Rating form completed by:

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Evaluator: JY/WAL/BL
Date: 06/28/2019

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

DATE: 2019-06-28

06-28-19

UC Santa Cruz Building Seismic Ratings

East Field House North Building

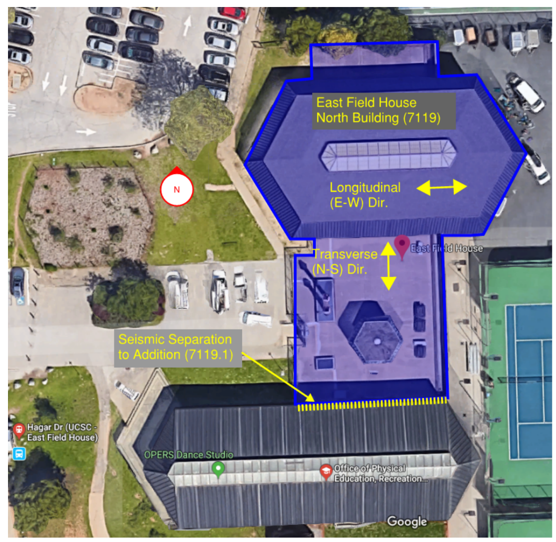
CAAN #7119

418 Hager Drive, Santa Cruz, CA 95064

UCSC Campus: Main Campus

Northwest Elevation (Looking Southeast)

Plan



Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	IV (Fair)	
Rating basis	Tier 1	ASCE 41-17[1]
Date of rating	2019	
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]	None	See recommendations on further evaluation and retrofit.
Is 2018-2019 rating required by UCOP?	Yes	Building was previously rated in 1998

[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 19 May 2017 *UC Seismic Safety Policy* and Method B of Section 321 of the 2016 *California Building Code*.

[2] Per Section III.A.4.i of the 26 March 2019 *UC Seismic Program Guidebook, Version 1.3*, 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.

Rating summary	Entry	Notes
Further evaluation recommended?	No	

Building information used in this evaluation

- Drawings for East Field House North Building (CAAN 7119)
    - Architectural drawings by Callister Payne & Rosse, "Santa Cruz Campus Playing Fields and Fieldhouse," dated 25 September 1964, Sheets A1-A10.
    - Structural drawings by Stefan J. Medwadowski, "Santa Cruz Campus Playing Fields and Fieldhouse," dated 25 September 1964, Sheets S1-S4.
    - Seismic retrofit structural drawing by Wildman & Morris, dated 9 February 1998. One page.
- Drawings for East Field House Addition (CAAN 7119.1), which provide information on the seismic separation between the original building and the additions.
    - Architectural drawings:
      2nd Story Addition: by J. Martin Rosse A.I.A. Architect, "Physical Activities Facilities East, University of California, Santa Cruz," dated 21 July 1975, Sheets A1 (existing conditions).
      Original 1st Story: by Callister, Payne & Rosse Architects, "Enclosed Courts, University of California, Santa Cruz," data 6 May 1961, Sheets 1 to 3.
    - Structural drawings:
      2nd Story Addition: by Sexton, FitzGerald & Kaplan, Engineers, "Physical Activities Facilities East, University of California, Santa Cruz," dated 21 July 1975, Sheets S1 through S5 corresponding to the building added on top of the racquetball court existing building.
      Original 1st Story: by Stefan J. Medwadowski Consulting Structural Engineer, "Enclosed Courts, University of California, Santa Cruz," dated 6 May 1961, Sheets S.1 through S.3.
      Original 1st Story foundation retrofit: by Stefan J. Medwadowski Consulting Structural Engineer, "Enclosed Courts, University of California, Santa Cruz," dated 7 April 1970, Sheets S.4.

Additional building information known to exist

None

Scope for completing this form

Reviewed structural drawings for original construction, made a brief site visit on 16 May 2019, and carried out ASCE 41-17 Tier 1 evaluation.

