C selected)	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.
C NC N/A U (C selected)	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 weights of the roof and second floor are 229 kips and 289 kips, respectively, therefore; it does not change by more than 50%.
C NC N/A U (C selected)	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6) **Comments:** The building floor plan is approximately rectangular, and shear walls are located around the perimeter of the structure. Since the mezzanine slab is located along the north wall, the center of mass of the second floor will shift to the north. However, it is not expected that the distance between the center of mass and center of rigidity will equal or exceed 20% of the building dimension.

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

GEOLOGIC SITE HAZARD

	Description
C NC N/A U (C selected)	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.

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	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 1-degree), and it not susceptible to landslide.
◉ C ○ NC ○ N/A ○ U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1) **Comments:** Per "Table 1 - UCSF Group 3 Buildings Geotechnical Characteristics and Geohazards" by Egan (2019), the site is not susceptible to surface fault rupture.

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

FOUNDATION CONFIGURATION

	Description
◉ C ○ NC ○ N/A ○ U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3) **Comments:** The building width is B = 40'-0" from Grid A to C. The building height from the 1st floor to the roof is H = 15"-6", B/H = 2.58 Sa = 1.43g for at BSE-2E 0.6x Sa = 0.86 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) **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 1 in 1961 structural drawings, concrete piers are restrained by grade beams in one direction and by a 5" thick concrete slab-on-grade.

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

UC Campus:	**San Francisco**			Date:	**10/10/2019**	
Building CAAN:	**2026**	Auxiliary CAAN:		By Firm:	**RUTHERFORD + CHEKENE**	
Building Name:	**Mount Zion, Building G**			Initials:	**EGM**	Checked: **BL**
Building Address:	**1675 Scott St, San Francisco, CA 94115**			Page:	**1**	of **4**

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type RM1-RM2

LOW AND MODERATE SEISMICITY

SEISMIC-FORCE-RESISTING SYSTEM

C NC N/A U	Description
(●) C ( ) NC ( ) N/A ( ) U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1) **Comments:** Due to the rectangular configuration and the exterior CMU walls, there are at least 2 lines of shear walls in each direction. Below the mezzanine level, there are 3 lines of walls in the longitudinal (E-W) direction, and 4 lines of wall in the transverse (N-S) direction.
(●) C ( ) NC ( ) N/A ( ) U	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) **Comments:** The maximum calculated wall stress is 61 psi which is below the ASCE 41 limit of 70 psi for reinforced masonry wall at all stories.
(●) C ( ) NC ( ) N/A ( ) U	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in. (1220 mm), and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3) **Comments:** Note EAS2 in 1961 structural drawings specifies typical condition of 8” concrete block walls reinforced with 2-#3 horizontal bars at 24” o.c. (ρ_{hor} = 0.0012) and a single layer of #5 vertical bars at 16” o.c. (ρ_{vert} = 0.0025).

STIFF DIAPHRAGMS

C NC N/A U	Description
( ) C ( ) NC (●) N/A ( ) U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4) **Comments:** The building does not contain precast diaphragms.

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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type RM1-RM2

CONNECTIONS

C NC N/A U	Description
C ● NC ○ N/A ○ U ○	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) **Comments:** Details ADS3 and AES3 in the 1961 structural drawings show ledger angles bolted to the perimeter top beams; however, the connection between these angles and the roof diaphragm is unknown – Details refer to metal deck specifications, which is unavailable. Wall anchorage for out-of-plane forces in the transverse (N-S) direction with the steel framing acting as cross ties is shown on Det. CBS3 in 1961 drawings. For the longitudinal (E-W) direction, the connections between the walls and the steel framing rely on Det. ES3 in 1961 drawings. Anchorage connections are adequate when performing the Quick Check. After the 1992 alterations, a steel angle connecting the underside of the mezzanine to the north wall was added. Steel anchors for this configuration are adequate when performing the Quick Check.
C ○ NC ○ N/A ● U ○	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) **Comments:** The building does not contain wood ledgers.
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:** It is unknown whether the roof metal deck is connected to the steel ledger angles around the perimeter of the CMU walls. The mezzanine slabs at the second floor are connected to the CMU walls with #5 dowels spaced at 16” o.c or 5/8” diameter bolts spaced at 16” o.c.
C ○ NC ○ N/A ● U ○	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.7.2) **Comments:** Building does not contain topping slabs or precast concrete diaphragms.
C ● NC ○ N/A ○ U ○	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4) **Comments:** Per Detail ES1 in the 1961 structural drawings, the CMU walls are doweled into the foundation.
C ● NC ○ N/A ○ U ○	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) **Comments:** Columns in this building are limited to the southern edge of the north mezzanine. Detail DDS2 in the 1978 drawings show a positive connection between the HSS columns to the beams.

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Building CAAN:	**2026**	Auxiliary CAAN:	By Firm:	**RUTHERFORD + CHEKENE**		
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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type RM1-RM2

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

STIFF DIAPHRAGMS

C NC N/A U	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) **Comments:** The stair on the northwest side of the structure is 11'-0" long, and the adjacent wall is 88'-0" long.
( ) 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) **Comments:** The stair on the west end of the north wall creates an 11'-0" long opening in the north mezzanine.

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:** Steel 10B11.5 beams in the transverse (N-S) direction function as cross ties between the north and south exterior walls.
(•) 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:** Skylight openings at the roof level are not adjacent to 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) **Comments:** Skylight openings at the roof level are not adjacent to shear walls.
( ) 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 structure does not contain straight sheathing.

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Collapse Prevention Structural Checklist For Building Type RM1-RM2

C NC N/A U (N/A selected)	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) **Comments:** The structure does not contain wood diaphragms.
C NC N/A U (N/A selected)	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) **Comments:** The structure does not contain wood diaphragms.
C NC N/A U (N/A selected)	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 structure contains a metal deck diaphragm at the roof.

CONNECTIONS

	Description
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. (3 mm) before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2) **Comments:** Anchors are not connected to wood structural elements.

APPENDIX C

UCOP Seismic Safety Policy Falling Hazards Assessment Summary

UC Campus:	**San Francisco**			Date:	**10/10/2019**		
Building CAAN:	**2026**	Auxiliary CAAN:		By Firm:	**Rutherford+Chekene**		
Building Name:	**UCSF Mt. Zion Building G**			Initials:	***EGM***	Checked:	***BL***
Building Address:	**1675 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)** **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:** The UCSF Mt. Zion campus assistant engineer indicates that gas is not supplied to Building G. However, gas is supplied to the adjacent structure, Building E. Building E is located in close proximity to Building G as the two structures are separated by a 3” seismic joint.
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**
Mechanical equipment	5	10	Roof top equipment consists of duct work
Roofing, waterproofing, and insulation	5	5	Built-up roof (smooth-surfaced) on 1/2" rigid insulation
Metal deck	2	2	18 ga. Metal deck assumed
Beams/girders	11	11	Concrete beams around perimeter and steel wide flange framing below roof
Steel truss	0.3	0.3	Steel truss added after 1992 alterations
MEP	3	3	MEP hung from underside of roof slab
Ceiling, lighting, and misc.	5	5	Acoustic panel ceiling, lighting, and misc. hung from underside of roof slab
Columns	0	0	
Partitions	0	0	
Total	32	37	

1 - 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.

2 - Excluding the steel truss, the roof framing was not modified during the 1978 and 1992 alterations.

3 - The steel truss located on Grid 2 is composed of TS4x4x1/4 posts, WT3x10 chords, and TS3x2x1/4 diagonals in web.

4 - The roof is directly supported by CMU walls and the steel truss. No columns extend to the roof.

5 - No partitions extend to the roof.

	Seismic Weight	Dead Load	
EAST MEZZANINE 2nd Floor MEP Rm.	**psf**	**psf**	**Remarks**
Mechanical equipment	10	20	Estimated equipment weight
Slab	100	100	8" NWC slab
Beams/girders	0	0	CMU walls support the slab.
MEP	5	5	MEP hung from underside of roof slab
Ceiling, lighting, and misc.	4	4	Lay-in ceiling, lighting, and misc. hung from underside of floor slab
Columns	0	0	
Partitions	5	0	
Total	124	129	

1 - This flat load is located at the second floor between Grids A-C/3-4. at EL. 9'-0" relative to the first floor.

2 - The equipment is assumed to weigh 20 psf where it is located. The equipment is located on approximately 1/2 of the room area and therefore, 10 psf is assumed for seismic mass.

3 - The mechanical framing is part of the original 1961 structure. The thickness is specified on Det. CS2 /S2.

4 - The concrete slab is directly supported by original CMU walls.

5 - The partitions are located between the first floor and the underside of the mechanical room only.

	Seismic Weight	Dead Load	
SOFFIT South Elevation	**psf**	**psf**	**Remarks**
Soffit framing	6	6	Metal stud framing encased in gyp. board
Lighting and misc.	3	3	Lighting, and misc. hung from underside
Columns	1	0	HSS steel columns
Partitions	0	0	
Total	10	9	

1 - This flat load represents an interior nonstructural soffit that is located on the south wall between Grids B.3-C/1-3. at EL. 8'-8" relative to the first floor.

2 - Per Det 1 / S-5, assembly is comprised of C joists at 16" o.c. covered with gyp. board.

3 - Flat load includes weight of (1) HSS4x4x1/4 and (7) HSS2.5x2.5x3/16 columns below soffit in a 411 ft^2 area. Column trib. height is 4'-4".

	Seismic Weight	Dead Load	
NORTH MEZZANINE 2nd Floor Open Office	**psf**	**psf**	**Remarks**
Flooring	5	5	Carpet and vinyl composition tiles
Slab	81	81	6.5" NWC slab
Beams/girders	1	1	Concrete beam below slab and steel angle at interface with wall
MEP	5	5	MEP hung from underside of roof slab
Ceiling, lighting and misc.	4	4	Lay-in ceiling, lighting, and misc. hung from underside of floor slab
Columns	0.2	0	HSS steel columns
Partitions	5	0	
Total	102	97	

1 - This flat load is located at the second floor between Grids A-A.3/1-4 at EL. 7'-9" relative to the first floor.

2 - This mezzanine was constructed during the 1978 renovation. The slab thickness is specified on Det. BSE / S2.

3 - During the 1992 renovation, a concrete beam was added on Grid 3, as shown on Det. 3/S-4.

4 - The concrete slab is supported by CMU walls.

5 -The flat load includes weight of (1) HSS4x4x1/4 and (6) HSS2.5x2.5x3/16 columns below soffit in a 1042 ft^2 area. Column trib. height is 3'-10.5".

6 - The partitions are located between the first floor and the underside of the mezzanine only.

