Rating form completed by:

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Text in green is to be part of UC Santa Cruz building database and may be part of UCOP database

UC Santa Cruz building seismic ratings

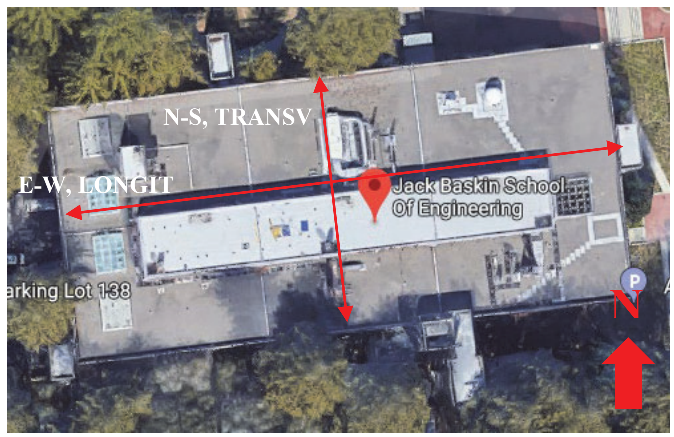
Jack Baskin Engineering

CAAN #7194

606 Engineering Loop, Santa Cruz, CA 95064

UCSC Campus: Main Campus

DATE: 2018-12-27



Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	V (Poor)	
Rating basis	Tier 1	ASCE 41-17[1]
Date of rating	2018	
Recommended list assignment (UC Santa Cruz category for retrofit)	Priority B	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application
Ballpark total construction cost to retrofit to IV rating[2]	High ($200-$400/sf)	See recommendations on further evaluation and retrofit
Is 2018-2019 rating required by UCOP?	Yes	We did not find a documented previous rating
Further evaluation recommended?	Tier 3 NLRHA	

[1] We translate this Tier 1 evaluation to a Seismic Performance Level rating using professional judgment. Non-compliant items in the Tier 1 evaluation do not automatically put a building into a particular rating category, but we evaluate such items 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. See Section III B of the UC Seismic Policy and Method B of Section 321 of the 2016 California Existing Building Code.

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

Building information used in this evaluation

- Architectural and Structural drawings by Reid & Tarics, "Engineering Unit 1, University of California, Santa Cruz," dated `5 June 1968, sheets A1-A22 and S1-S22.
- University of California building database information provided by Jose Sanchez (UCSC) on 2018-11-20.

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 and limited quick checks. We made a brief site visit. We did not perform the Tier 1 nonstructural evaluation, but we looked for potentially hazardous nonstructural components during our site visit. No nonstructural hazards were identified.

Brief description of structure

The Jack Baskin Engineering Building was designed in 1968 by the office of Reid & Tarics, Architects and Engineers; construction was completed in late 1970. According to campus records the building has 184,000 gross square feet and is mainly used for engineering labs and offices. The building is all cast-in-place reinforced concrete construction with the exception of the penthouse roof, which is lightweight steel.

The building is three stories above a full basement. The superstructure measures 48 feet in height from the main roof to the first level. Story height is typically 16'-0" at the ground story and above and 13'-0" at the basement. There is a rooftop mechanical penthouse that occupies about 20% or the roof area.

The building measures 308 feet long (east-west) by 140 feet (north-south), including balconies that extend 10 feet beyond the exterior walls on the south side and at the east and west ends. The floor plate is divided along its length into three structurally separate parts by two narrow separation joints (refer to Figure). The separation joints are presumably intended to allow building shrinkage or temperature movement in the longitudinal direction. The slab structure is keyed across the joints restrain relative movement along the joint while allowing the joint to open and close. Each portion between joints is structurally independent for support of gravity loads and for resisting seismic forces in the east-west direction. However, the central portion is reliant on the transfer of lateral forces through the toothed separation joints to the end portions for seismic lateral force resistance in the north-south direction.

Above the first level, the exterior framing is formed by a perimeter frame of wide columns, spaced at 16-foot centers, and deep beams; the concrete beams and columns are infilled with glazing. In addition to providing the weather enclosure, the frame is the primary seismic lateral-force-resisting system for the building. Below the first floor level, basement walls that are aligned with the frames above enclose the building and provide the lateral-force-resisting system.

There are mechanical shafts at the center of each portion of the building. The shafts are surrounded by thin concrete walls, but they extend only 10 feet above the floor to allow for services to access the ceilings. As such, these cores provide no appreciable resistance to seismic forces. There are light captive short columns at the corners of the shafts, which are highly vulnerable to earthquake damage. However, we expect that the floor span is designed to span over these columns.

There are 4 stair cores abutting the building exterior. The stair shafts are separated from the main building by narrow joints. Based on the narrow joints and the stiffness of the shear wall cores, it should be expected that the building will pound against the cores in the event of a moderately severe earthquake.