Brief description of structure

The East Field House North Building is a one-story reinforced concrete shear wall building with two rectangular sections sandwiching a flattened hexagon or diamond-shaped general purpose room with high roof. It houses an approximately 14,000 square feet sports facility. Tennis courts are to the east, and a seismically independent addition was later constructed to the south in approximately 1975. The out-to-out dimensions for the north rectangle are 23'4" in the north-south direction and 59'0" in the east-west direction. The general purpose room is 66'2" north-south and 125'10-3/4" east-west. The south rectangle is 86'4" north-south and 72'0" east-west. The structure is sited on a sloped site which is low on the east side which corresponds to the first floor elevation and high on the west side which corresponds to the terrace level above the two rectangular sections. The terrace roofs cover an outdoor walkway on the east side of the rectangular sections. The structure was designed in 1964 by architects Callister Payne & Rosse. Stefan J. Medwadowski was the structural engineer. The construction completion date is unknown, but it is assumed to be 1964.

The central diamond-shaped general purpose room contains a copper roof supported by steel space truss system that is approximately 35'-0" above first floor. There are two rectangular sections located to the north and south of the general purpose room. The north rectangular section functions as a light storage area. Storage shelves, steel fences, and light partition walls were observed inside. No falling hazard items were identified. The south rectangular section contains a sport medical office, two locker rooms, a stair to roof terrace, a mechanical room, and an open-air transformer room with two metal flues extends above the terrace. Both sections have an exposed concrete terrace roof which features a dugout area along the east edge of the roof. The concrete slab roof depth varies between 3" to 6" and spans one-way between 6"x12" concrete joists at 3'-0" on center. The joists span between concrete beams that bear on concrete bearing walls at the ends and intermediate concrete columns at the interior and in the covered walkway. The slabs, joists, and beams were cast in place monolithically. An octagonal built-up roof over the stairs has a roof of 2x tongue and groove wood sheathing over wood purlins bearing on a steel space frame which is mounted to concrete capitals atop concrete shear walls.

<u>Building condition</u>: During the site visit, the facility coordinator for the building indicated that the roof has had a consistent leaking problem over the years. Given the age of the structure, all structural steel frame and connections appeared to be in relatively good condition. No significant cracks were observed on the visible concrete wall faces. Bug holes and rock pockets from the initial concrete casting and stains from efflorescence were observed on the exterior face of the walls on the east side. Most of the interior faces of the concrete walls are covered by furring with limited visibility. The concrete slab is covered by a hardwood floor and was therefore not observed.

<u>Identification of levels:</u> The building has one story with two roof elevations: low roofs as pedestrian terraces over the rectangular sections and the high roof over the general purpose room. The site slopes down from west to the east, so the first story walls are on-grade on the east side and 12'-0" below grade on the west side. The roof terrace of the rectangular sections is level with the west higher grade; whereas the top of the roof of the general purpose room is another 23'-0" above the roof terrace.

<u>Foundation System:</u> In the general purpose room, the walls around the perimeter bear on a continuous strip footing. Below the columns, which are distributed within the concrete walls, are 3'-0" diameter by 9'-0" minimum deep circular reinforced concrete caissons. These caissons are tied together with 1'-2" x 1'-2" grade beams that are reinforced with #3 stirrups at 16" o.c. The structural drawings provide a summary of the geotechnical boring logs and show sands and clays over the top of white marble which varies in elevation. Foundation details specify that the caissons are to be embedded at least 1'0" into the top of the white marble formation.

In the two rectangular sections, the exterior shear walls are supported by strip footings on grade and are doweled into similar size reinforced concrete caissons that typically occur at wall ends and/or intersections. The concrete columns, both interior ones inside the locker rooms and exterior ones supporting the covered walk, are supported by concrete caissons. Interior concrete walls, including the shear walls around the stairwell, are supported by 1'-2" wide continuous strip footing with #3 transverse dowels @ 12" o.c into the concrete slab and roof.