Story Weight

ROOF

Diaphragm			
Diaphragm Load	**Floor Area (ft^2)**	**Floor Weight (psf)**	**Diaphragm Load Seismic Weight (kips)**
Roof	3,552	32	112

Tributary Walls to Roof			
Wall Line	**Tributary Height (ft)**	**Horizontal Area (ft^2)**	**Wall Seismic Weight (kips)**
A	3.875	20.2	21
C	7.75	44.4	45
1	7.75	29.3	30
2	3.875	2.8	3
4	3.25	14.7	15

Nonstructural Soffit			
Wall Line	**Total Weight (kips)**	**Percentage resisted by Roof (%)**	**Soffit Seismic Weight (kips)**
A	4	56%	2
		Σ =	**229**

SECOND FLOOR / MEZZANINE

Diaphragm			
Diaphragm Load	**Floor Area (ft^2)**	**Floor Weight (psf)**	**Diaphragm Load Seismic Weight (kips)**
East Mezzanine	696	124	86
North Mezzanine	1,042	102	106

Tributary Walls to the second floor			
Wall Line	**Tributary Height (ft)**	**Horizontal Area (ft^2)**	**Wall Seismic Weight (kips)**
A	7.75	46.3	47
B	4.5	3.3	4
2	7.75	5.7	6
3	4.5	7.7	9
4	7.75	27.0	28
		Σ =	**286**

Floor Levels	**Total Seismic Weight (kips)**	
Roof	229	
Second Floor	286	
First Floor		
Σ =	**550**	**kips**

Notes

1 - Seismic base is set at the first floor.

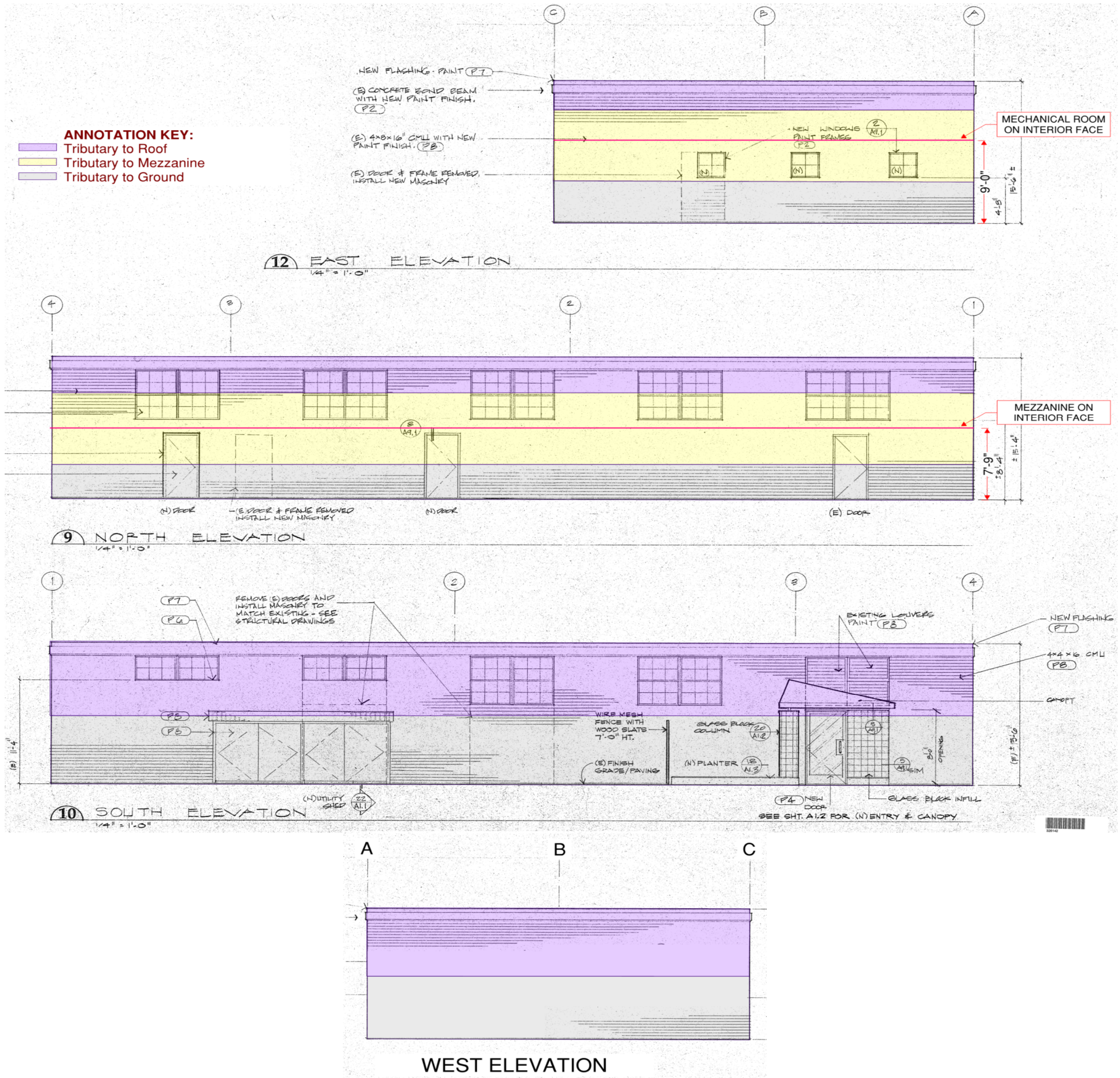
2 - Elevations are estimated based upon Sheet A2.2 in the 1992 drawings and are specified with respect to top of slab at first floor.

3 - Detail EAS2 in the original 1961 drawings specifies typ. 8" CMU walls as solid grouted. Normal weight CMU is assumed. W_{CMU} = 84 psf.

4- The nonstructural soffit is attached to wall on Line C. Its contribution to the roof is calculated as a reaction assuming a simple supported beam spanning from the first floor to the roof with a lateral load located at El. 8'-8".

5 - The wall weight includes exterior and interior CMU walls. 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.

Tributary Wall Heights



Period

C_t=	0.02
h_n (ft)=	15.50
B=	0.75

T=	0.16	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 roof.

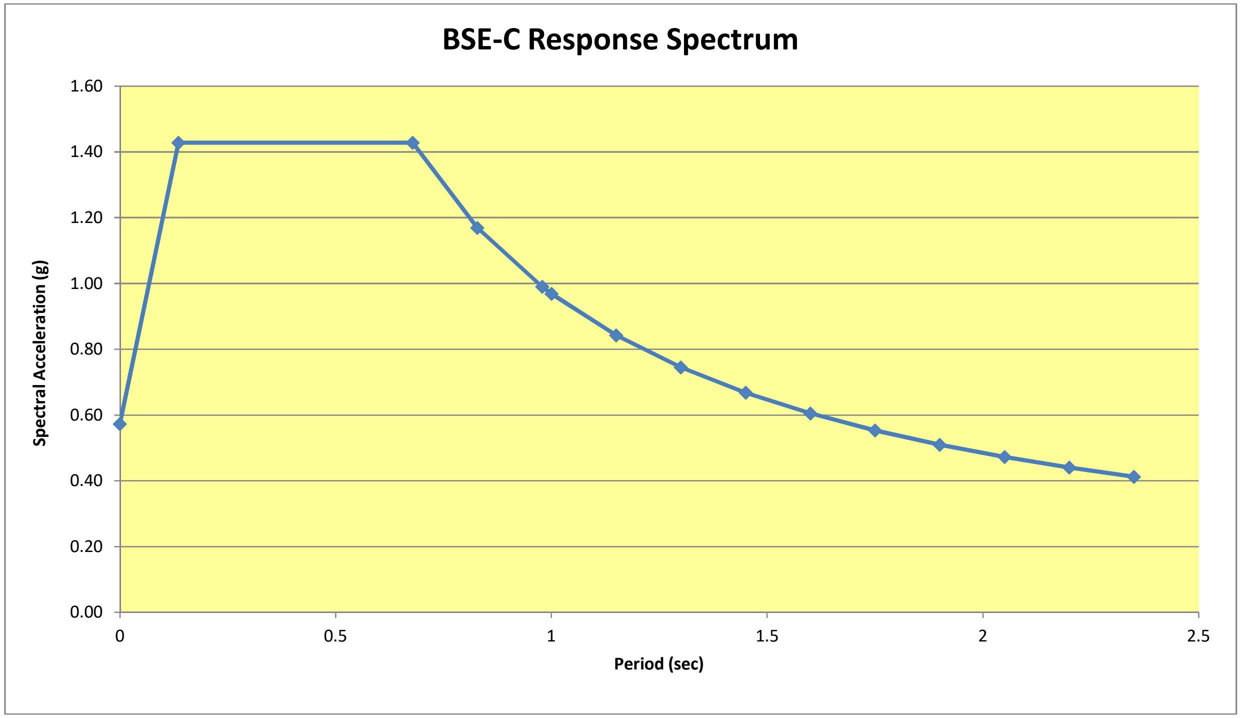
where

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	0.84
1.30	0.75
1.45	0.67
1.60	0.61
1.75	0.55
1.90	0.51
2.05	0.47
2.20	0.44
2.35	0.41



BSE-C

β =	0.05	
B_1 =	1.00	
S_S =	1.431	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 =	0.16	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 Vertical 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=	0.16	s	
Sa=	1.43	g	(See Note 3)
W=	550	kips	
C=	1.0	Per ASCE 41-17 Table 4-7	

V=	787	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)
Roof	7.75	15.50	229	3,546	0.61	484	484
Second Floor	7.75	7.75	286	2,220	0.39	303	787
Σ =	15.5		515	5,766	1	787	

Notes:

1- The base of building is set at 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) 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.

5 - The structure contains a flexible wood diaphragm at the roof and a rigid mezzanine diaphragm at the mezzanine slabs. Since the concrete diaphragms are only partial and do not extend across the entire floor plan, the building is considered to be dominantly type RM1 and a C = 1.0 is used.

