UCSF

Rating form completed by: MAFFEI STRUCTURAL ENGINEERING
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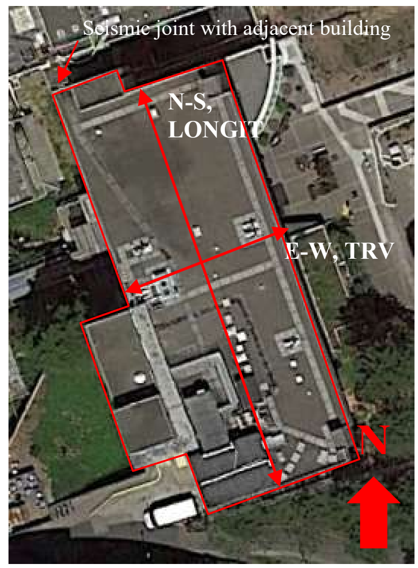
Text in green is to be part of UCSF building database and may be part of UCOP database

UCSF building seismic ratings

School of Nursing, University of California San Francisco

CAAN #2410
2 Koret Way, San Francisco, CA 94131
UCSF Campus: Parnassus

DATE: 2020-06-26



Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	VI	Findings based on drawing review and Tier 1 and Tier 3 nonlinear evaluation[1]
Rating basis	Tier 1 and Tier 3 NL in progress	ASCE 41-17
Date of rating	2018	
Recommended UCSF priority category for retrofit	Priority A	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application for modification
Ballpark total construction cost to retrofit to IV rating[2]	Very High (> $400/sf)	See recommendations on further evaluation and retrofit.
Is 2018-2019 rating required by UCOP?	Yes	Building previously rated IV but does not have a fully documented previous review
Further evaluation recommended?	Tier 3 NLRHA	Further evaluation would help understanding of the severity of deficiencies (but would be unlikely to improve rating)

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

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

Tier 3 nonlinear evaluation

Aspects of this report are superseded by the Tier 3 Nonlinear evaluation in progress by MSE. Key revisions based on the nonlinear findings are shown in this report in orange font. The Seismic Performance Level Rating is revised to VI. The nonlinear findings will provide revised information on the significance of the potential deficiencies.

Building information used in this evaluation

- Structural drawings by Ephraim G. Hirsch, "School of Nursing Building, University of California San Francisco Medical Center," dated 1972-03-31 (17 sheets).

Additional building information known to exist

- None

Scope for completing this form

Reviewed structural drawings for original construction and carried out ASCE 41-17 Tier 1 evaluation. Made only brief exterior site visit. Did not evaluate non-structural life-safety hazards, but discussed with SRC members and UCSF staff who know the building.

Brief description of structure

The building has an area of approximately 100,000 square feet. It was designed in the early 1970s by structural engineer Ephraim Hirsch and architect George Matsumoto and Associates. Construction was completed in 1972. It has six stories above grade plus penthouses over the service core and at the southwest corner. The main floor plate is rectangular in plan 180 ft north-south by 71 ft east-west. A service core, 64 ft north-south by 21 ft east-west, is located at the west side of the building.

Identification of levels: The building has a partial basement below the service core, designated as Level 1 on the original structural drawings. The building entrance occurs at grade and is designated as Level 2. Over most of the footprint, the building has 7 stories, with the Roof occurring at Level 8.

The natural grade slopes downward to the north. The cut and retaining walls at the north end of the building changed the natural grade so that the current site has a minimal slope down to the north.

Foundation system: The site is located at the foot of the slope of Mount Sutro, and the south end of the building is embedded into the hillside. At this end, approximately 35 ft of soil height is retained by a tieback wall, which is structurally separated from the building. Shallow foundations bearing on rock are used at the south end of the building. The bedrock slopes downward to the north, and north end of the building is supported on 6-ft diameter drilled, cast-in-place concrete piers. The building has perimeter grade beams and a slab-on-grade.

Structural system for vertical (gravity) load: Typical floors are framed using a one-way, reinforced concrete joist and girder system (with joists spanning east-west) to support a 4½" thick conventionally reinforced slab. Girders span to reinforced concrete columns, which are typically 24" x 33".

Structural system for lateral forces: Concrete walls in the east-west direction, at typical floors located on the north and south ends of the building. Walls also surround the service core, but this appears to be seismically separated from the rest of the structure by a four-inch separation joint, at all above-grade levels. Typical walls are 10" thick with #4 @16" each face vertical and horizontal reinforcement, and #7 vertical bars at wall boundaries. Columns and girders form moment frames in the North-South direction that are expected to participate along with walls in resisting lateral forces.