<u>Structural system for vertical (gravity) load:</u> In the general purpose room, the copper roof panels and rigid insulation panels are supported by the built-up purlins that are comprised of a glulam made of three 2x6s. These purlins are bolted to the steel angles that are welded onto the face of the steel space truss tube elements. The steel space truss system is comprised of 6x4x1/2, 6x6x3/8, 4x4x3/8 and 3 1/2x3 1/2x1/4 steel tube elements that welded or bolted together and converge to 6x6x1/2 stub columns that are welded to ¼" thick steel plates secure atop precast concrete capitals with four 1 1/8" dia. x 2'-0" anchor bolts. The 1'-9"wide by 6'long by 2'deep concrete capitals are reinforced with four #4 wall vertical bars extended up continuously at the center; another four 1'-8" longitudinal #4 bars are placed, one at each corner, and wrapped with three #4 transverse stirrups. The typical concrete columns in the general purpose room are 10"x28" reinforced with five #11 longitudinal bars and confined with a set of two #3 stirrups spaced at 12" o.c. Two 8'-0" #11 bars are added on the interior face of the column, presumably for the outward thrust force from the space truss. Two #4 bars are placed at the center of the column to match the typical wall and connect with the #4 or #5 dowel bars through the column/wall joint at 10" or 12" o.c.

In the two rectangular sections, the concrete roof slab, the concrete joists and the concrete beams were monolithically cast in place. The slab is reinforced with #3 bars spaced @ 12" o.c. each way, and its depth varies from 6" at center to 3" towards both exterior edges. The concrete joists, spaced at 3'-0" o.c. and typically 6" wide by 12" deep below bottom of concrete slab, are reinforced with two #4 longitudinal bars running continuously top and bottom. The concrete beam sizes vary, either 35" wide x 12" deep, 24" wide x 12" deep or 31" wide x 18 ½" deep measuring below bottom of the slab. They are typically reinforced with two continuous #7 or #8 longitudinal bars top and bottom, which are tied together with #4 "U"-bars located at 18" on center. #10 and #11 longitudinal bars are used instead at Beam B2 which is a single-span beam. At the beam ends, the aforementioned longitudinal bars extend and bend into the outside of the wall reinforcement with a 90-degree standard hook. Standard hook dowels are placed at all typical slab/wall, joist/wall and beam/wall joints. Per Details Y/S-2 and Z/S-2, bottom bars in the east-west joists and north-south beams typically have a short lap on top of the columns and are thus not continuous across the columns.

Structural system for lateral forces: The lateral force-resisting system uses concrete diaphragms at the terrace level to span to reinforced concrete shear walls in both orthogonal directions of the building. These shear walls are placed around the full perimeter of the general purpose room, around the two rectangular sections, and also around the hexagon stair well. The exterior shear walls are, in the N-S direction, 12” thick with double layer #4 bars spaced at 12” o.c. and 8” thick in the E-W direction with single layer #4 bars spaced at 10” o.c. Interior shear walls are 8” thick, typically reinforced with a single layer of #4 bars spaced at 10” o.c. or a double layer of #4 bars spaced at 18” o.c. The shear walls around the general purpose rooms are doweled to concrete columns at each ends; whereas the shear walls in the rectangular sections are uniformly reinforced without specially detailed boundary elements with concentrated vertical reinforcing. Per the ASCE 41-17 Tier-1 Quick Check, the shear stress demands in the concrete shear walls are 14 psi in the N-S direction and 17 psi in the E-W direction, which is 11 and 15 percent, respectively, of the estimated shear capacity of the wall at 122 psi.

In 1988, a 12’-0” long 9” thick concrete shear wall was added to the east perimeter of the north rectangular section during a seismic upgrade, which helped reduced inherent torsion due to the concentration of N-S direction shear walls towards the east side of the building.