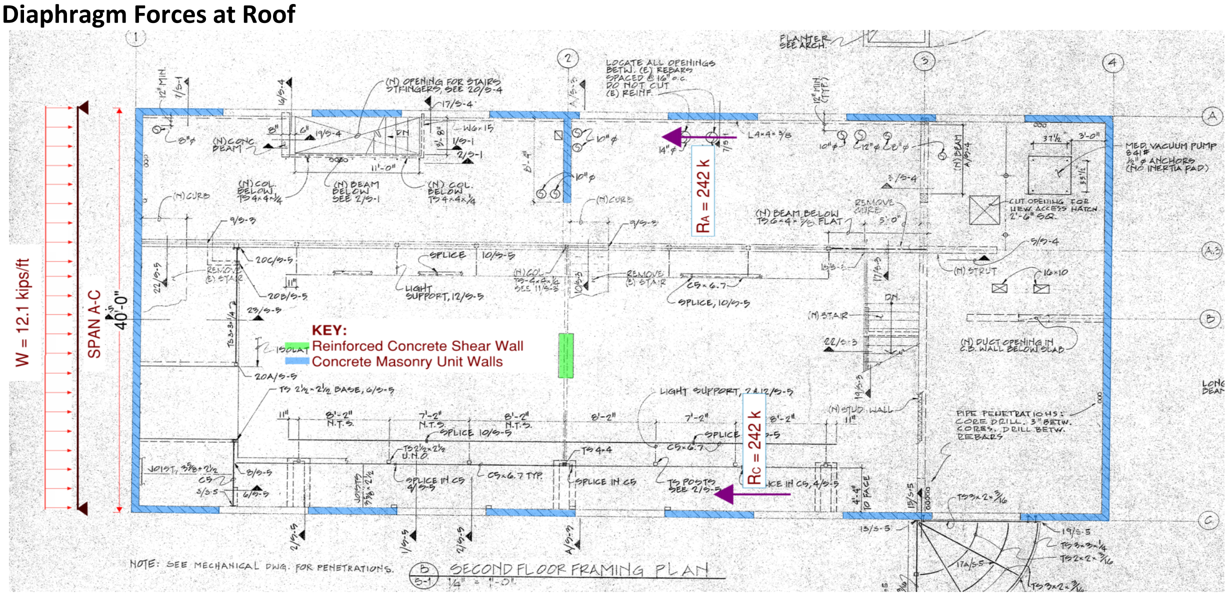
Seismic Force Distribution in Shear Walls

Seismic Force Acting in Longitudinal (E-W) Direction

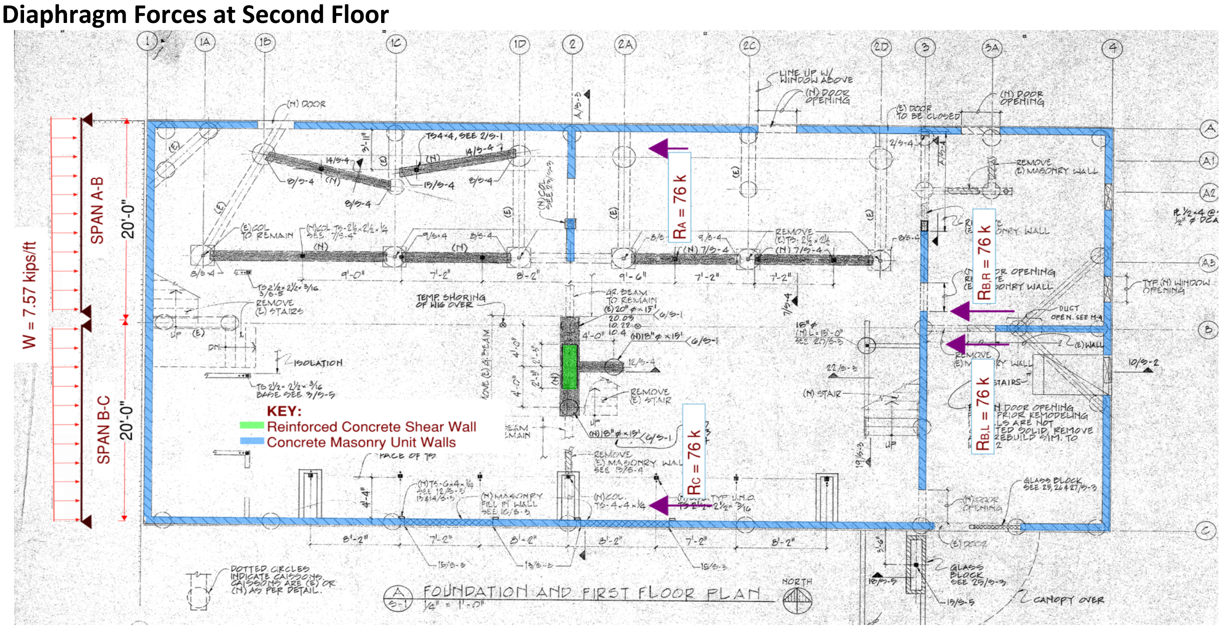
Level	Grids	Seismic Force (kips)	Total Length Seismic Force is Acting (ft)	Distributed Load (kips/ft)	Span Length (ft)	Reaction (kips)
Roof	A-C	484	40.0	12.10	40.0	242
Second Floor	A-B	303	40.0	7.57	20.0	76
Second Floor	B-C	303	40.0	7.57	20.0	76

Forces are distributed to walls based upon tributary area. The applied force is simplified to be a uniform line load and local increase due to the location of the mezzanine masses are ignored.

Diaphragm Forces at Roof



Diaphragm Forces at Second Floor



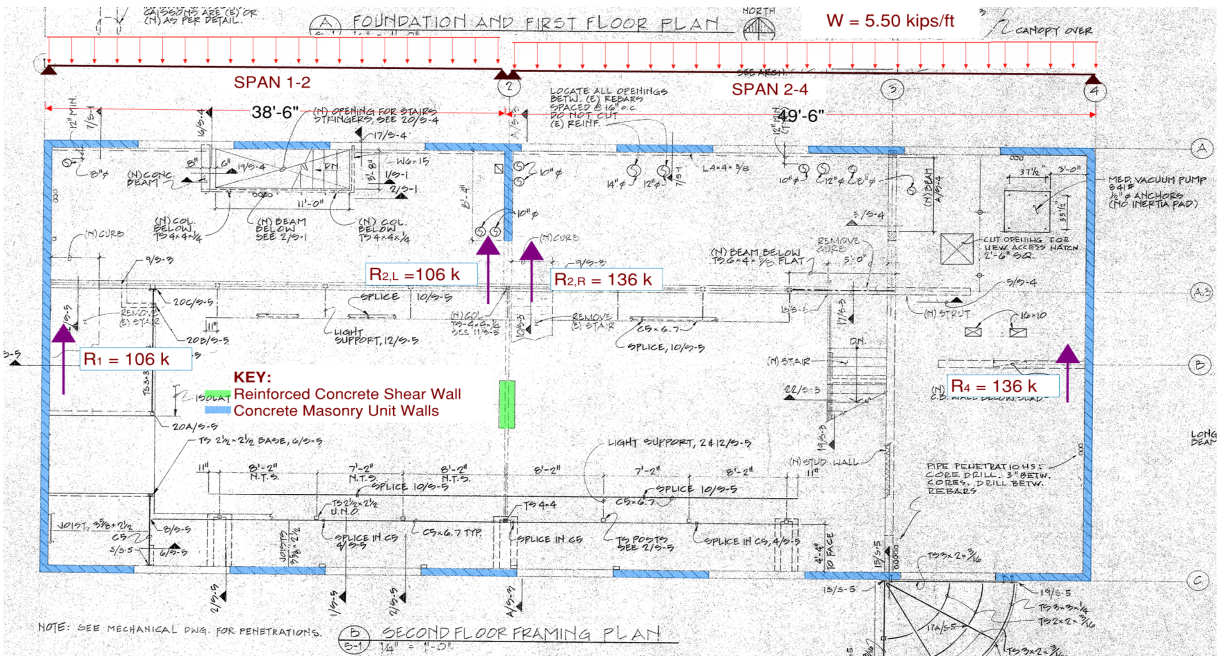
Seismic Force Distribution in Shear Walls

Seismic Force Acting in Transverse (N-S) Direction

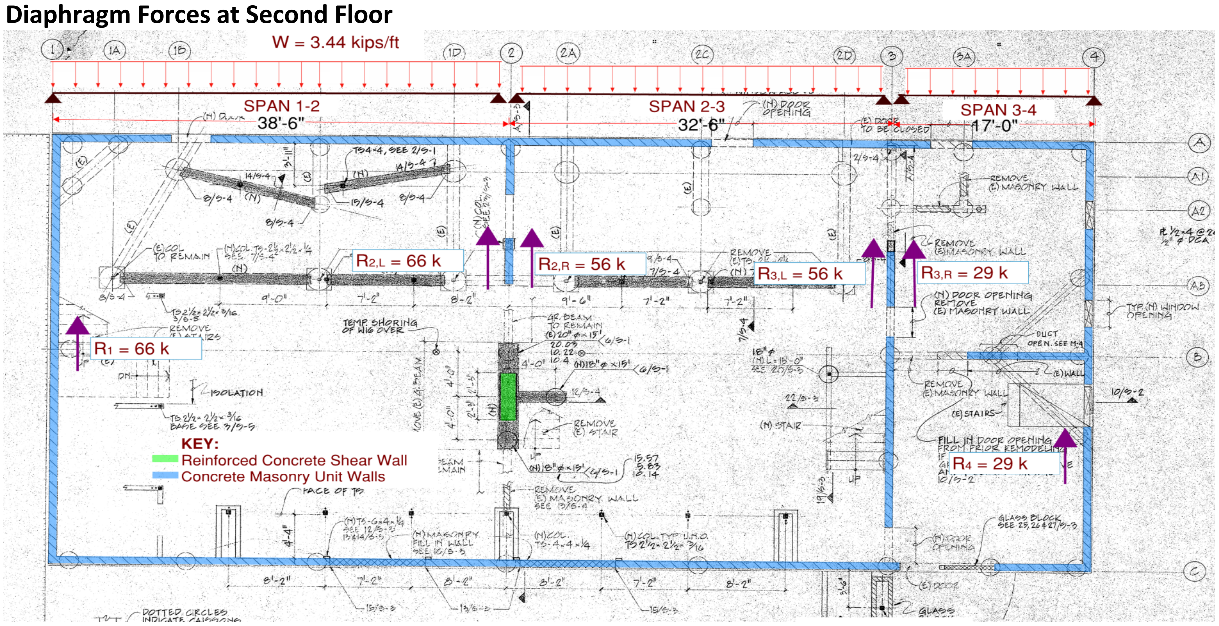
Level	Grids	Seismic Force (kips)	Total Length Seismic Force is Acting (ft)	Distributed Load (kips/ft)	Span Length (ft)	Reaction (kips)
Roof	1-2	484	88.0	5.50	38.5	106
Roof	2-4	484	88.0	5.50	49.5	136
Second Floor	1-2	303	88.0	3.44	38.5	66
Second Floor	2-3	303	88.0	3.44	32.5	56
Second Floor	3-4	303	88.0	3.44	17.0	29

Forces are distributed to walls based upon tributary area. The applied force is simplified to be a uniform line load and local increase due to the location of the mezzanine masses are ignored.