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

Identified seismic deficiencies of the building include the following, in decreasing order of severity:

Potential deficiency	Description
Shear-critical columns	All columns are moment frame columns and are shear-governed because of heavy vertical reinforcement (including #18, #14, and #11 bars, which are spliced with butt welds) and inadequate tie spacing and shear reinforcement. Column shear demand exceeds the Tier 1 check by a factor of 4.
Story concentration	The moment frames have strong-column/weak-beam proportions close to code limits, which may allow a story mechanism of concentrated deformation.
Short columns	There is a partial height wall at the bottom story (above Level 2) on Grid B that creates short columns at Grid intersections B6, B7, and B8. Although there is a two-bay wall in line with the short columns, the columns are still vulnerable to shear failure which in an extreme case could lead to gravity collapse. The columns have #4@4" ties which could help reduce the collapse risk.
B/C joints	Beam-column joints at many columns have ties at 12" o.c. which could allow joint failure to govern.
Shear in moment frame girders	The moment frame girders have waffle-slab voids in the middle of the girder section in the middle half of the clear span. This reduces the shear capacity of the girders for seismic demand. This should be evaluated to establish whether the girders are susceptible to shear failure.
Discontinuous wall	Part of the Line 1 wall is discontinuous below Level 4. Below Level 4, the wall shifts from Line 1 to Line 4. This contributes to the plan torsion by limiting the overturning stiffness and strength of the wall on Line 1. East-west walls have adequate wall area and shear stress of approximately $4\sqrt{f'c}$.
Shear-critical walls	Given the low horizontal reinforcement ratio of the walls, they may exhibit shear-critical behavior.
Pounding	The four-inch separation between the elevator and service core and the rest of the building is insufficient to prevent pounding between the two structures in the larger earthquake motions. The pounding could induce plan torsion into the response and would create somewhat increased forces because of impact.
Floor diaphragm	At Level 3, there is a large floor opening at the northeast corner that affects the floor diaphragm in-plane behavior.

The deficiencies, in particular the first three, are likely to compromise seismic performance. Columns are vulnerable to shear failure that could lead to collapse. The deformation in columns may be exacerbated by a story concentration and shear-governed behavior in concrete walls. The presence of three short columns increases vulnerability. The column on Line B1 may be vulnerable because it supports demands from a wall above that discontinues (although the wall above in this area could help support the floors above in the event of heavy column damage).

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

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

None known by those familiar with the building. Masonry or heavy plaster not expected based on knowledge of the building and 1970s date of construction. We did not visit building interior.

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

Recommendations for further evaluation or retrofit

We recommend that the University perform a more detailed seismic evaluation, preferably Tier 3 NLRHA, to establish the severity of the deficiencies and whether the building should be rated V or VI. If desired, further evaluation could also define a conceptual scope of retrofitting for this building. Further evaluation would be unlikely to show the building as better than V because of the moment frame column vulnerability. We recommend a nonlinear response-history analysis that accounts for the behaviors related to the deficiencies, identifying the potential nonlinear mechanisms, and including the column and wall shear behavior, joint shear behavior, force transfer at the wall on Line F-1, other floor diaphragm transfers, and demand on reinforcement butt weld splices. Applicable retrofit measures may include fiber wrapping of columns and/or adding concrete walls or similar elements to balance the plan torsion.