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

Identified seismic deficiencies of the building include the following:

- The high roof of the general purpose room creates a vertical irregularity, as well as a large opening in the diaphragm. However, the concrete walls bound the general purpose room and rectangular portions, so significant issues from out-of-phase behavior between the general purpose room and rectangular portions are not expected. Loads from the high roof steel structure are delivered at concentrated points from supporting tubes through baseplates and anchor bolts to the column capitals which rise 2’0” above the concrete shear walls at clerestory windows. The capitals have some confinement, and they have adequate capacity to transfer the loads from the steel truss to the column and down to the wall, conservatively assuming they are force-controlled elements.
- The separation between the North Building and the addition to the south is shown as 1” on the addition drawings which is less than the 2.2” required by the ASCE 41-17 Tier 1 Quick Check, but the structures align at the concrete terrace levels and they are relatively stiff shear wall structures. Damage from pounding is considered a comparatively low concern.

The space truss roof system appears to be in good condition and is positively anchored down to shear walls lines below. Due to its relative light weight and flexibility compared with the concrete below, the building will act as a podium structure so that the dynamic base of the steel truss assembly will be effective the concrete terrace roof, and inelastic demands on the truss elements are anticipated to be limited.

Overall, the reinforced concrete shear walls contain reasonable reinforcing ratios and ductile connection details that can help transfer the diaphragm forces to the walls. The lateral shear demands at BSE-C level are less than 15% of the nominal capacity per ASCE 41-17 Tier 1 Quick Check (See Appendix D). The 1988 seismic upgrade improved the seismic performance of the building and reduced the inherent torsion of the original wall system.

Most concrete columns are flexurally controlled per ASCE 41-17 Tier 1 Quick Check (See Appendix D), meaning they are expected to remain ductile and should be able to develop their full capacity during an earthquake event at BSE-C level. The one interior column located in the locker room is shear critical, but it still contains sufficient capacity to resist the shear force induced by drift of the building conservatively assuming a fixed-fixed end condition and a ½” assumption of drift.

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	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)	Y	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.[3]

No nonstructural life safety concerns were identified, but it is not known if gas-fueled equipment such as heaters and boilers are used.

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.	Unknown

Basis of rating

The building is assigned a Seismic Performance Level rating of IV. The short columns in between at the clerestory windows at the roof of the general purpose room have adequate capacity to resist the demands, and the lack of a compliant seismic separation at the adjacent building to the south is not considered to be a significant enough deficiency to reduce the rating as the concrete levels align.

Recommendations for further evaluation or retrofit

None.

Peer review of rating

This seismic evaluation was discussed in a peer review meeting on 28 May 2019. Reviewers present were Joe Maffei of Maffei Structural Engineering and Robert Graff of Degenkolb Engineers. Comments from the reviewers have been incorporated into this report. The reviewers agreed with the assigned rating.

Additional building data	Entry	Notes
Latitude	36.994242	
Longitude	-122.055037	

[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 if and where nonstructural hazards may occur.

Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	1	The building is below grade on the west side more than ½ level
Number of stories (basements) below lowest perimeter grade	0	
Building occupiable area (OGSF)	13,716	
Is the building on a sloping site?	Y	The difference in foundation embedment depth from the east side of the building to the west side equals to one story
Risk Category per 2016 CBC Table 1604.5	II	
Estimated fundamental period	0.14 sec	Estimated using ASCE 41-17 equation 4-4 and 7-18
Building structural height, h_n	14 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
Site data		
975-year hazard parameters S_s, S_1	1.281, 0.485	From SEAOC/OSHPD website
Site class	D	
Site class basis	Geotech[4]	See footnote below
Site parameters F_a, F_v	1.0, 1.815	From SEAOC/OSHPD website
Ground motion parameters S_{cs}, S_{c1}	1.281, 0.880	From SEAOC/OSHPD website
S_a at building period	1.28	
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	