Diaphragm Forces at Roof



Diaphragm Forces at Second Floor



Average Wall Stress Check

Average Stresses

Ms = 4.5

Seismic Force Acting in Longitudinal (E-W) Direction

Wall on Line A

Story	Story Seismic Force Demand	Shear Force Demand	Wall Area	Average Shear Stress Demand	Tier 1 Shear Limit	Wall OK?
	(kips)	(kips)	(ft^2)	(psi)	(psi)	
Roof - Second Floor	242	242	31	12	70	OK
Second Floor - First Floor	76	318	49	10	70	OK

Wall on Line B

Story	Story Seismic Force Demand	Shear Force Demand	Wall Area	Average Shear Stress Demand	Tier 1 Shear Limit	Wall OK?
	(kips)	(kips)	(ft^2)	(psi)	(psi)	
Second Floor - First Floor	151	151	6	37	70	OK

Wall on Line C

Story	Story Seismic Force Demand	Shear Force Demand	Wall Area	Average Shear Stress Demand	Tier 1 Shear Limit	Wall OK?
	(kips)	(kips)	(ft^2)	(psi)	(psi)	
Roof - Second Floor	242	242	31	12	70	OK
Second Floor - First Floor	76	318	51	10	70	OK

Seismic Force Acting in Transverse (N-S) Direction

Wall on Line 1

Story	Story Seismic Force Demand	Shear Force Demand	Wall Area	Average Shear Stress Demand	Tier 1 Shear Limit	Wall OK?
	(kips)	(kips)	(ft^2)	(psi)	(psi)	
Roof - Second Floor	106	106	28	6	70	OK
Second Floor - First Floor	66	172	25	10	70	OK

Wall on Line 2

Story	Story Seismic Force Demand	Shear Force Demand	Wall Area	Average Shear Stress Demand	Tier 1 Shear Limit	Wall OK?
	(kips)	(kips)	(ft^2)	(psi)	(psi)	
Roof - Second Floor (CMU Wall)	242	101	5	29	70	OK
Roof - Second Floor (Concrete Wall)	242	141	6	36	100	OK
Second Floor - First Floor (CMU Wall)	122	223	6	61	70	OK

Note - A portion of the wall from the roof to the second floor on Line 2 is concrete and a portion is CMU. The forces are distributed based upon relative rigidity assuming the shear rigidity of a cantilevered wall. See below:

CMU wall and concrete shear wall below truss resist Shear Demand at roof on Grid 2

Em =	1350	ksi	See Notes 5 and 6
Ec =	3321	ksi	See Note 7
Accumulated Shear Force Demand =	242	kips	
CMU Wall Area =	762	in^2	
Height CMU wall =	7.75	ft	
Shear stiffness of cantilver wall (3H/AE) =	3688	kip/in	
CIP Concrete Wall Area =	864	in^2	
Height Concrete Wall =	15.5	ft	
Shear stiffness of cantilver wall (3H/AE) =	5140	kip/in	
Transferred Shear to CMU Wall=	101	kips	(Shear Force resisted by CMU wall and transferred to lower level)
Average Shear Stress in CMU Wall =	29	psi	
Tier 1 Shear Stress Limit =	70	psi	
Acceptance criteria	OK		
Transferred Shear to CIP Concrete Wall=	141	kips	
Average Shear Stress in CIP Concrete Wall =	36	psi	
Tier 1 Shear Stress Limit =	100	psi	
Acceptance criteria	OK		

Wall on Line 3

Story	Story Seismic Force Demand	Shear Force Demand	Wall Area	Average Shear Stress Demand	Tier 1 Shear Limit	Wall OK?
	(kips)	(kips)	(ft^2)	(psi)	(psi)	
Second Floor - First Floor	85	85	15	9	70	OK

Wall on Line 4

Story	Story Seismic Force Demand	Shear Force Demand	Wall Area	Average Shear Stress Demand	Tier 1 Shear Limit	Wall OK?
	(kips)	(kips)	(ft^2)	(psi)	(psi)	
Roof - Second Floor	136	136	28	8	70	OK
Second Floor - First Floor	29	165	20	13	70	OK

Notes:

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

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

Table 4-8. M_s Factors for Shear Walls

Wall Type	CP[a]	LS[a]	IO[a]
	Level of Performance		
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 - Tier 1 shear stress limit of 70 lb/in^2 is defined for buildings with reinforced masonry shear walls based upon Table 17-34/ASCE 41-17.

4 - Gridline 2 contains steel truss connecting the flexible roof diaphragm to a concrete shear wall. This calculations assumes the lateral load is resisted by CMU wall and concrete wall below truss.

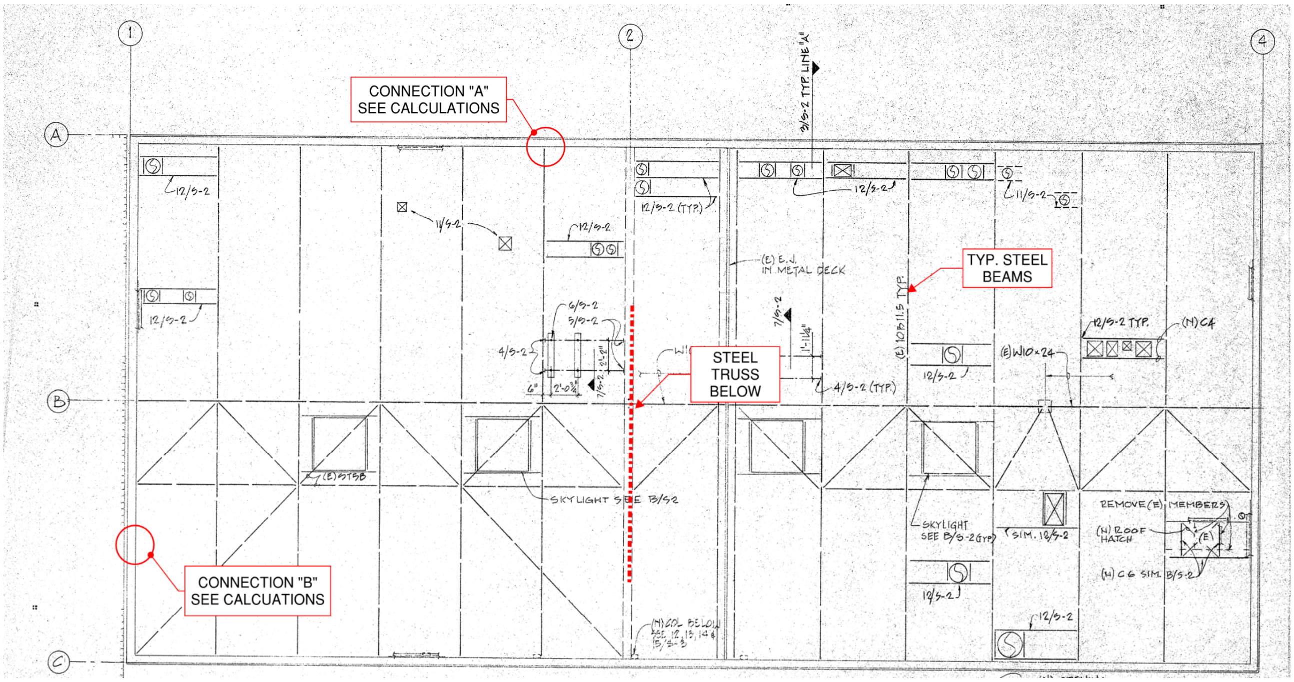
5 - Em = 900 f'm per ASCE 41-17 Section 11.2.3.7 & TMS 402 Section 1.8.2.2.1 for concrete block.

6 - f'm = 1500 psi for reinforced soild grouted units per ASCE 41-17, Table 11-2b.

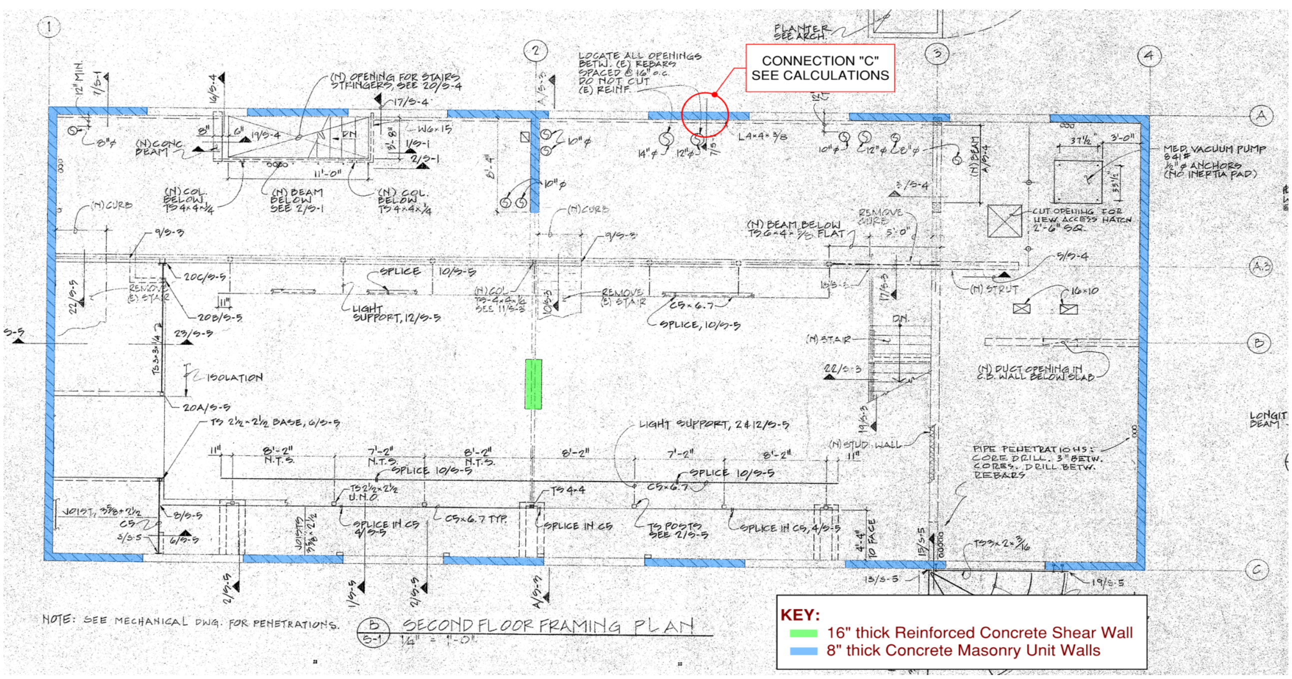
7- Ec = $wc^{1.5}$ x 33 x sqrt(f'c) per ACI 318 Section 8.5.1. Compressive strength of wall below truss is 3000 psi based on General Notes in 1992 drawings.

Plan of connection locations

See the following pages for the out-of-wall anchorage calculations of connection A, B, and C, which are located as indicated on the plans below:



Roof Plan



Second floor Plan

Flexible Diaphragm Connection Forces Per Tier 2 Procedure - Connection "A"

Tier 2 Procedure per Section 7.2.11.1 in ASCE 41-17:

7.2.11.1 Out-of-Plane Wall Anchorage to Diaphragms. Each wall shall be positively anchored to all diaphragms that provide lateral support for the wall or are vertically supported by the wall.

$$F_p = 0.4 S_{XS} k_a k_h \chi W_p \quad (7\text{-}9)$$

$$F_{p,\min} = 0.2 k_a \chi W_p \quad (7\text{-}10)$$

$$k_a = 1.0 + \frac{L_f}{100} \quad (7\text{-}11)$$

$$k_h = \frac{1}{3}\left(1 + \frac{2z_a}{h_n}\right) \quad (7\text{-}12)$$

Per Section 10.3.6.1 in ASCE 41-17, "cast-in-place connection systems shall be considered **force-controlled."**

Per Section 7.5.2.2.2 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;

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

Connection between Roof Diaphragm and Exterior CMU Walls at South Elevation

Reference: Detail CBS3 in 1961 structural drawings

Design Parameters:

χ =	0.8	(Table 7-2 / ASCE 41-17)	κ =	0.75	(Table 6-1 / ASCE 41-17, for default material properties)
S_{XS} =	1.431	g	χ =	1.0	(Collapse Prevention Performance Level)
w_p =	84	psf	C_1C_2 =	1.0	(Per FEMA P-2006, Section 4.7.4, the factors J, C1, and C2 do not apply to Fp
			J =	1.0	forces and the presumption is that there is no ductility or limiting mechanism for reducing out-of-plance forces.)