At the era of this building's construction, there was a focus in Bay Area building design practice on shrinkage and thermal induced cracking in concrete buildings. In buildings of this era that we have seen, separation joints of various types were introduced to address such concerns, sometimes to the detriment of seismic performance. Baskin Engineering is divided along its length into 3 parts (roughly 100 foot sections) by structural separation joints. Each portion has an independent gravity load resisting system, with side-by-side columns on each side of each joint at both the façade and interior. Each portion has 6 bays of frame at the north and south façade to resist lateral seismic forces in the east west direction and seismic behavior should be quite regular. However, in the orthogonal direction, the central section has no seismic frames to resist forces in the north-south direction, and is reliant on transfer across the toothed separation joints to provide resistance. The frames in the end sections are located at the ends of the building away from the joints, resulting in substantial torsion under earthquake forces.

Foundation System: The structure is founded below the basement level on 30-inch diameter cast-in-place drilled piers extending down to rock. A single pier is located beneath each column. Upper portions of piers are well confined by spiral reinforcement. Pier caps are interconnected by a grid of grade beams running in orthogonal directions.

Structural system for vertical (gravity) load: The three floors and roof are framed with a 21” deep concrete waffle slab system. The system has 9-inch wide ribs spaced at 4’-0” centers and a 4-inch thick slab. There are 12’-9” square drop panels at interior columns. Interior columns are spaced at 32 feet on center and are typically 36 inches in diameter with spiral transverse reinforcement. At the separation joints, pairs of 36-inch square columns replace the typical round columns.

A concrete perimeter frame of deep beams and columns spaced at 16-foot centers forms the façade of the building and supports gravity loads. A 10-foot wide balcony, framed with one way joists aligned with the waffle ribs, cantilevers out from the exterior frame on 3 sides.

Structural system for lateral forces: A perimeter concrete moment resisting frame forms the primary lateral-force-resisting system for the superstructure. Frame columns, measuring 24 inches by 55 inches, are spaced at 16 feet on center. Columns are tied with transverse reinforcement at 4 inch spacing over their full height. Frame beams are 18 inches wide and 6’-0” deep, measured from the top of waffle slab. The east and west facades each contain 8 bays. The north and south facades each contain 18 bays. These are separated into 3 groups of 6 bays, separated by building separation joints. At the joints, half columns are used on each side of the joint.

The waffle slab at each level serves as the floor diaphragm to deliver inertial story forces to the perimeter frame. As previously discussed, the diaphragm is separated into 3 parts by toothed expansion joints. The toothing of the diaphragm consists of 2-1/2” wide by 12-inch-long by full height keys. There is no reinforcement crossing the joints, including diaphragm chords.

Concrete walls, which are aligned with the frame above, resist lateral forces at the basement and establish the seismic base of the building at Level 1. At the rooftop penthouse, which has a lightweight steel roof, lateral forces are resisted by a concrete frame in the east west direction and by the frame columns acting as cantilevers in the north-south direction.

Stair cores located exterior to the building and structurally separated have concrete shear walls for lateral force resistance. It is expected that floors of the building will pound against the cores to the probable benefit of the building and detriment to the cores.

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

Identified seismic deficiencies and behaviors of the building include the following:

- Under north-south direction motions, plan torsion is created in the parts of the building at the east and west ends as a result of the separation joints. We expect that increased drift at the center of the building will result. Substantial demand will be placed on frame elements in the end portions of the building to resist the torsion. Demand on frames at north and south facades should be evaluated for orthogonal load case, because they will need to resist the plan torsion, unless the diaphragm is tied by new chord elements (as recommended).
- Frames on the north and south elevation do not satisfy strong column-weak beam test, substantially failing the test when only beam flexure behavior is assumed and failing to a lesser extent when beam shear failure is considered. This indicates a possibility of a story mechanism between the first and second floor levels and resulting increase in deformation demand.
- Frame beams on the north and south facades are strongly shear critical; these beams have less transverse reinforcement than the frame beams on the east and west facades.

- Frame columns on the east and west facades are heavily reinforced for flexure, and are flexure/ diagonal tension critical at the first level. Although the columns have closely spaced transverse reinforcement (#5 hoops plus #4 crosstie @ 4” o.c.) which are adequate to initially develop the column flexural strength, the shear strength of the column will degrade with moderate to high bending ductility demand leading to the possibility of shear failure as the building drifts. Similar concern exists at the square interior gravity columns at separation joints, which are flexure/ diagonal tension critical at all levels.
- Quick check for strength of columns substantially failed test—i.e. DCR = 3.4 N-S and 7.7 E-W., indicating that building may not have adequate strength and stiffness.
- Floors will pound against shear wall cores that abut building, likely leading to local damage at the joint where pounding occurs and potentially having more severe effect on stair core walls. Pounding is not expected to harm building, except for local affects.
- Captive short columns at mechanical cores are likely to fail in shear, however we expect that the floor can span over the columns if they fail.
- Columns at the penthouse lack closely spaced ties at base of cantilever columns.