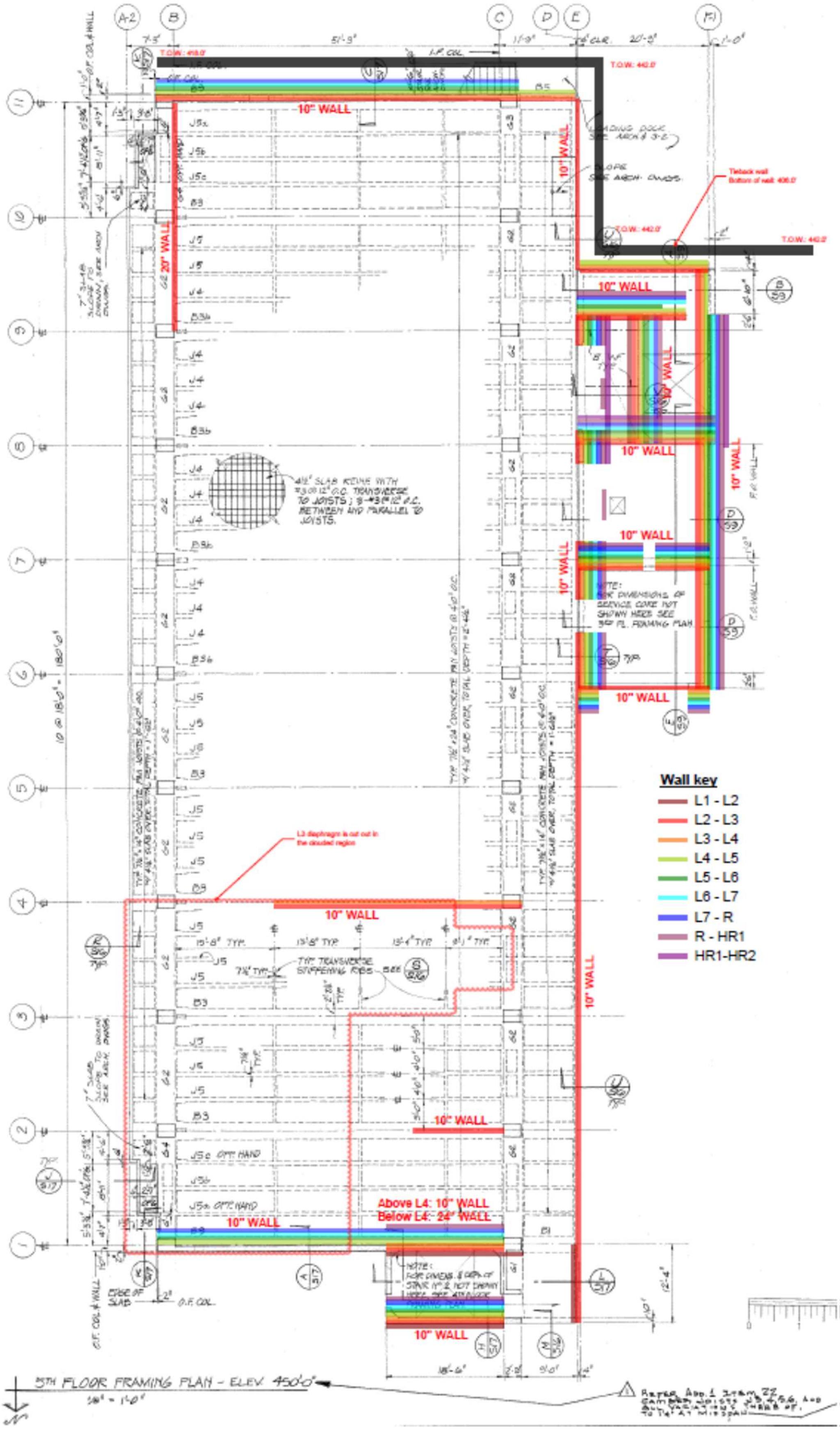
Peer review comments on rating

The reviewing structural members of the UCSF Seismic Review Committee (SRC) agree that the rating is at best near the low end of V (Poor), with one member preferring a rating of VI (Very Poor). The SRC agree that further study, such as Tier 3 nonlinear, is important to define a more specific scope of retrofitting for this building, and that such further evaluation would be unlikely to show a rating above V (Poor). The Tier 3 nonlinear evaluation in progress indicates a rating of VI. The Tier 3 Nonlinear evaluation is currently under peer review.

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

Additional building data	Entry	Notes
Latitude	37.762482	
Longitude	-122.45900	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	6	
Number of stories (basements) below lowest perimeter grade	1	Partial basements at core, stairs, elevator
Building occupiable area (OGSF)	91287	From UCOP spreadsheet
Risk Category per 2016 CBC 1604.5	III	Occupant load > 500 (campus to confirm) and contains educational occupancy above 12th grade
Building structural height, h_n	78 ft	Structural height defined per ASCE 7-16 Section 11.2
Coefficient for period, C_t	0.02	Estimated using ASCE 41-17 equation 4-4 and 7-18
Coefficient for period, ⍰	0.75	Estimated using ASCE 41-17 equation 4-4 and 7-18
Site data		
975 yr hazard parameters S_s, S_1	1.548, 0.611	
Site class	C	
Site class basis	Geotech Parameters	UCSF Group 1 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
Site parameters F_a, F_v	1.2, 1.4	Per ASCE 7-16 Tables 11.4-1 and 11.4-2
Ground motion parameters S_{cs}, S_{c1}	1.858, 0.855	UCSF Group 1 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
S_a at building period	0.67	
Site V_{s30}	570 m/s	
V_{s30} basis	Estimated	UCSF Group 1 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
Liquefaction potential	No	
Liquefaction assessment basis	Study	UCSF Group 1 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
Landslide potential	No	
Landslide assessment basis	Study	UCSF Group 1 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
Active fault-rupture identified at site?	No	
Fault rupture assessment basis	Study	UCSF Group 1 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
Site-specific ground motion study?	No	
Applicable code		
Applicable code or approx. date of original construction	Built: 1972 Code: 1967 UBC	Code identified on Sheet S-1
Applicable code for partial retrofit	None	No partial retrofit known