BOLTS IN TENSION

Tension Demand

Anchor spacing =	6.7	ft	
Trib. Wall Height =	7.8	ft	
A_p =	51.7	ft^2	
k_a =	1.49		(minimum of 2.0 and 1 + 49ft/100ft)
k_h =	1.0		(1.0 for flexible diaphragms)
F_p =	2.96	kips	(Maximum of Eq. 7-9 and 7-10)
Q_{UF} =	2.96	kips	(Per Eq. 7-35, considering Q_E = F_p)

Tension Capacity

Tension Capacity is checked using Hilti Profis ®. See following pages.

Anchorage Check with Hilti PROFIS®

Connection demand

W_{DL} =	37	psf	
W_{LL} =	20	psf	
Trib. Area =	66.67	ft^2	Anchor spacing x 40ft /4
Shear due to gravity =	3.06	kips	(Considerind load combination 1.1DL + 0.275LL)
Tension force =	2.96	kips	(Tension force equals Q_{UF})
Vert. Ecc. Moment =	11.29	kips-in	(Moment due to vertical eccentricity between bolts at bottom of the beam flange and the center of the cast-in-anchors, 2.96 kips x 3.8125in)
Plan Ecc. Moment =	7.45	kips-in	(Moment from plan eccentricity of gravity load from the centriod of the 4" bearing area to the face of the wall, 3.06 kips x 2.4375in)
Applied Moment =	11.29	kips-in	(Conservatively, moment due to vertical eccentricity is applied in Hilti Profis input)

Tension Load	Capacity (including κ)		Demand		Utilization	
Steel strength	6,986	lb	1,932	lb	28%	
Pullout strength	6,129	lb	1,932	lb	32%	
Concrete Breakout	11,537	lb	4,131	lb	36%	
					36%	(Maximum)
Shear Load						
Steel strength	4,191	lb	765	lb	18%	
Pryout strength	35,786	lb	3,060	lb	9%	
Concrete edge failure	10,125	lb	3,060	lb	30%	
					30%	(Maximum)
Interaction	32%					

BOLTS IN SHEAR

Shear Demand

Anchor spacing =	6.7	ft	
Trib. Wall Height =	7.8	ft	
A_p =	51.7	ft^2	
F_p =	2.96	kips	(Maximum of Eq. 7-9 and 7-10)
Q_{UF} =	2.96	kips	(Per Eq. 7-35, considering Q_E = F_p)

Shear Capacity

No. bolts =	2		
D_{bolt} =	0.625	in	
A_{bolt} =	0.307	in^2	
Fy =	36	ksi	(ASTM A36 assumed, Table 4-5 in ASCE 41-17 for default yield strength)
Q_{CL} =	13.3	kips	(Lower-bound shear capacity, Q_{CL} = 0.6 x No. bolts x Fy x A_{bolt})
κQ_{CL} =	9.9		
Q_{UF} / (κQ_{CL}) =	0.30		
Acceptance criteria	OK		

STEEL ANGLE BENDING

Angle properties:

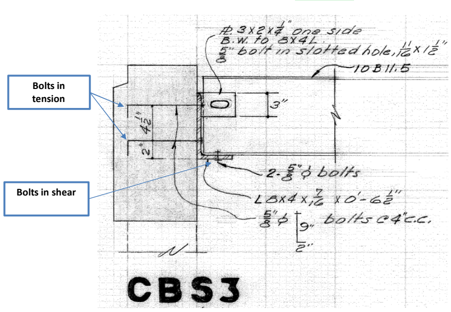
Thickness =	0.4375	in	
Width =	6.5	in	
Fy =	37	ksi	(ASTM A36 assumed, Table 4-5 / ASCE 41-17)
Zy =	0.31	in3	(Zy = t^2 x b / 4)

Capacity

M_{CL} =	11.5	kips-in	(M_{CL} = Fy Zy)
κM_{CL} =	8.6	kips-in	

Demand

Tension force =	2.96	kips
Eccentricity =	1.5625	in
M_{UF} =	4.6	kips-in
M_{UF} / (κM_{CL}) =	0.54	
Acceptance criteria	OK	



Note:

1 -The 0.75 seismic reduction factor in ACI 318, Section 17.2.3.4.4 applied to concrete failure modes to determine the design tensile strength o f concrete is applied as the concrete failure modes have reduced capacity under cyclic loads.

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Specifier's comments:

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 ~~GR. 36~~ 5/8**
Item number:	not available
Effective embedment depth:	h_{ef} = 8.000 in.
Material:	ASTM F 1554
Proof:	Design Method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.250 in.
Anchor plate[R] :	l_x x l_y x t = 6.500 in. x 8.000 in. x 0.250 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 9.250 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

Lower-bound steel strength is 27 ksi, per Section 10.2.2.5 / ASCE 41-17

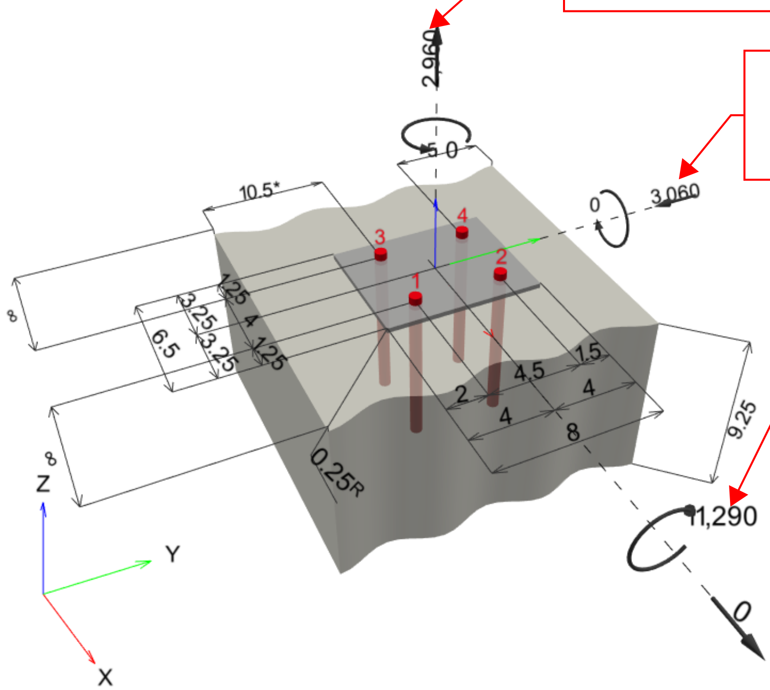
Lower-bound concrete strength, per Table 10-2 / ASCE 41-17

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]

Out-of-plane seismic load per Eq. 7-9 / ASCE 41-17

1.1DL + 0.275 LL Gravity reaction from beam



Moment due to vertical eccentricity
M = (2.96k)(3.8125in)

Conservatively, only the moment due to vertical eccentricity is applied - the counteracting moment from the plan eccentricity of the gravity load is neglected.

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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 2{,}960$; $V_x = 0$; $V_y = -3{,}060$; $M_x = -11{,}290$; $M_y = 0$; $M_z = 0$;	yes	39

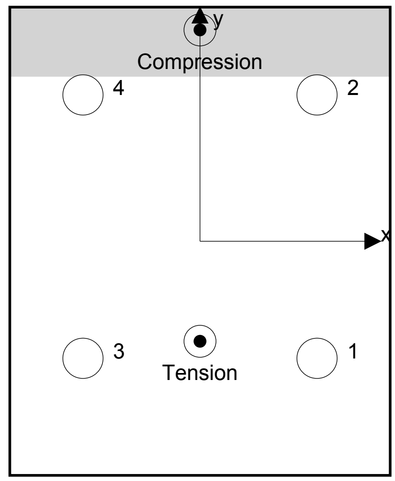
2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,932	765	0	-765
2	134	765	0	-765
3	1,932	765	0	-765
4	134	765	0	-765



max. concrete compressive strain: 0.07 [‰]
max. concrete compressive stress: 309 [psi]
resulting tension force in (x/y)=(0.000/-1.709): 4,131 [lb]
resulting compression force in (x/y)=(0.000/3.611): 1,171 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity κ N_n [lb]	Utilization $\beta_N = N_{ua}/\kappa N_n$	Status
Steel Strength*	1,932	~~9,831~~ 6,986	~~20~~ 28	OK
Pullout Strength*	1,932	~~5,720~~ 6,129	~~34~~ 32	OK
Concrete Breakout Failure**	4,131	~~10,708~~ 11,537	~~39~~ 36	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-14 Eq. (17.4.1.2)
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.23	~~58,000~~ 1.5 x 27,000 psi = 40,500 psi

Calculations

N_{sa} [lb]
~~13,108~~ 9,315 lb

Results

N_{sa} [lb]	ϕ_{steel}	κ	κ $\phi\ N_{sa}$ [lb]	N_{ua} [lb]
~~13,108~~ 9,315 lb	~~0.750~~ 1.0	0.75	~~9,831~~ 6,986 lb	1,932

3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\, N_p$ ACI 318-14 Eq. (17.4.3.1)
$N_p = 8\, A_{brg}\, f'_c$ ACI 318-14 Eq. (17.4.3.4)
$\phi\ N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.45	1.000	3,000

Calculations

N_p [lb]
10,896

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	κ	$\phi_{nonductile}$	κ $\phi\ N_{pn}$ [lb]	N_{ua} [lb]
10,896	~~0.700~~ 1.0	0.750	0.75	1.000	~~5,720~~ 6,129	1,932

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-14 Eq. (17.4.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
8.000	0.000	1.959	5.000	1.000

c_{ac} [in.]	k_c	λ_a	f_c' [psi]
-	24	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
560.00	576.00	1.000	0.860	0.825	1.000	29,745

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	K	$\phi_{nonductile}$	K N_{cbg} [lb]	N_{ua} [lb]
20,510	~~0.700~~ 1.0	0.750	0.75	1.000	~~10,768~~ 11,537	4,131

4 Shear load

	Load V_{ua} [lb]	Capacity κ V_n [lb]	Utilization $\beta_V = V_{ua}/\cancel{\phi}\kappa V_n$	Status
Steel Strength*	765	~~5,112~~ 4,191	~~15~~ 18	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	3,060	~~33,401~~ 35,786	~~10~~ 9	OK
Concrete edge failure in direction y-**	3,060	~~9,450~~ 10,125	~~33~~ 30	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.23	~~58,000~~ 1.5 x 27,000 psi = 40,500 psi

Calculations

V_{sa} [lb]
~~7,865~~ 5,588 lb

Results

V_{sa} [lb]	ϕ_{steel}	κ	κ $V_{sa,eq}$ [lb]	V_{ua} [lb]
~~7,865~~ 5,588 lb	~~0.650~~ 1.0	0.75	~~5,112~~ 4,191 lb	765

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4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	8.000	0.000	0.000	5.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	24	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
560.00	576.00	1.000	1.000	0.825	1.000	29,745

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	K	$\phi_{nonductile}$	K V_{cpg} [lb]	V_{ua} [lb]
47,715	~~0.700~~	1.000	0.75	1.000	~~33,401~~	3,060
	1.0				35,786	

4.3 Concrete edge failure in direction y-

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$	ACI 318-14 Eq. (17.5.2.1b)
$\phi\ V_{cbg} \geq V_{ua}$	ACI 318-14 Table 17.3.1.1
A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)	
$A_{Vc0} = 4.5\ c_{a1}^2$	ACI 318-14 Eq. (17.5.2.1c)
$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$	ACI 318-14 Eq. (17.5.2.5)
$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$	ACI 318-14 Eq. (17.5.2.6b)
$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$	ACI 318-14 Eq. (17.5.2.8)
$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f_c'}\ c_{a1}^{1.5}$	ACI 318-14 Eq. (17.5.2.2a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
10.500	-	0.000	1.000	9.250

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
5.000	1.000	0.625	3,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
328.38	496.13	1.000	1.000	1.305	15,631

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	κ	$\phi_{nonductile}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
13,500	~~0.700~~ 1.0	1.000	0.75	1.000	~~9,450~~ 10,125	3,060

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
~~0.384~~ 0.36	~~0.324~~ 0.30	5/3	~~30~~ 32	~~OK~~

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2018, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.