Structural deficiency	Affects rating?	Structural deficiency	Affects rating?
Lateral system stress check (wall shear, column shear or flexure, or brace axial as applicable)	Y	Openings at shear walls (concrete or masonry)	N
Load path	N	Liquefaction	N
Adjacent buildings	Y	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	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]

We walked through all floors of the building and we were able to enter about 30% of the rooms. We did not observe any non-structural life-safety hazards. Most of the spaces that we observed were classrooms and faculty offices. When more detailed evaluations of the building are made, as we recommend, they should include further review of details of construction of glazed wall at concrete frame façade should be included, in particular at exit areas.

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	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

[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.

Discussion of rating

The generally good quality of the concrete detailing, in particular the closely spaced transverse reinforcement in columns, is our basis for rating the building V (Poor) rather than VI (Very Poor). As described in the "Description of deficiencies" section of this report, closely-spaced transverse reinforcement is expected to prevent pre-emptive diagonal tension shear failure of columns (as would occur in many non-ductile concrete buildings of this era), but the columns still have limited ductility (flexure/shear failure) and may be subject to shear failure at larger lateral displacement demands that are imposed on them by the building's seismic-force resisting system and configuration.

Recommendations for further evaluation or retrofit

We recommend that the University perform a more detailed evaluation as a precursor to probable structural retrofit. Partial retrofit to provide chord ties across separation joints, at roof level at a minimum, should be considered if more extensive retrofit cannot be accomplished in a short time frame. The structure would benefit from a Tier 3 evaluation using nonlinear response-history analysis (NLRHA).

Peer review of rating

This seismic evaluation was discussed in a peer review meeting on 24 July 2019. The reviewers present were Bret Lizundia of R+C and Jay Yin of Degenkolb. Comments from the reviewers have been incorporated into this report. The reviewer agreed with the assigned rating.

Additional building data	Entry	Notes
Latitude	37.000438	
Longitude	-122.0631353	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	3	
Number of stories (basements) below lowest perimeter grade	1	
Building occupiable area (OGSF)	183951	
Risk Category per 2016 CBC Table 1604.5	III	Educational Use with Occupant Load > 500
Estimated fundamental period	0..59 sec	Estimated using ASCE 41-17 equation 4-4 and 7-18
Building height, h_n	48 ft	Structural height defined per ASCE 7-16 Section 11.2
Coefficient for period, C_t	0.018	Estimated using ASCE 41-17 equation 4-4 and 7-18
Exponent on height for period, β	0.90	Estimated using ASCE 41-17 equation 4-4 and 7-18
Site data		
975 yr hazard parameters S_s, S_1	1.286, 0.488	
Site class	D	
Site class basis[4]	Geotech	See footnote below

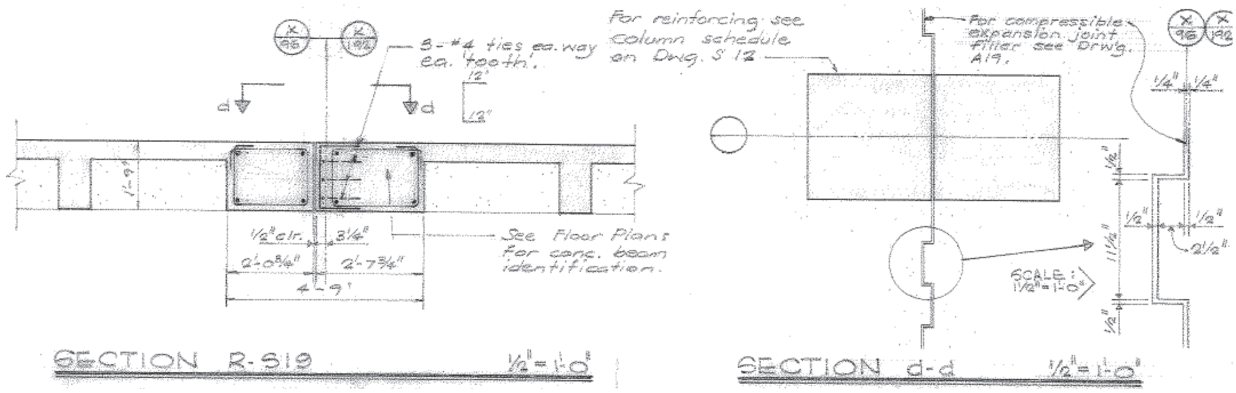
[4] Determination of site class and assessment of geotechnical hazards are based on correspondence with Pacific Crest Geotechnical Engineers and Nolan, Zinn, and Associates Geologists. [*Revised Geology and Geologic Hazards, Santa Cruz Campus, University of California*, Job # 04003-SC 13 May 2005]. Site class is taken as D throughout the main campus of UC Santa Cruz. The following links provide hazard maps for liquefaction, landslide, and fault rupture:
https://gis.santacruzcounty.us/mapgallery/Emergency%20Management/Hazard%20Mitigation/LiquifactionMap2009.pdf
https://gis.santacruzcounty.us/mapgallery/Emergency%20Management/Hazard%20Mitigation/LandslideMap2009.pdf
https://gis.santacruzcounty.us/mapgallery/Emergency%20Management/Hazard%20Mitigation/FaultZoneMap2009.pdf