Applicable code for full retrofit	None	No full retrofit known
Model building data		
Model building type North-South	C1 Conc. Moment frame + C2 Conc. wall	
Model building type East-West	C2 Conc. wall	
FEMA P-154 score	N/A	Not included here because we performed ASCE 41 Tier 1 evaluation.
Previous ratings		
Most recent rating	IV	In spreadsheet. Basis for rating is unknown
Date of most recent rating	-	Rating date is unknown
2nd most recent rating	Good	In spreadsheet. Basis for rating is unknown
Date of 2nd most recent rating	-	Rating date is unknown
3rd most recent rating	-	
Date of 3rd most recent rating	-	
Appendices		
ASCE 41 Tier 1 checklist included here?	Yes	Refer to attached checklist file


10" WALL
20" WALL
Tieback wall
Bottom of wall: 406.0'
L3 diaphragm is cut out in the clouded region
Above L4: 10" WALL
Below L4: 24" WALL
Wall key
L1 - L2
L2 - L3
L3 - L4
L4 - L5
L5 - L6
L6 - L7
L7 - R
R - HR1
HR1-HR2
5TH FLOOR FRAMING PLAN - ELEV 450'-0"

ASCE 41-13 - LS Basic.docx

LOW SEISMICITY

BUILDING SYSTEMS - *GENERAL*

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	LOAD PATH: The structure shall contain 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)	
☒	☐	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1a, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)	Seismic joint at connection to Clinical Sciences building is not clearly shown on the structural drawings. Seismic joint is assumed adequate for this evaluation but should be verified.
☐	☒	☐	☐	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)	Portion of L3 diaphragm at north end of building is connected to only one wall in the north-south direction.

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	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)	Determined qualitatively by drawing review.
☒	☐	☐	☐	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)	Determined qualitatively by drawing review.
☐	☒	☐	☐	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)	Line 1 wall discontinues at Level 3 (shifts to Line 4).
☐	☒	☐	☐	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)	Line 1 wall discontinues at Level 3 (shifts to Line 4).
☒	☐	☐	☐	MASS: There is no change in effective mass 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)	Determined qualitatively by drawing review.

C	NC	N/A	U	Description	Comments
☐	☒	☐	☐	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)	Building is nonconforming for north-south direction. May be nonconforming in east-west direction due to Line 1 wall offset at Level 3.

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

GEOLOGIC SITE HAZARD

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	LIQUIFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)	Based on CGS North Quadrangle map (http://maps.conservation.ca.gov/cgs/informationwarehouse).
☐	☒	☐	☐	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be 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)	Building is on the border of a landslide investigation area per the CGS North Quadrangle map.
☒	☐	☐	☐	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)	Based on USGS mapped faults.

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

FOUNDATION CONFIGURATION

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	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)	53 ft / 78 ft = 0.68 $0.6S_a$ = 0.6(0.67) = 0.40 OK
☒	☐	☐	☐	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)	Note that foundations are not well tied in the east-west direction, but they are laterally restrained by competent soil.

Building Name:	UCSF School of Nursing		Date:	29 November 2018	
Building Address:	2 Koret Way, San Francisco, CA 94131		Page:	1 of 2	
Job Number:	j0149	Job Name: UCSF building seismic ratings	By: LAB	Checked:	KCT

ASCE 41-13 - LS Type C2-C2a.docx

Low And Moderate Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5.2.5.1)	The building has complete frames except at the service core.
☒	☐	☐	☐	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)	The building meets a strict interpretation of this requirement (more than 2 lines) but has little actual redundancy.
☐	☒	☐	☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 lb/in.^{2} or $2\sqrt{f'_{ci}}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)	East-west walls satisfy the requirement. North-south walls fail. The Level 2 north-south wall has an unreduced shear stress (i.e. ignoring the Ms factor) of ~19 $\sqrt{f'_c}$. Considering the factor Ms = 4, the D/C ratio = 2.4.
☒	☐	☐	☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)	All walls meet criteria.

Connections

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)	Diaphragms are not flexible.
☐	☒	☐	☐	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)	Collectors are present on Lines 1, 11, E. No collector and limited diaphragm connection for primary north-south wall on Line F-1.
☒	☐	☐	☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)	Foundation dowels match wall reinforcement size and spacing.

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

Seismic-Force-Resisting System

C	NC	N/A	U	Description	Comments
☐	☒	☐	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)	Typical columns are shear critical.

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	FLAT SLABS: Flat slabs or plates not part of the seismic- force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)	Beam and joist floor system. Also, girders have considerable reinforcement continuous through columns.
☐	☒	☐	☐	COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than *d*/2 and are anchored into the confined core of the beam with hooks of 135 degrees or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)	Coupling beam at E/8 – E/9 does not have closed hoops (although hoops are spaced at ~d/4).

Connections

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)	Pile caps are not used, but piles are well-doweled into grade beams.