~~Fastening meets the design criteria!~~

FASTENING MEETS THE TIER 1 / ASCE 41-17 CRITERIA

Flexible Diaphragm Connection Forces Per Tier 2 Procedure - Connection "B"

Tier 2 Procedure per Section 7.2.11.1 in ASCE 41-17:

7.2.11.1 Out-of-Plane Wall Anchorage to Diaphragms. Each wall shall be positively anchored to all diaphragms that provide lateral support for the wall or are vertically supported by the wall.

$$F_p = 0.4 S_{XS} k_a k_h \chi W_p \quad (7\text{-}9)$$

$$F_{p,\min} = 0.2 k_a \chi W_p \quad (7\text{-}10)$$

$$k_a = 1.0 + \frac{L_f}{100} \quad (7\text{-}11)$$

$$k_h = \frac{1}{3}\left(1 + \frac{2z_a}{h_n}\right) \quad (7\text{-}12)$$

Per Section 10.3.6.1 in ASCE 41-17, "cast-in-place connection systems shall be considered **force-controlled."**

Per Section 7.5.2.2.2 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;

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

Connection between Metal Deck and Exterior CMU Walls

Reference: Detail AES3 in 1961 structural drawings

Design Parameters:

χ =	0.8	(Table 7-2 / ASCE 41-17)
S_{XS} =	1.431 g	
w_p =	84 psf	

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	(Per FEMA P-2006, Section 4.7.4, the factors J, C1, and C2 do not apply to Fp forces and the presumption is that there is no ductility or limiting mechanism for reducing out-of-plance forces.)
J =	1.0	

BOLTS IN TENSION

<u>Tension Demand</u>

Anchor spacing =	2.67 ft	
Trib. Wall Height =	7.75 ft	
A_p =	20.7 ft^2	
k_a =	1.40	(minimum of 2.0 and 1 + 40ft/100ft)
k_h =	1.0	(1.0 for flexible diaphragms)
F_p =	1.11 kips	(Maximum of Eq. 7-9 and 7-10)
Q_{UF} =	1.11 kips	(Per Eq. 7-35, considering $Q_E = F_p$)

<u>Tension Capacity</u>

Tension Capacity is checked using Hilti Profis ®. See following pages.

Anchorage Check with Hilti PROFIS®

<u>Connection demand</u>

W_{DL} =	37 psf	
W_{LL} =	20 psf	
Trib. Area =	8.9 ft^2	Anchor spacing x 6.67ft / 2
Shear due to gravity =	0.41 kips	(Considerind load combination 1.1DL + 0.275LL)
Tension force =	1.11 kips	(Tension force equals Q_{UF})
Vert. Ecc. Moment =	1.67 kips-in	(Moment due to vertical eccentricity between the bottom of the metal deck and the cast-in-anchor, 1.11 kips x 1.5in)
Plan Ecc. Moment =	0.61 kips-in	(Moment from plan eccentricity of gravity load from the centroid of the 3" wide bearing area to the face of the wall, 0.41 kips x 1.5in)
Applied Moment =	2.28 kips-in	(Combination of moments due to vertical eccentricity and due to plan eccentricity is applied in Hilti Profis input)

Tension Load	Capacity (including κ)	Demand		Utilization	
Steel strength	6,986 lb	1,827	lb	26%	
Pullout strength	6,129 lb	1,827	lb	30%	
Concrete Breakout	5,796 lb	1,827	lb	32%	
				32%	(Maximum)
Shear Load					
Steel strength	4,191 lb	410	lb	10%	
Pryout strength	15,456 lb	410	lb	3%	
Concrete edge failure	9,127 lb	410	lb	4%	
				10%	(Maximum)
Interaction	17%				

STEEL ANGLE BENDING

Angle properties:

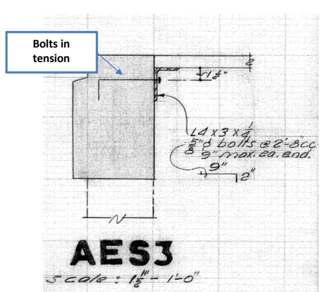
Thickness =	0.1875 in	(Using 3/16", per Det. ADS3)
Width =	32 in	(Anchorage spacing)
Fy =	37 ksi	(ASTM A36 assumed, Table 4-5 / ASCE 41-17)
Zy =	0.28 in3	(Zy = t^2 x b / 4)

<u>Capacity</u>

M_{CL} =	10 kips-in	(M_{CL} = Fy Zy)
κM_{CL} =	7.805 kips-in	

<u>Demand</u>

Tension force =	1.11 kips
Eccentricity =	1.5 in
M_{UF} =	1.67 kips-in
M_{UF} / (κM_{CL}) =	0.21
Acceptance criteria	OK



Notes:

1 -The 0.75 seismic reduction factor in ACI 318, Section 17.2.3.4.4 applied to concrete failure modes to determine the design tensile strength o f concrete is applied as the concrete failure modes have reduced capacity under cyclic loads.

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Specifier's comments:

1 Input data

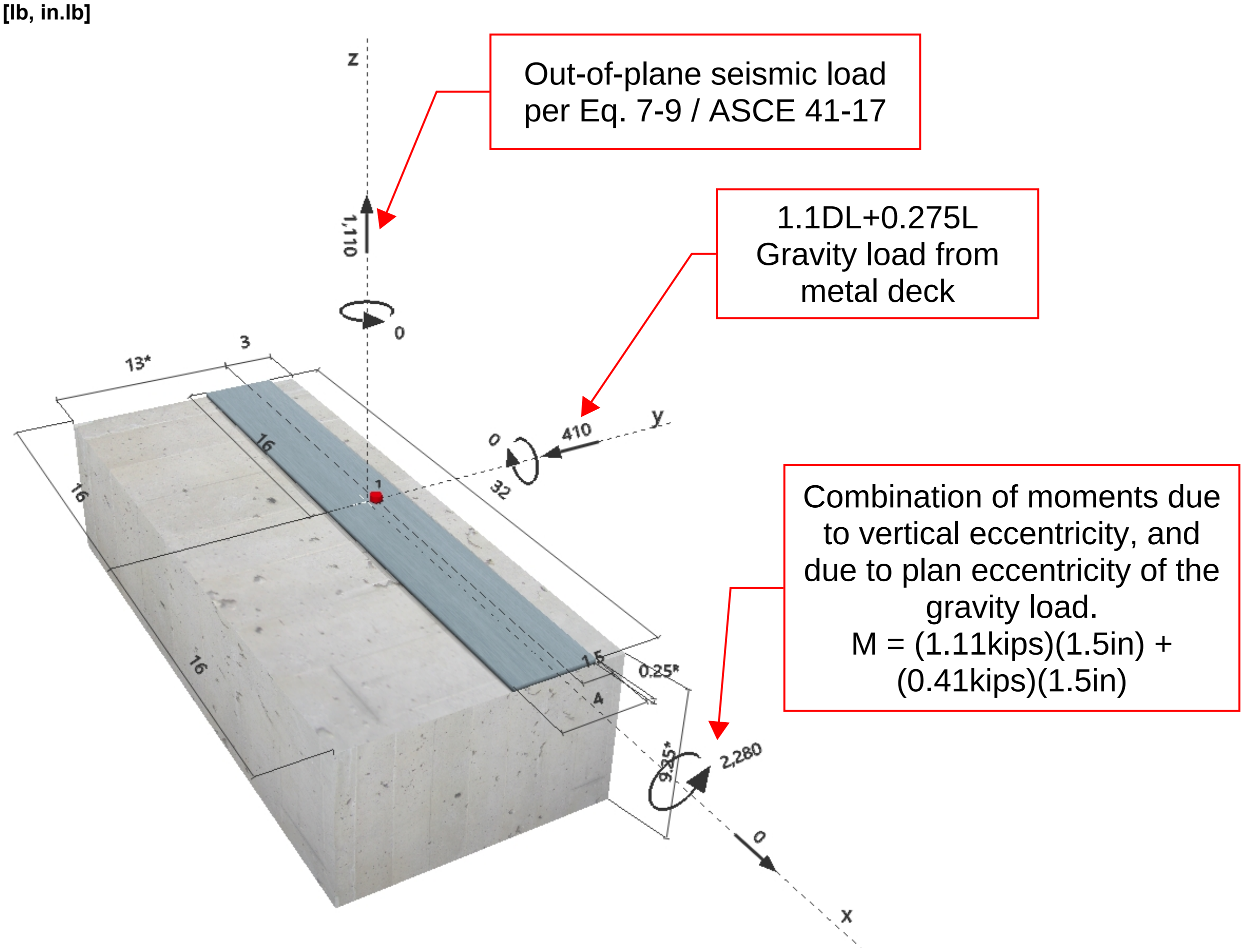
Anchor type and diameter:	**Hex Head ASTM F 1554 ~~GR. 36~~ 5/8**
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-11 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.250 in.
Anchor plate:	l_x x l_y x t = 32.000 in. x 4.000 in. x 0.250 in.; (Recommended plate thickness: not calculated
Profile:	no profile
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 9.250 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (D.3.3.4.3 (d))
	Shear load: yes (D.3.3.5.3 (c))

Lower-bound steel strength is 27 ksi, per Section 10.2.2.5 / ASCE 41-17

Lower-bound concrete strength, per Table 10-2 / ASCE 41-17

[R] - The anchor calculation is based on a rigid baseplate assumption.