Site parameters F_a, F_v[5]	1, 1.81	
Ground motion parameters S_{cs}, S_{c1}	1.286, 0.885	
S_a at building period	1.29	
Site V_{s30}	900 ft/s	
V_{s30} basis	Estimated	Estimated based on site classification of D.
Liquefaction potential	Low	
Liquefaction assessment basis	County map	See footnote below
Landslide potential	Low	
Landslide assessment basis	County map	See footnote below
Active fault-rupture identified at site?	No	
Fault rupture assessment basis	County map	See footnote below
Site-specific ground motion study?	No	
Applicable code		
Applicable code or approx. date of original construction	Built: 1968 Code: 1964 UBC	Code inferred based on design year
Applicable code for partial retrofit	None	
Applicable code for full retrofit	None	No full retrofit
FEMA P-154 data		
Model building type North-South	C1 - Conc. Frame	
Model building type East-West	C1 - Conc. Frame	
FEMA P-154 score	N/A	N/A for ASCE 41 Tier 1 evaluation.
Previous ratings		
Most recent rating	VI (Very Poor)	
Date of most recent rating	Unknown	
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

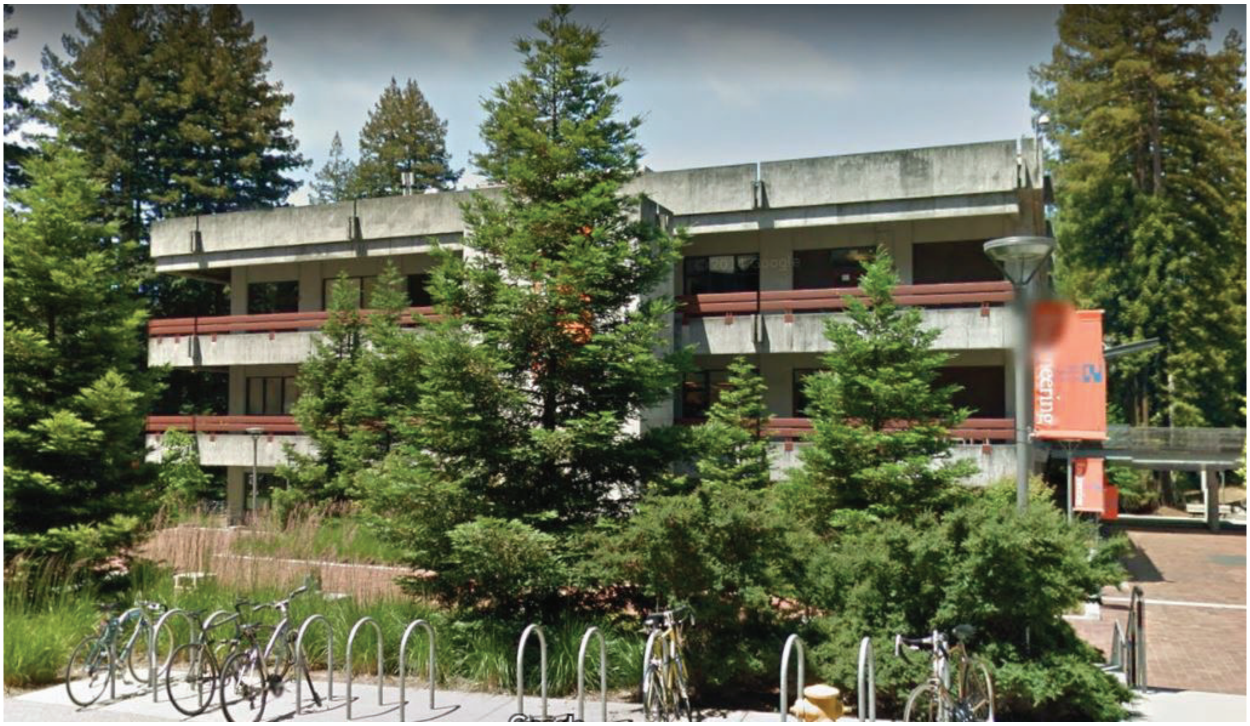
[5] F_V factor used does not include the requirements of Section 11.4.8-3 of ASCE 7-16 that are applicable to Site Class D, and which per Exception 2 would result in an effective F_V factor of 2.72 (1.5 times larger). At the Santa Cruz main campus this only affects structures with T>0.69 seconds. We understand that the appropriateness of this requirement of Section 11.4.8 might be reviewed by UCOP.