Diaphragms (Flexible Or Stiff)

C	NC	N/A	U	Description	Comments
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)	Conforming, although there is a large diaphragm opening at Level 3.
☐	☒	☐	☐	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)	The opening at the Line 4 wall and Level 3 is more than

Flexible Diaphragms

C	NC	N/A	U	Description	Comments
☐	☐	☒	☐	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)	Diaphragm is not flexible.
☐	☐	☒	☐	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)	Diaphragm is not flexible.
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)	Diaphragm is not flexible.
☐	☐	☒	☐	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)	Diaphragm is not flexible.
☐	☐	☒	☐	OTHER DIAPHRAGMS: The diaphragm does 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)	Diaphragm is not flexible.

UC Campus:	San Francisco		Date:	29 November 2018		
Building CAAN:	2410	Auxiliary CAAN:	By Firm:	Maffei Structural Engineering		
Building Name:	UCSF School of Nursing		Initials:	*LAB*	Checked:	*KCT*
Building Address:	j0149		Page:	1	of	4

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type C1

Low Seismicity

Seismic-Force-Resisting System

	Description
C NC N/A U ☐ ☐ ☐ ☐	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1) **Comments:**
C NC N/A U ☐ ☐ ☒ ☐	COLUMN AXIAL STRESS CHECK: The axial stress caused by unfactored gravity loads in columns subjected to overturning forces because of seismic demands is less than 0.20*f'c*. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than 0.30*f'c*. (Commentary: Sec. A.3.1.4.2. Tier 2: Sec. 5.5.2.1.3) **Comments: Gravity stress on typical interior columns exceeds 0.2*f'c*.**

Connections

	Description
C NC N/A U ☐ ☐ ☐ ☐	CONCRETE COLUMNS: All concrete columns are doweled into the foundation with a minimum of four bars. (Commentary: Sec. A.5.3.2. Tier 2: Sec. 5.7.3.1) **Comments:**

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

Seismic-Force-Resisting System

	Description
C NC N/A U ☐ ☐ ☐ ☐	REDUNDANCY: The number of bays of moment frames in each line is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1) **Comments:**

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

UC Campus:	San Francisco			Date:	29 November 2018		
Building CAAN:	2410	Auxiliary CAAN:		By Firm:	Maffei Structural Engineering		
Building Name:	UCSF School of Nursing			Initials:	*LAB*	Checked:	*KCT*
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Collapse Prevention Structural Checklist For Building Type C1

C NC N/A U ☐ ☐ ☐ ☐	COLUMN SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.4.3.2, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'c}$. (Commentary: Sec. A.3.1.4.1. Tier 2: Sec. 5.5.2.1.4) **Comments: Column shear exceeds the defined limit by approximately a factor of 4.**
C NC N/A U ☐ ☐ ☐ ☐	FLAT SLAB FRAMES: The seismic-force-resisting system is not a frame consisting of columns and a flat slab or plate without beams. (Commentary: Sec. A.3.1.4.3. Tier 2: Sec. 5.5.2.3.1) **Comments:**

High Seismicity (Complete The Following Items In Addition To The Items For Low And Moderate Seismicity)	
Seismic-Force-Resisting System	
	Description
C NC N/A U ☐ ☐ ☐ ☐	PRESTRESSED FRAME ELEMENTS: The seismic-force-resisting frames do not include any prestressed or post-tensioned elements where the average prestress exceeds the lesser of 700 lb/in.^{2} (4.83 MPa) or $f'_c/6$ at potential hinge locations. The average prestress is calculated in accordance with the Quick Check procedure of Section 4.4.3.8. (Commentary: Sec. A.3.1.4.4. Tier 2: Sec. 5.5.2.3.2) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	CAPTIVE COLUMNS: There are no columns at a level with height/depth ratios less than 50% of the nominal height/depth ratio of the typical columns at that level. (Commentary: Sec. A.3.1.4.5. Tier 2: Sec. 5.5.2.3.3) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	NO SHEAR FAILURES: The shear capacity of frame members is able to develop the moment capacity at the ends of the members. (Commentary: Sec. A.3.1.4.6. Tier 2: Sec. 5.5.2.3.4) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	STRONG COLUMN—WEAK BEAM: The sum of the moment capacity of the columns is 20% greater than that of the beams at frame joints. (Commentary: Sec. A.3.1.4.7. Tier 2: Sec. 5.5.2.1.5) **Comments:**