Geometry [in.] & Loading [lb, in.lb]



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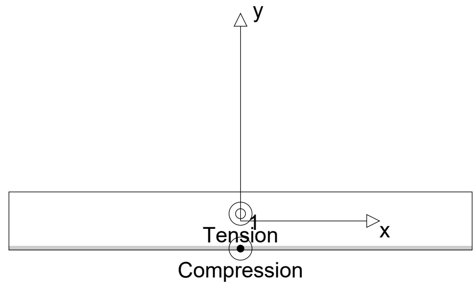
2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,827	410	0	-410

max. concrete compressive strain:	0.04 [‰]
max. concrete compressive stress:	157 [psi]
resulting tension force in (x/y)=(0.000/0.500):	1,827 [lb]
resulting compression force in (x/y)=(0.000/-1.905):	717 [lb]

Anchor forces are calculated based on the assumption of a rigid baseplate.

3 Tension load

	Load N_{ua} [lb]	Capacity κ N_n [lb]	Utilization $\beta_N = N_{ua}/\cancel{\phi}\kappa N_n$	Status
Steel Strength*	1,827	~~9,831~~ 6,986	~~19~~ 26	OK
Pullout Strength*	1,827	~~5,720~~ 6,129	~~32~~ 30	OK
Concrete Breakout Strength**	1,827	~~5,409~~ 5,796	~~34~~ 32	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-11 Eq. (D-2)

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.23	~~58,000~~ 1.5 x 27,000 psi = 40,500 psi

Calculations

N_{sa} [lb]
~~13,108~~ 9,315 lb

Results

N_{sa} [lb]	ϕ_{steel}	κ	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
~~13,108~~ 9,315 lb	~~0.750~~ 1.0	0.75	~~9,831~~ 6,986 lb	1,827

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3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-11 Eq. (D-13)

$N_p = 8 A_{brg} f'_c$ ACI 318-11 Eq. (D-14)

$\phi\ N_{pN} \geq N_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.45	1.000	3,000

Calculations

N_p [lb]
10,896

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	κ	$\phi_{nonductile}$	κ N_{pn} [lb]	N_{ua} [lb]
10,896	~~0.700~~ 1.0	0.750	0.75	1.000	~~5,720~~ 6,129	1,827

3.3 Concrete Breakout Strength

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-11 Eq. (D-3)

$\phi\ N_{cb} \geq N_{ua}$ ACI 318-11 Table D.4.1.1

A_{Nc} see ACI 318-11, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-11 Eq. (D-5)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-11 Eq. (D-8)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-11 Eq. (D-10)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-11 Eq. (D-12)

$N_b = k_c\ \lambda_a \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-11 Eq. (D-6)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
6.000	0.000	0.000	3.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
216.00	324.00	1.000	1.000	0.800	1.000	19,320

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	κ	$\phi_{nonductile}$	κ N_{cb} [lb]	N_{ua} [lb]
10,304	~~0.700~~ 1.0	0.750	0.75	1.000	~~5,409~~ 5,796	1,827

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4 Shear load

	Load V_{ua} [lb]	Capacity κ ϕ V_n [lb]	Utilization β_V = V_{ua}/κ ~~ϕ~~ V_n	Status
Steel Strength*	410	~~5,112~~ 4,191	~~9~~ 10	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	410	~~14,425~~ 15,456	~~3~~ 03	OK
Concrete edge failure in direction y-**	410	~~8,519~~ 9,127	~~5~~ 04	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-11 Eq. (D-29)

$\phi\, V_{steel} \geq V_{ua}$ ACI 318-11 Table D.4.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.23	~~58,000~~ 1.5 x 27,000 psi = 40,500 psi

Calculations

V_{sa} [lb]
~~7,865~~ 5,588 lb

Results

V_{sa} [lb]	ϕ_{steel}	κ	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
~~7,865~~ 5,588 lb	~~0.650~~ 1.0	0.75	~~5,112~~ 4,191 lb	410

4.2 Pryout Strength

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b \right]$ ACI 318-11 Eq. (D-40)

$\phi\, V_{cp} \geq V_{ua}$ ACI 318-11 Table D.4.1.1

A_{Nc} see ACI 318-11, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-11 Eq. (D-5)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}} \right) \leq 1.0$ ACI 318-11 Eq. (D-8)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-11 Eq. (D-10)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-11 Eq. (D-12)

$N_b = k_c\, \lambda_a \sqrt{f_c'}\, h_{ef}^{1.5}$ ACI 318-11 Eq. (D-6)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	3.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	24	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
216.00	324.00	1.000	1.000	0.800	1.000	19,320

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	κ	$\phi_{nonductile}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
20,608	~~0.700~~ 1.0	1.000	0.75	1.000	~~14,425~~ 15,456	410

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4.3 Concrete edge failure in direction y-

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\, \psi_{c,V}\, \psi_{h,V}\, \psi_{parallel,V}\, V_b$ — ACI 318-11 Eq. (D-30)

$\phi\ V_{cb} \geq V_{ua}$ — ACI 318-11 Table D.4.1.1

A_{Vc} see ACI 318-11, Part D.6.2.1, Fig. RD.6.2.1(b)

$A_{Vc0} = 4.5\, c_{a1}^2$ — ACI 318-11 Eq. (D-32)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$ — ACI 318-11 Eq. (D-36)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-11 Eq. (D-38)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-11 Eq. (D-39)

$V_b = \left(7 \left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f_c'}\, c_{a1}^{1.5}$ — ACI 318-11 Eq. (D-33)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
10.667	16.000	0.000	1.000	9.250

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
5.000	1.000	0.625	3,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
296.00	512.00	1.000	1.000	1.315	16,005

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	κ	$\phi_{nonductile}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
12,169	~~0.700~~ 1.0	1.000	0.75	1.000	~~8,519~~ 9,127	410

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
~~0.338~~ 32	~~0.080~~ 10	5/3	~~18~~ 17	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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Phone I Fax:	|	Date:	10/21/2019
E-Mail:			

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-11 Appendix D, Part D.3.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Part D.3.3.4.3 (b), Part D.3.3.4.3 (c), or Part D.3.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Part D.3.3.5.3 (a), Part D.3.3.5.3 (b), or Part D.3.3.5.3 (c).
- Part D.3.3.4.3 (b) / part D.3.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Part D.3.3.4.3 (c) / part D.3.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Part D.3.3.4.3 (d) / part D.3.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.

~~Fastening meets the design criteria!~~

FASTENING MEETS THE TIER 1 / ASCE 41-17 CRITERIA

Flexible Diaphragm Connection Forces Per Tier 2 Procedure - Connection "C"

Tier 2 Procedure per Section 7.2.11.1 in ASCE 41-17:

7.2.11.1 Out-of-Plane Wall Anchorage to Diaphragms. Each wall shall be positively anchored to all diaphragms that provide lateral support for the wall or are vertically supported by the wall.

$$F_p = 0.4 S_{XS} k_a k_h \chi W_p \qquad (7\text{-}9)$$

$$F_{p,\min} = 0.2 k_a \chi W_p \qquad (7\text{-}10)$$

$$k_a = 1.0 + \frac{L_f}{100} \qquad (7\text{-}11)$$

$$k_h = \frac{1}{3}\left(1 + \frac{2z_a}{h_n}\right) \qquad (7\text{-}12)$$

Per Section 11.5.2 in ASCE 41-17, "anchors embedded into existing or new masonry walls shall be considered **force-controlled components."**

Per Section 7.5.2.2.2 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):

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

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;

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

Connection between Mezzanine slab and Exterior CMU Walls at North Elevation

Reference: Detail 7/S-1 in 1992 structural drawings

Design Parameters:

χ =	0.8	(Table 7-2 / ASCE 41-17)	
S_{XS} =	1.431	g	
w_p =	84	psf	
k_a =	1.0		(1.0 for rigid diaphragms)
k_h =	0.67		(0.33 x (1 + 2x(7.75 / 15.5))

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	(Per FEMA P-2006, Section 4.7.4, the factors J, C1, and C2 do not apply to Fp forces and the presumption is that there is no ductility or limiting mechanism for reducing out-of-plance forces.)
J =	1.0	

BOLTS IN TENSION

Tension Demand

Anchor spacing =	1.33	ft	
Trib. Wall Height =	7.8	ft	
A_p =	10.3	ft^2	
F_p =	0.26	kips	(Maximum of Eq. 7-9 and 7-10)
Q_{UF} =	0.26	kips	(Per Eq. 7-35, considering $Q_E = F_p$)
Q_{UF} with moment contrib =	0.43	kips	(Tension demand including moment due to eccentricity on out-of-plane tension is derived from Hilti Profis. See following pages)

Tension Capacity

$Q_{ICC\ report}$ =	1.274	kips	(See Note 2)
ICC report Factor of Safety =	5		(See Note 2)
Embedment Factor =	0.7901		(See Note 3)
ES to LB conversion Factor =	0.77		(See Note 4)
Q_{CL} =	3.87	kips	(See Note 5)
κQ_{CL} =	2.9	kips	
Q_{UF} / (κQ_{CL}) =	0.15		(Q_{UF} includes moment contribution)
Acceptance criteria	OK		

BOLTS IN SHEAR

Shear Demand

Anchor spacing =	1.0	ft	
Trib. Wall Height =	7.8	ft	
A_p =	7.8	ft^2	
F_p =	0.20	kips	(Maximum of Eq. 7-9 and 7-10)
Q_{UF} =	0.20	kips	(Per Eq. 7-35, considering $Q_E = F_p$)

Shear Capacity

No. bolts =	1		
D_{bolt} =	0.5	in	
A_{bolt} =	0.196	in^2	
Fy =	36	ksi	(ASTM A36 assumed, Table 4-5 in ASCE 41-17 for default yield strength)
Q_{CL} =	4.2	kips	(Lower-bound shear capacity, Q_{CL} = 0.6 x No. bolts x Fy x A_{bolt})
κQ_{CL} =	3.2	kips	
Q_{UF} / (κQ_{CL}) =	0.06		
Acceptance criteria	OK		

STEEL ANGLE BENDING

Angle properties:

Thickness =	0.375	in	(Using 3/16", per Det. ADS3)
Width =	16	in	(Anchor spacing)
Fy =	37	ksi	(ASTM A36 assumed, Table 4-5 / ASCE 41-17)
Zy =	0.56	in3	(Zy = t^2 x b / 4)

Capacity

M_{CL} =	21	kips-in	(M_{CL} = Fy Zy)
κM_{CL} =	15.6	kips-in	

Demand

Tension force =	0.3	kips
Eccentricity =	1.5	in
M_{UF} =	0.4	kips-in
M_{UF} / (κM_{CL}) =	0.03	
Acceptance criteria	OK	

Notes:

1 -The mezzanine slab was constructed in 1978 after the original construction in 1961. This steel angle was added during the 1992 alterations. As such, the steel angle connection does not resist vertical gravity loads.

2- The table of the ICC-ES Evaluation report with the allowable tension loads for the ITW red head trubolt anchor specified in the structural drawings is included in the next page. The factor of safety is specified on footnote (7).

3- The 4" embedment of the 5/8"ϕ bolt is specified on the notes of Sheet S2 in the 1992 drawings.