[redacted]

Annotated Floor Plan

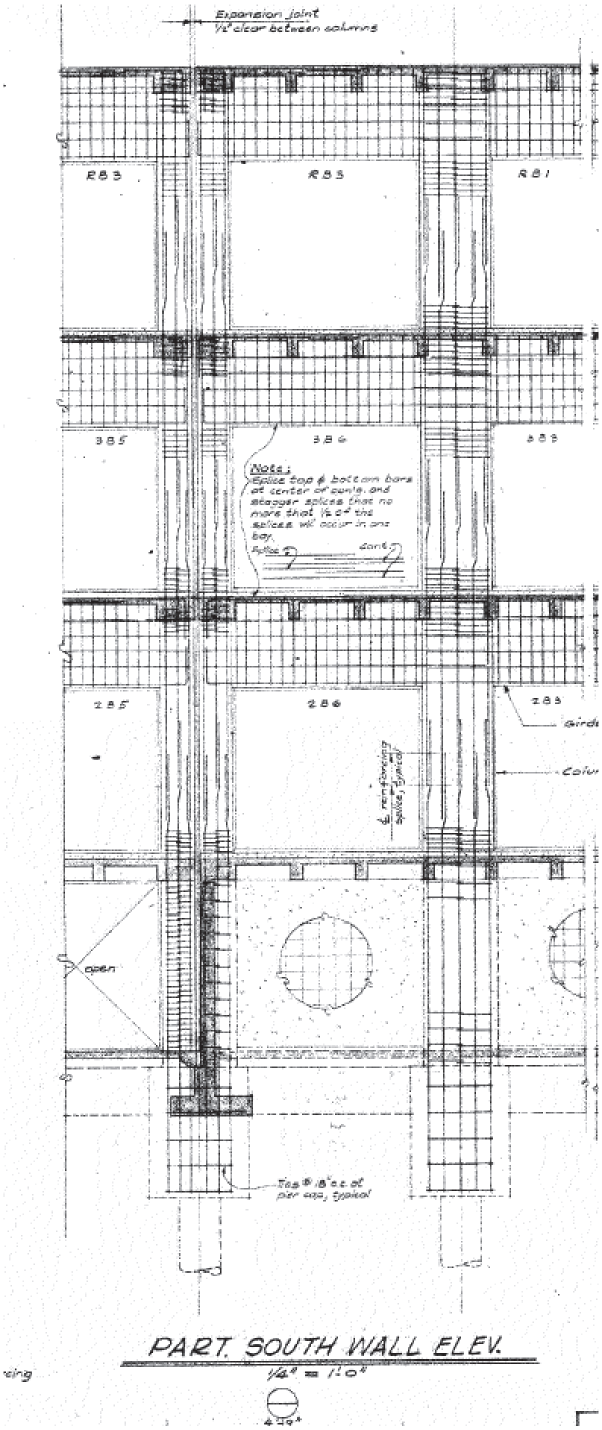


Section at separation joint/key



East elevation

South elevation



South wall elevation

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Building Address:	606 Engineering Loop			Page:	1	of	3

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

LOW SEISMICITY

BUILDING SYSTEMS - *GENERAL*

C NC N/A U	Description
☒ C ☐ 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:**
☐ C ☒ 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:** *Pounding will occur where building abuts the stiffer stair cores at 4 locations around the perimeter. Local dame is highly likely and damage to walls at cores is also likely*
☒ C ☐ 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:**

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

C NC N/A U	Description
☒ C ☐ 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:**
☒ C ☐ NC ☐ N/A ☐ U	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:**
☒ C ☐ NC ☐ N/A ☐ U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3) **Comments:**

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

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ASCE 41-17
Collapse Prevention Basic Configuration Checklist

C NC N/A U	
C: [X] NC: [ ] N/A: [ ] U: [ ]	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4) **Comments:**
C: [X] NC: [ ] N/A: [ ] U: [ ]	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5) **Comments:**
C: [ ] NC: [X] N/A: [ ] U: [ ]	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6) **Comments:** *Separation joints result in substantial torsion in the end portions of the building. This should be corrected by tying building together across joint at north and south (i.e. chords), regardless of computed behavior.*

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

GEOLOGIC SITE HAZARD

	Description
C: [X] NC: [ ] N/A: [ ] U: [ ]	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2m) under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1) **Comments:**
C: [ ] NC: [ ] N/A: [ ] U: [X]	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:**
C: [X] 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:**

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

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ASCE 41-17
Collapse Prevention Basic Configuration Checklist

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:**
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:**

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

Low Seismicity

Seismic-Force-Resisting System

C NC N/A U	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 $0.20f'_c$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than $0.30f'_c$. (Commentary: Sec. A.3.1.4.2. Tier 2: Sec. 5.5.2.1.3) **Comments:**

Connections

C NC N/A U	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

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

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ASCE 41-17
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:** *DCR's are quite large for this spot check, when considering frame columns only, e.g. DCR = 7.7 in NS direction with Ms = 1.75.*
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:** *Not the primary lateral system, but will provide some resistance upon softening of frame.*

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:** *Captive columns occur at the c mechanical shafts, where walls between columns are 10 feet high (4 feet clear of soffit above). It is expected that floor and roof can span to primary columns after loss of captive columns.*
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:** *Type A frame columns, at the east and west ends of the building, are flexure/ shear critical at between the 1st and 2nd levels at high ductility demands. Other frame columns are flexure critical.* *Spandrels are shear critical at north and south elevations at 2nd and 3rd levels.*
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:** *Frames at the north and south facades substantially fail check, if spandrel shear failure is not considered. Frames fail check, but by a much lesser amount if spandrel shear failure is considered. See checks.*