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C NC N/A U ☐ ☐ ☐ ☐	BEAM BARS: At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment are continuous throughout the length of the members. (Commentary: A.3.1.4.8. Tier 2: Sec. 5.5.2.3.5) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	COLUMN-BAR SPLICES: All column-bar lap splice lengths are greater than $35d_b$ and are enclosed by ties spaced at or less than $8d_b$. Alternatively, column bars are spliced with mechanical couplers with a capacity of at least 1.25 times the nominal yield strength of the spliced bar. (Commentary: Sec. A.3.1.4.9. Tier 2: Sec. 5.5.2.3.6) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	BEAM-BAR SPLICES: The lap splices or mechanical couplers for longitudinal beam reinforcing are not located within $l_b/4$ of the joints and are not located in the vicinity of potential plastic hinge locations. (Commentary: Sec. A.3.1.4.10. Tier 2: Sec. 5.5.2.3.6) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	COLUMN-TIE SPACING: Frame columns have ties spaced at or less than $d/4$ throughout their length and at or less than $8d_b$ at all potential plastic hinge locations. (Commentary: Sec. A.3.1.4.11. Tier 2: Sec. 5.5.2.3.7) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	STIRRUP SPACING: All beams have stirrups spaced at or less than $d/2$ throughout their length. At potential plastic hinge locations, stirrups are spaced at or less than the minimum of $8d_b$ or $d/4$. (Commentary: Sec. A.3.1.4.12. Tier 2: Sec. 5.5.2.3.7) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	JOINT TRANSVERSE REINFORCING: Beam–column joints have ties spaced at or less than $8d_b$. (Commentary: Sec. A.3.1.4.13. Tier 2: Sec. 5.5.2.3.8) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2) **Comments:**
C NC N/A U ☐ ☐ ☐ ☐	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3) **Comments:**

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Diaphragms

	Description
C NC N/A U ☐ ☐ ☐ ☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1) **Comments:**

Connections

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

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Project:__________________
Subject:__________________
By:__________________
Date:__________________

SEISMIC EVALUATION OF EXISTING BUILDINGS - TIER 1 SCREENING

ASCE 41-13 Chapter 4

General

Architect	George Matsumoto & Associates Architects-Planners		
Structural Engineer	Ephraim G. Hirsch		
Location	2 Koret Way, San Francisco, CA 94131		
Design date	1971		
Latitude	37.762482		(Google Earth)
Longitude	-122.45900		"
Stories above grade	9		

Seismic parameters

*This needs to be verified based on occupant load. It is assumed to exceed 50

Risk Category	III*	(ASCE 7-10 Table 1.5-1, IBC 2012 Table 1604.5, CBC 2013 Table 1604.5)	
Site Class	B	https://earthquake.usgs.gov/hazards/urban/sfbay/soiltyp	(2.4.1.6)
Liquefaction hazard	Low	http://geomaps.wr.usgs.gov/sfgeo/liquefaction/susceptil	(4.3.4)
S_{DS}	1.134	https://seismicmaps.org/ http://earthquake.usg	(2.4.1.1, 2.5)
S_{D1}	0.523	Based on MCEr, used for level of seismicity	"
S_{XS}	0.907	Based on "Seismic Hazard Level," used for *V*	(2.4.1)
S_{X1}	0.352	"	"

Scope

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

Material properties

				Notes	
Concrete	f'_c	3000	psi	Specified on drawings, NWC	(4.2.3, Table 4-2)
Reinf.	f_y	40	ksi	Column bars are "hard" grade	(4.2.3, Table 4-3)
Steel	F_y	N/A	ksi	N/A	(4.2.3, Table 4-4, 4-5)

Checklists

Benchmark building	No	(Table 4-6)
Checklist(s) req'd	16.1.2LS Life Safety Basic Configuration	(Table 4-7)
	16.10LS Life Safety Structural for Building Type C2	"
	~~16.17 Nonstructural Checklist~~ (not performed)	"

Seismic forces

V	13676	kip	$V = Cs_a W$	= 0.67W	(4-1)
W	20394	kip	building weight		(4.5.2.1)
C	1.0				(Table 4-8)
S_a	0.67	g	$S_a = S_{x1}/T \le S_{XS}$		(4-4)
T	0.52	sec	$T = C_t h_n^{\beta}$		(4-5)
C_t	0.020				"
β	0.75				"
h_n	78	ft	building height		

Story Forces

Story	w	story ht	h	wh^k	F_{story}	F_{story} (4-3a)	V_{story} (4-3b)
	kip	ft	ft			kip	kip
Roof	3253.75		78	267956	0.28	3783	
7	3428.1	13.0	65	234728	0.24	3314	3783
6	3428.1	13.0	52	187261	0.19	2644	7096
5	3428.1	13.0	39	139943	0.14	1976	9740
4	3428.1	13.0	26	92825	0.10	1310	11716
3	3428.1	13.0	13	46013	0.05	650	13026
2		13.0	0				13676
Total	20394			968725	1.0	13676	

k 1.01 k = 1.0 for T < 0.5, 2.0 for T > 2.5, linear interpolation between

$F_{story} = V(wh^k)/(\Sigma wh^k)$ (4-3a)