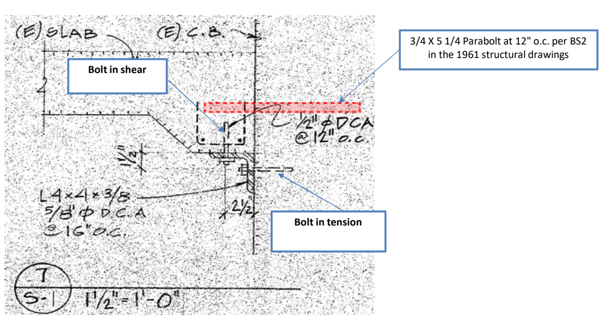
4 - The embedment factor is derived from the equation 6-5 in the TMS 402/602-16. This factor reduces the capacity of the bolt considering the actual embedment of 4" instead of the 4.5" in the ICC-ES report.

$$\frac{A_{pt,current}}{A_{pt,ICC\ report}} = \frac{\pi \times l^2{}_{b,current}}{\pi \times l^2{}_{b,ICC\ report}} = \frac{(4in)^2}{(4.5in)^2} = 0.79$$

5 - The expected-strength to lower-bound convertion factor is calculated as the minimum value obtained from Table 9-3 for steel, and Table 11-1 in the ASCE 41-17, i.e. the minimum value of 1.1^{-1} and 1.3^{-1}.

6 - The lower-bound tension capacity of the anchor bolts is computed using the following equation:

$$Q_{CL} = (Embedment\ factor) \times (ES\ to\ LBconversion\ factor) \times (ICC\ Factor\ of\ Safety) \times Q_{ICC\ report}$$



the 3/8 in. sleeve anchor only.

9. Testing should occur 24 hours minimum after installation of the subject anchors.

~~DRILLED-IN ANCHOR BOLTS TO BE~~

DCA = [redacted] WEDGE ANCHORS TRUBOLT

TCA = [redacted] DYNASET

OR RAWL STEEL DROP-IN

WHEN INSTALLING DRILLED-IN ANCHORS IN EXISTING NON-PRESTRESSED REINFORCED CONC. USE CARE AND CAUTION TO AVOID CUTTING OR DAMAGING THE EXISTING REBARS. MAINTAIN A MINIMUM CLEARANCE OF ONE INCH BETWEEN THE REINFORCING STEEL AND THE DRILLED-IN ANCHOR.

BOLT ϕ	BASE MATERIAL	EMBEDMENT (IN")	TEST IN CMU
1/4	CONC	1½	
3/8	"	3	
1/2	"	4	
1/4	CMU	2⅜	208
3/8	"	2¾	464
1/2	"	3½	720
5/8	"	4"	992

PROOF TEST 50% OF ALL ANCHORS IN TENSION IF ONE ANCHOR FAILS, THEN TEST ALL OTHER ANCHORS

EXCERPT FROM ICC ESR-4058 REPORT FOR TRUBOLT POST INSTALLED ANCHORS

TABLE 1—ALLOWALE TENSION AND SHEAR LOADS FOR THE TRUBOLT+ WEDGE ANCHORS INSTALLED IN FULLY GROUTED CMU CONSTRUCTION[1,2,3]

Anchors Installed in the Face of Fully Grouted CMU Construction								
Anchor Diameter (inches)	Embedment Depth[4] (inches)	Installation Torque (ft-lbf)	Anchor Location[5,6] (inches)				Allowable Loads For Anchors Installed At Distances ≥ Critical Edge Distance, C_{cr}, And Critical Spacing, S_{cr} (lbf)	
			Edge/End Distance		Spacing			
			Critical C_{cr}	Minimum C_{min}	Critical S_{cr}	Minimum S_{min}	Tensions[5,7]	Shears[5,7]
1/4	1 1/8	5	12	4	8	4	183	273
	2 1/4	8					311	
3/8	1 5/8	15	12	4	8	4	276	638
	2 3/4	25					552	
1/2	2 1/4	45	12	4	8	4	550	907
	3 3/4						706	985
5/8	2 3/4	70	12	4	8	4	816	1600
	4 1/2						1274	
3/4	3 1/4	100	12	4	8	4	893	1615
	5						1195	

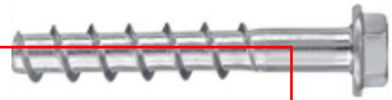
Anchors Installed in the Top of Fully Grouted CMU Construction										
Anchor Diameter (inches)	Embedment Depth[4] (inches)	Installation Torque (ft-lbf)	Anchor Locations (inches)					Allowable Loads For Anchors Installed At Distances > Critical End Distance, $C_{cr\text{-}End}$, Critical Spacing, S_{cr}, Minimum Edge Distance, C_{min}, (ibf)		
			End Distance		Spacing		Edge Distance	Tension[7,8]	Shear	
			Critical $C_{cr\text{-}End}$	Minimum $C_{min\text{-}End}$	Critical S_{cr}	Minimum S_{min}	Minimum C_{min}		⊥ To Wall[7,8]	// To Wall[7,8]
3/8	2 1/2	25	12	4	8	4	1 3/4	669	233	562
1/2	3	45	12	4	8	4	2 1/4	1021	289	871
5/8	4 1/2	70	12	4	8	4	2 3/4	1203	466	1134

For **SI**: 1 inch = 25.4 mm; 1 lbf = 0.0044 kN, 1 ksi = 6.894 MPa.

[1]Tabulated loads are for anchors installed in fully grouted CMU wall construction consisting of materials in compliance with Section 3.2 of this report. The specified compressive strength of masonry, f'_m, is minimum 2,000 psi (13.8 MPa) at 28 days.
[2]Allowable loads are based on periodic special inspection being provided during anchor installation. Special inspection requirements must comply with Section 4.3 of this report.
[3]Allowable loads may be increased in accordance with Section 5.3 and Table 3 of this report, where permitted by the IBC or its referenced standards.
[4]Embedment depth is measured from the outside face of the masonry to the end of the mandrel.
[5]Critical and minimum edge distances and critical and minimum spacing must comply with this table. Refer to Figure 2. Critical edge distance and critical spacing are valid for anchors resisting the tabulated allowable tension or shear loads. Table 2 tabulates allowable tension and shear load reduction factors for anchors installed between critical and minimum edge distances and spacing.
[6]Figure 2 illustrates permitted and prohibited anchor installation locations. Section 4.2 of this report provides additional installation details.
[7]Tabulated allowable loads are based on a factor of safety of five (5).
[8]Critical and minimum end distance, critical and minimum spacing, and minimum edge distance must comply with this table and Figure 3. Critical end distance and critical spacing are valid for anchors resisting the tabulated allowable tension or shear loads. Table 2 for allowable tension and shear load reduction factors for anchors installed between critical and minimum end distances and spacing.

Specifier's comments:

1 Input data

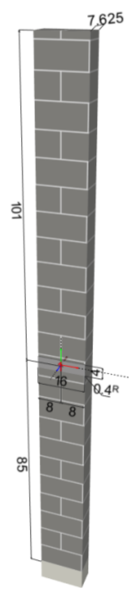


Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 5/8 (5)**
Item number:	418080 KH-EZ 5/8"x5 1/2"
Effective embedment depth:	h_{ef} = 5.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3056
Issued I Valid:	7/1/2019 | 10/1/2019
Proof:	Design Method ASD Masonry
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.400 in.
Anchor plate[R] :	l_x x l_y x t = 16.000 in. x 4.000 in. x 0.400 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.;
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
	Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	no

PROFIS IS USED ONLY TO COMPUTE THE TENSION FORCE DUE TO PRYING. THE ANCHOR DESIGN IS NOT USED.

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.]



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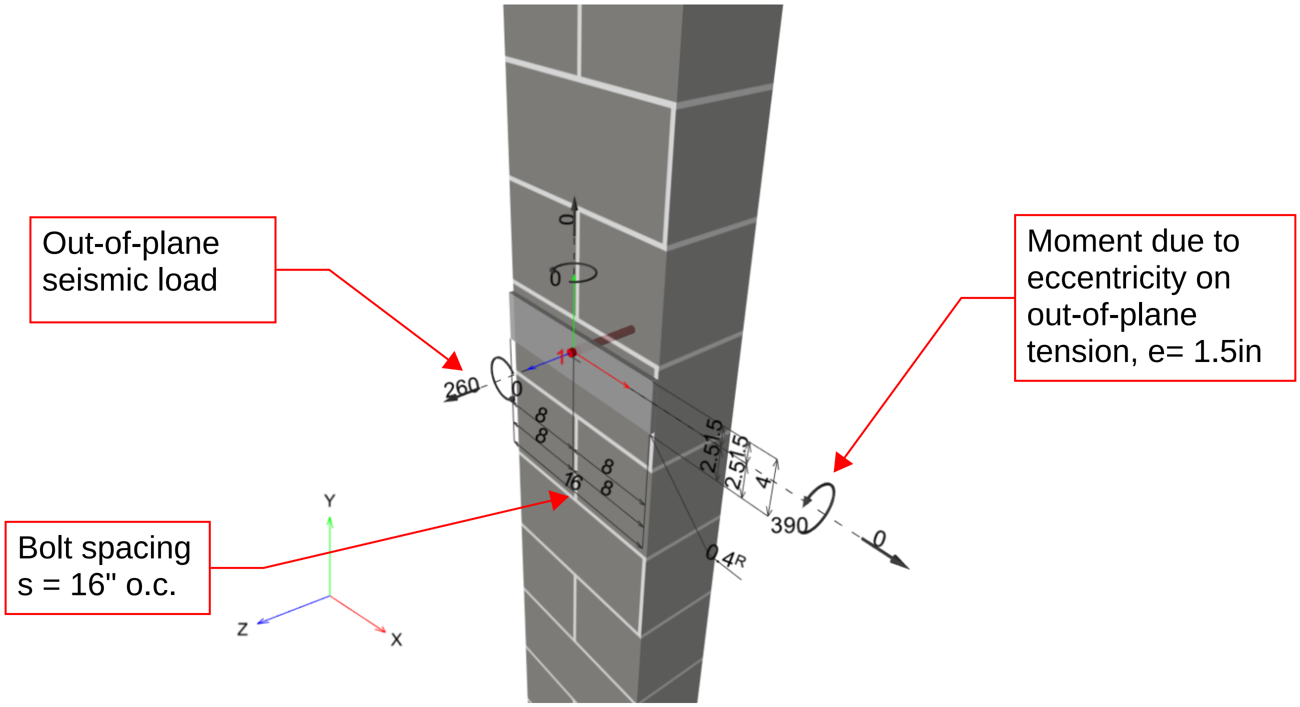
Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Oct 17, 2019	Date:	10/22/2019
Fastening point:			

Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 260; V_x = 0; V_y = 0; M_x = 390; M_y = 0; M_z = 0;	no	32

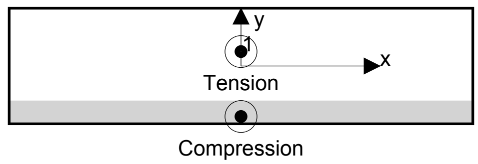
2 Load case/Resulting anchor forces

Load case: Service loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	434	0	0	0



max. compressive strain: 0.02 [‰]
max. compressive stress: 29 [psi]
resulting tension force in (x/y)=(0.000/0.500): 434 [lb]
resulting compression force in (x/y)=(0.000/-1.747): 174 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

Tension demand including moment contribution