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

UC Campus:	Santa Cruz			Date:	12/27/2018	
Building CAAN:	7194	Auxiliary CAAN:		By Firm:	Maffei	
Building Name:	J Baskin Engineering			Initials:		Checked:
Building Address:	606 Engineering Loop			Page:	3 of 4	

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

C NC N/A U	
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:** *Splices near midheight of column, staggered Cadweld couplers in first story, typ. Very good detailing*
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:** *Ties at 4" o.c., typ. Note that 90 deg hooks on hoops, not 135 deg. 180 deg hooks on crossties.*
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:** *Stirrups @ 8" o.c., typ. with #10 min longitudinal bars.*
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:** *Square columns at building expansion joints 30% over at high ductility demand (m>5); ok at moderate ductility. Consider fiberwrap top and bottom.*
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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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type C1

Diaphragms

C NC N/A U	Description
NC: X	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:** *The diaphragm is split by joints, eliminating chords, and relying on frame action on the north and south sides to resist torsion. Although there is a load path, end wings will be subjected to large torsional demand and drift will be substantially larger at center.*

Connections

C NC N/A U	Description
C: X	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:**

Note: **C** = Compliant **NC** = Noncompliant **N/A** = Not Applicable **U** = Unknown www.maffei-structure.com

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SEISMIC EVALUATION OF EXISTING BUILDINGS - TIER 1 SCREENING

ASCE 41-17 Chapter 4

General				Reference
Building	Baskin Engineering			
Architect	Reid & Tarics			
Structural Engineer	Reid & Tarics			
Location	606 Engineering Loop, Santa Cruz, CA 95064			
Design date	1968			
Latitude	37.000438			https://hazards.atcouncil.org/
Longitude	-122.0631353			"
Stories above grade	3	Plus Basement		

Seismic parameters

Risk Category	III	2016 CBC Table 1604.5		
Site Class	C	Assumed		(ASCE 41-17 2.4.1.6, ASCE 7-16 Chapter 20)
Liquefaction hazard	Low	Assumed		(ASCE 41-17 3.3.4)
Landslide hazard	Low	Assumed		
S_{DS}	1.306	https://hazards.atcouncil.org/	Based on ASCE 7-16 DE, used to determine "Level of Seismicity"	(ASCE 41-17 Eq 2-4)
S_{D1}	0.585	https://hazards.atcouncil.org/	Based on ASCE 7-16 DE, used to determine "Level of Seismicity"	(ASCE 41-17 Eq 2-5)
S_{XS}	1.286	For BSE-2E hazard level	https://hazards.atcouncil.org/	(ASCE 41-17 Table 2-2)
S_{X1}	0.89	For BSE-2E hazard level	https://hazards.atcouncil.org/	(ASCE 41-17 Table 2-2)

Scope

Performance level	Limited Safety	Ms = 1.75	(ASCE 41-17 Sec 4.4.3.2)	(ASCE 41-17 Table 2-2)
Seismic hazard level	BSE-2E			(ASCE 41-17 Table 2-2)
Level of seismicity	High			(ASCE 41-17 Table 2-4)
Building type	C1: Concrete moment frame			(ASCE 41-17 Table 3-1)

Material properties				Notes	
Concrete	f'_c	4000	psi	Specified on drawings, NWC	(ASCE 41-17 Table 10-4)
Reinf.	f_y	40	ksi	Not specified on Drawings	(ASCE 41-17 Table 10-4)
Steel	F_y	N/A	ksi	N/A	(ASCE 41-17 Table 9-1)

Checklists

Benchmark building	No	(ASCE 41-17 Table 3-2)
Checklist(s) req'd	17.1.2 Basic Configuration	(ASCE 41-17 Table 4-6)
	17.12 Structural Checklist for Building Types C1	(ASCE 41-17 Table 4-6)
	~~17.19 Nonstructural Checklist~~ (not performed)	(ASCE 41-17 Table 4-6)

Seismic forces

V	37294	kip	$V = Cs_a W$	= 1.29W	(ASCE 41-17 Eq 4-1)
W	29000	kip	building weight		(ASCE 41-17 4.4.2.1)
C	1.0		Convert linear elastic to inelastic disp.		(ASCE 41-17 Table 4-7)
S_a	1.29	g	$S_a = S_{x1}/T \leq S_{XS}$		(ASCE 41-17 Eq 4-3)
T	0.59	sec	$T = C_t h_n^{\beta}$		(ASCE 41-17 Eq 4-4)
C_t	0.018				(ASCE 41-17 Eq 4-4)
β	0.90				(ASCE 41-17 Eq 4-4)
h_n	48	ft	building height		(ASCE 41-17 Eq 4-4)

Story Forces (ASCE 41-17 4-2a) (ASCE 41-17 4-2b)