$V_{story} = \Sigma_{above} F_{story}$ (4-3b)

Shear stress in shear walls (4-9) (4-9)

Story	$A_{w\,N\text{-}S}$ in^2	$A_{w\,E\text{-}W}$ in^2	v_{NS}^{avg} psi	v_{EW}^{avg} psi	D/C_{NS}	D/C_{EW}
Roof						
7	11280	22200	84	43	0.8	0.4
6	11280	22200	157	80	1.4	0.7
5	11280	22200	216	110	2.0	1.0
4	12360	28440	237	103	2.2	0.9
3	12360	28440	263	115	2.4	1.0
2	36480	28440	94	120	0.9	1.1
Total						

M_s 4.0 (Table 4-9)

v_{limit} 110 psi $v_{limit} = 2\sqrt{f_c'} \geq 100$ psi

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

Penthouses			
Floor Area	**2000**	**ft^2**	
hstory =	**NA**	**ft**	
Gravity Load Cases			
	Short-term	**Seismic**	
Component	**lb/ft^2**	**lb/ft^2**	**Comments**
8" concrete slab	100.0	100.0	150 lb/ft^3
Concrete walls below	26.7	26.7	~95 linear ft of wall to penthouses
MEP/sprinklers	4.0	4.0	
Ceiling/lights	3.0	3.0	
Roofing + insulation	8.0	8.0	
Miscellaneous	1.0	1.0	
Windows	9.2	9.2	Assume 8 psf on glass area
DL = ΣComponents	52.0	52.0	
Expected LL	0.0	0.0	Roof live load is unlikely to occur during an earthquake
Total seismic		52.0	

Roof			
Floor Area	**14650**	**ft^2**	
hstory =	**13.00**	**ft**	
Gravity Load Cases			
	Short-term	**Seismic**	
Component	**lb/ft^2**	**lb/ft^2**	**Comments**
Partitions below	5.0	5.0	
4.5" concrete slab	56.3	56.3	150 lb/ft^3
~8" x 24" joists	32.1	32.1	~2350 linear ft/floor
~32" x 24" interior beams	25.4	25.4	~466 linear ft/floor
~18" x 24" N & S perimeter beams	3.2	3.2	~103 linear ft/floor
East spandrel	6.1	6.1	~180 linear ft/floor
West spandrel	3.2	3.2	~120 linear ft/floor
~40" x 24" typical girder	25.3	25.3	~370 linear ft/floor
Concrete walls below	4.9	4.9	~89 linear ft of 10" wall
Concrete walls above and below	14.6	14.6	~95 linear ft of wall to penthouses
24" x 33" typical columns below	6.8	6.8	22 columns/floor
MEP/sprinklers	4.0	4.0	
Ceiling/lights	3.0	3.0	
Roofing + insulation	8.0	8.0	
Miscellaneous	1.0	1.0	
Windows	1.3	1.3	Assume 8 psf on glass area
DL = ΣComponents	200.0	200.0	
Expected LL	15.0	15.0	Expected live load
Total seismic		215.0	

Typical Floor 3-7			
Floor Area	**14650**	**ft^2**	
hstory =	**13.00**	**ft**	
Gravity Load Cases			
	Short-term	**Seismic**	
Component	**lb/ft^2**	**lb/ft^2**	**Comments**
Partitions above/below	10.0	10.0	
4.5" concrete slab	56.3	56.3	150 lb/ft^3
~8" x 24" joists	32.1	32.1	~2350 linear ft/floor
~32" x 24" interior beams	25.4	25.4	~466 linear ft/floor
~18" x 24" N & S perimeter beams	3.2	3.2	~103 linear ft/floor
East spandrel	6.1	6.1	~180 linear ft/floor
West spandrel	3.2	3.2	~120 linear ft/floor
~40" x 24" typical girder	25.3	25.3	~370 linear ft/floor
Concrete walls below	33.3	33.3	~300 linear ft of 10" wall/floor
24" x 33" typical columns	13.6	13.6	22 columns/floor
MEP/sprinklers	4.0	4.0	
Ceiling/lights	3.0	3.0	
Floor finish	1.0	1.0	
Miscellaneous	1.0	1.0	
Windows	1.3	1.3	Assume 8 psf on glass area
DL = ΣComponents	219.0	219.0	
Expected LL	15.0	15.0	Expected live load
Total seismic		234.0	