Story	w	story ht	h	wh^k	F_{story}	F_{story}	V_{story}
	kip	ft	ft			kip	kip
Roof	10200		48	579019	0.53	19664	
3	9400	16.0	32	349542	0.32	11871	19664
2	9400	16.0	16	169600	0.15	5760	31534
1		16.0	0				37294
Total	29000			1098161	1.0	37294	
k	1.04	k = 1.0 for T < 0.5, 2.0 for T > 2.5, linear interpolation between					

Frame Capacity

V_{NS}^{avg}	V_{EW}^{avg}
10,000	14000

$F_{story} = V(wh^k)/(\Sigma wh^k)$ (ASCE 41-17 4-2a)

$V_{story} = \Sigma_{above} F_{story}$ (ASCE 41-17 4-2b)

Shear stress in Frame Columns (ASCE 41-17 4-7)

Story	$A_{c\,N\text{-}S}$	$A_{c\,E\text{-}W}$	v_{NS}^{avg}	v_{EW}^{avg}	D/C_{NS}	D/C_{EW}
	in^2	in^2	psi	psi		
Roof						
3	21120	47520	532	236	4.2	1.9
2	21120	47520	853	379	6.7	3.0
1	21120	47520	1009	448	8.0	3.5
Total						

M_s 1.75 (ASCE 41-17 Table 4-8)

v_{limit} 126 psi $v_{limit} = 2\sqrt{f_c'} \geq 100$ psi

$v^{avg} = (1/M_S)(V_{story}/A_c)$ (ASCE 41-17 Eq 4-8)

Weight takeoff

	Floor	Roof
Basic Waffle	129 psf	129 psf
Add for drops	19 psf	19 psf
Add for int cols	10 psf	5 psf
Add for frame beams	12 psf	12 psf
Add for frame col	23 psf	12 psf
Partitions	15 psf	8 psf
Ceiling/ Mech	10 psf	12 psf
Roofing		40 psf
Total	219 psf	237 psf
Weight	**9433 kps**	**10226 kps**

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SHEAR STRENGTH OF CONCRETE ELEMENTS

FEMA 306 Section 5.3.6

Input

f'_{ce}	6000	psi	concrete expected strength
λ	1.00		lightweight aggregate factor = 1.0 NWC, 0.85 sand LWC, 0.75 LWC
μ	1.4	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
$f_{ye\_longitudina}$	50	ksi	expected shear friction steel yield strength
b_w	24.0	in	width
l_w	55.0	in	length of wall (depth of beam or column)
h_w	10.0	ft	clear height of wall or column (length of beam or spandrel)
ρ_n	0.00854		transverse reinforcement ratio
A_s	41.5	in^2	longitudinal reinforcement area
P	0	kip	axial load
M_{n_1}	3290	k-ft	moment strength at one end of element (e.g. top)
M_{n_2}	3290	k-ft	moment strength at other end of element (e.g. bottom)
c	19.9	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 **Flexure/ Diagonal**

$V_{n\_flexure} = (M_{n1} + M_{n2})/h$ =	658	kip	
$V_{n\_diagonal\_tension\_at\_low\_ductility\_demand}$ =	943	kip	($\mu \le 2$)
$V_{n\_diagonal\_tension\_at\_high\_ductility\_demand}$ =	587	kip	($\mu \ge 5$)
$V_{n\_sliding\_shear}$ =	1056	kip	

Diagonal Tension Shear

	$\mu \le 2$	$\mu \ge 5$			
				flexural ductility demand	
$V_{n\_diagonal}$	943	587	kip	$= V_c + V_s + V_p$	
V_c	429	74	kip	$= \alpha\beta k_{rc}(f'_{ce})^{1/2} b_w (0.8 l_w)$	
V_s	513	513	kip	$= \rho_n f_{ye} b_w h_d$	
V_p	0	0	kip	$= ((l_w - c) N_u) / (2M/V)$	
k_{rc}	3.5	0.6			
α	1.5	1.5		$= 3 - M/(0.8 l_w V)$	$(1.0 \le \alpha \le 1.5)$
β	1.000	1.000		$= 0.5 + 20\rho_g$	(≤ 1.0)
ρ_g	0.03144	0.03144		longitudinal reinforcement ratio	
M/V	60.0	60.0	in	$= h_w/2$	assumes that beams/floors are stiffer than column (fixed- Adjust M/V calculation for other conditions.
h_d	50.1	50.1	in	$= (l_w - c)\cot\theta$	$(\le h_w)$
N_u	0	0	k	axial load	
$2M/V$	120	120	in	$= h_w$	assumes that beams/floors are stiffer than column (fixed- Adjust M/V calculation for other conditions.

Sliding Shear

$V_{n\_sliding}$	1056	kip	$= A_{vf} f_y \mu$	$(\le 0.2 f_c A_c, 800 A_c)$
A_{vf}	41.5	in^2	area of shear friction reinforcement	

SHEAR STRENGTH OF CONCRETE ELEMENTS

FEMA 306 Section 5.3.6

Input

f'_{ce}	6000	psi	concrete expected strength
λ	1.00		lightweight aggregate factor = 1.0 NWC, 0.85 sand LWC, 0.75 LWC
μ	1.4	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
$f_{ye\_longitudina}$	50	ksi	expected shear friction steel yield strength
b_w	55.0	in	width
l_w	24.0	in	length of wall (depth of beam or column)
h_w	10.0	ft	clear height of wall or column (length of beam or spandrel)
ρ_n	0.00927		transverse reinforcement ratio
A_s	41.5	in^2	longitudinal reinforcement area
P	0	kip	axial load
M_{n_1}	1435	k-ft	moment strength at one end of element (e.g. top)
M_{n_2}	1435	k-ft	moment strength at other end of element (e.g. bottom)
c	8.7	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

Flexure

$V_{n\_flexure} = (M_{n1} + M_{n2})/h$ =	287	kip	
$V_{n\_diagonal\_tension\_at\_low\_ductility\_demand}$ =	843	kip	($\mu \le 2$)
$V_{n\_diagonal\_tension\_at\_high\_ductility\_demand}$ =	606	kip	($\mu \ge 5$)
$V_{n\_sliding\_shear}$ =	1056	kip	

Diagonal Tension Shear

	$\mu \le 2$	$\mu \ge 5$			
				flexural ductility demand	
$V_{n\_diagonal}$	843	606	kip	$= V_c + V_s + V_p$	
V_c	286	49	kip	$= \alpha\beta k_{rc} (f'_{ce})^{1/2} b_w (0.8 l_w)$	
V_s	557	557	kip	$= \rho_n f_{ye} b_w h_d$	
V_p	0	0	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	0.03144	0.03144		longitudinal reinforcement ratio	
M/V	60.0	60.0	in	$= h_w/2$	assumes that beams/floors are stiffer than column (fixed- Adjust M/V calculation for other conditions.
h_d	21.8	21.8	in	$= (l_w - c)\cot\theta$	$(\le h_w)$
N_u	0	0	k	axial load	
$2M/V$	120	120	in	$= h_w$	assumes that beams/floors are stiffer than column (fixed- Adjust M/V calculation for other conditions.

Sliding Shear

$V_{n\_sliding}$	1056	kip	$= A_{vf} f_y \mu$	$(\le 0.2 f_c A_c, 800 A_c)$
A_{vf}	41.5	in^2	area of shear friction reinforcement	

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SHEAR STRENGTH OF CONCRETE ELEMENTS

FEMA 306 Section 5.3.6

Input

f'_{ce}	6000	psi	concrete expected strength
λ	1.00		lightweight aggregate factor = 1.0 NWC, 0.85 sand LWC, 0.75 LWC
μ	1.4	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
$f_{ye\_longitudinal}$	50	ksi	expected shear friction steel yield strength
b_w	36.0	in	width
l_w	36.0	in	length of wall (depth of beam or column)
h_w	14.25	ft	clear height of wall or column (length of beam or spandrel)
ρ_n	0.00139		transverse reinforcement ratio
A_s	20.3	in^2	longitudinal reinforcement area
P	0	kip	axial load
M_{n_1}	1290	k-ft	moment strength at one end of element (e.g. top)
M_{n_2}	1290	k-ft	moment strength at other end of element (e.g. bottom)
c	6.5	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

Flexure/ Diagonal

$V_{n\_flexure} = (M_{n1} + M_{n2})/h$ =	181	kip	
$V_{n\_diagonal\_tension\_at\_low\_ductility\_demand}$ =	334	kip	($\mu \le 2$)
$V_{n\_diagonal\_tension\_at\_high\_ductility\_demand}$ =	144	kip	($\mu \ge 5$)
$V_{n\_sliding\_shear}$ =	1037	kip	

Diagonal Tension Shear

	$\mu \le 2$	$\mu \ge 5$			
				flexural ductility demand	
$V_{n\_diagonal}$	334	144	kip	$= V_c + V_s + V_p$	
V_c	229	39	kip	$= \alpha\beta k_{rc}(f'_{ce})^{1/2} b_w (0.8 l_w)$	
V_s	105	105	kip	$= \rho_n f_{ye} b_w h_d$	
V_p	0	0	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	0.814	0.814		$= 0.5 + 20\rho_g$	(≤ 1.0)
ρ_g	0.01568	0.01568		longitudinal reinforcement ratio	
M/V	85.5	85.5	in	$= h_w/2$	assumes that beams/floors are stiffer than column (fixed- Adjust M/V calculation for other conditions.
h_d	42.1	42.1	in	$= (l_w - c)\cot\theta$	$(\le h_w)$
N_u	0	0	k	axial load	
$2M/V$	171	171	in	$= h_w$	assumes that beams/floors are stiffer than column (fixed- Adjust M/V calculation for other conditions.

Sliding Shear

$V_{n\_sliding}$	1037	kip	$= A_{vf} f_y \mu$	$(\le 0.2 f_c A_c, 800 A_c)$
A_{vf}	20.3	in^2	area of shear friction reinforcement	