SHEAR STRENGTH OF CONCRETE ELEMENTS

FEMA 306 Section 5.3.6

Input

f'_{ce}	4500	psi	concrete expected strength
λ	1.00		lightweight aggregate factor = 1.0 NWC, 0.85 sand LWC, 0.75 LWC
μ	1	λ	per ACI 318 11.7.4 = 1.4 monolithic, 1.0 roughened, 0.6 not roughened, 0.7 bars
$f_{ye\_transverse}$	50	ksi	expected transverse steel yield strength
$f_{ye\_longitudina}$	75	ksi	expected shear friction steel yield strength
b_w	24.0	in	width
l_w	33.0	in	length of wall (depth of beam or column)
h_w	13.00	ft	clear height of wall or column (length of beam or spandrel)
ρ_n	0.00278		transverse reinforcement ratio
A_s	31.2	in^2	longitudinal reinforcement area
P	**500**	kip	axial load
M_{n_1}	2620	k-ft	moment strength at one end of element (e.g. top)
M_{n_2}	2620	k-ft	moment strength at other end of element (e.g. bottom)
c	7.4	in	distance from extreme compressive fiber to neutral axis
θ	35	degrees	35 degrees unless limited to larger angles by the potential corner to corner crack
			for corner to corner crack, use θ= max(35, atan(l_w/h_w))= 35

Behavior Mode

Diagonal

$V_{n\_flexure} = (M_{n1} + M_{n2})/h$ =	403	kip	
$V_{n\_diagonal\_tension\_at\_low\_ductility\_demand}$ =	353	kip	($\mu \le 2$)
$V_{n\_diagonal\_tension\_at\_high\_ductility\_demand}$ =	229	kip	($\mu \ge 5$)
$V_{n\_sliding\_shear}$ =	634	kip	

Diagonal Tension Shear

	$\mu \le 2$	$\mu \ge 5$			
				flexural ductility demand	
$V_{n\_diagonal}$	353	229	kip	$= V_c + V_s + V_p$	
V_c	149	26	kip	$= \alpha\beta k_{rc}(f'_{ce})^{1/2} b_w (0.8 l_w)$	
V_s	122	122	kip	$= \rho_n f_{ye} b_w h_d$	
V_p	82	82	kip	$= ((l_w - c) N_u) / (2M/V)$	
k_{rc}	3.5	0.6			
α	1.0	1.0		$= 3 - M/(0.8 l_w V)$	$(1.0 \le \alpha \le 1.5)$
β	1.000	1.000		$= 0.5 + 20\rho_g$	(≤ 1.0)
ρ_g	0.03941	0.03941		longitudinal reinforcement ratio	
M/V	78.0	78.0	in	$= h_w/2$	assumes that beams/floors are stiffer than column (fixed- Adjust M/V calculation for other conditions.
h_d	36.6	36.6	in	$= (l_w - c)\cot\theta$	$(\le h_w)$
N_u	500	500	k	axial load	
$2M/V$	156	156	in	$= h_w$	assumes that beams/floors are stiffer than column (fixed- Adjust M/V calculation for other conditions.

Sliding Shear

$V_{n\_sliding}$	634	kip	$= A_{vf} f_y \mu$	$(\le 0.2 f_c A_c, 800 A_c)$
A_{vf}	31.2	in^2	area of shear friction reinforcement	

Input Variables	Units
Section Properties	
h	33
b	24

Reinforcement				
Layer	Number	Bar	A_{si}	d_{si}
-	-	-	in^2	in
1	5.55	#14	12.4875	2.63
2	2	#11	3.12	9.25
3	0	#9	0	0
4	0	#9	0	0
5	0	#9	0	0
6	0	#9	0	0
7	0	#9	0	0
8	0	#9	0	0
9	2	#11	3.12	23.75
10	5.55	#14	12.4875	30.37
			31.215	

Material Properties		
f'_C	4,500	
f_Y	75,000	
E_S	29,000,000	lb/in
ε_Y	0.002586	in/in
E_C	3,823,676	lb/in^2

Computed Results - Concrete		
$\varepsilon_{C\text{-}Crushing}$	0.00300	in/in
F_C	560439	lb
M_C	7536503	lb-in

Results		
M_Y	2620	k-ft
F_Y	0.000405405	in^{-1}
c	7.4	

Depth Factor		
b_1	0.825	--
a	6.105	in

Computed Results - Steel				
Layer	ε_S	f_S	F_S	M_S
	in/in	lb/in^2	lb	lb-in
1	0.001934	56080	700296	9713100
2	-0.000750	-21750	-67860	-491985
3	0.000000	0	0	0
4	0.000000	0	0	0
5	0.000000	0	0	0
6	0.000000	0	0	0
7	0.000000	0	0	0
8	0.000000	0	0	0
9	-0.006628	-75000	-234000	1696500
10	-0.009312	-75000	-936563	12990122
		Sum	-538127	23907737

KP: Check Equilibrium F_c + Sum(F_S)	56846	31884755
	57	2657
Whitney: Check Equilibrium F_c + Sum(F_S)	22312	31444241
	22	2620