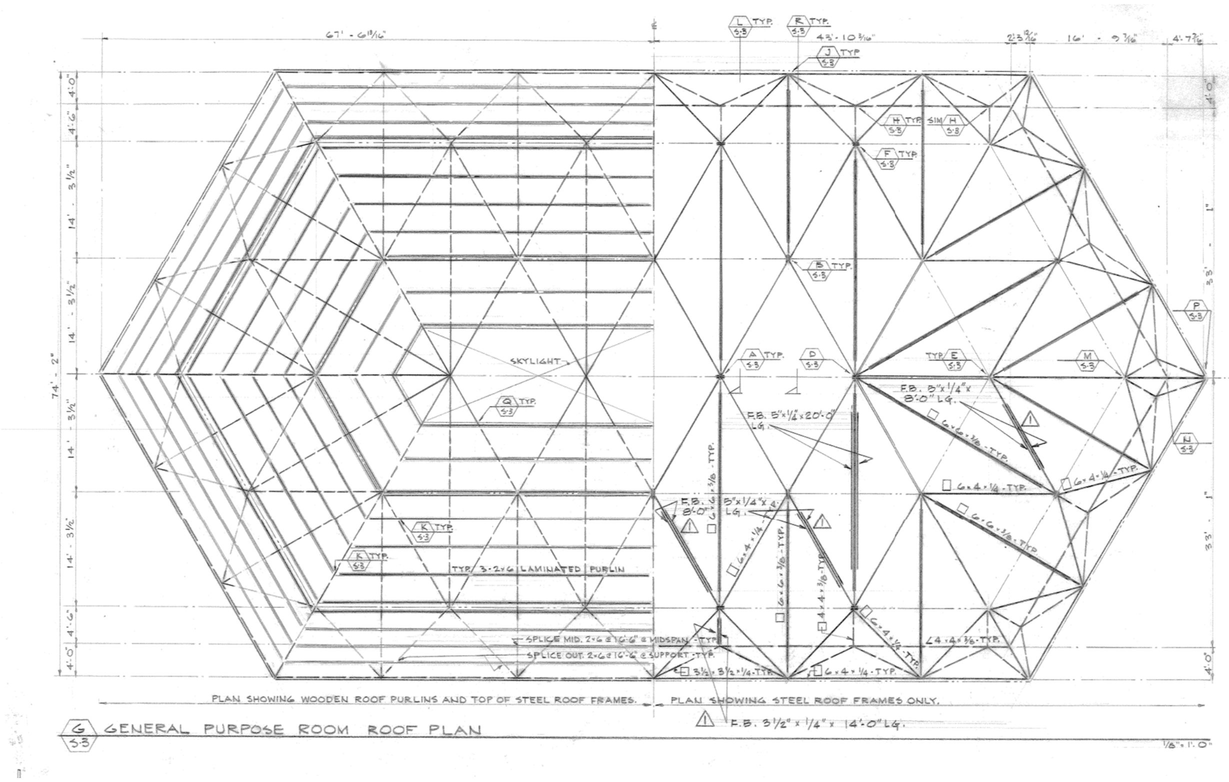
[4] Determination of site class and assessment of geotechnical hazards are based on correspondence with Pacific Crest Geotechnical Engineers and Nolan, Zinn, and Associates Geologists. [*Revised Geology and Geologic Hazards, Santa Cruz Campus, University of California*, Job # 04003-SC 13 May 2005]. Site 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

Applicable code		
Applicable code or approx. date of original construction	Built: 1964 Code: 1961 UBC	
Applicable code for partial retrofit	1985 UBC	Seismic Upgrade in 1988
Applicable code for full retrofit	None	No full retrofit
FEMA P-154 data		
Model building type North-South	C2 -Concrete Shear Wall	
Model building type East-West	C2 -Concrete Shear Wall	
FEMA P-154 score	N/A	Not included here because we performed ASCE 41 Tier 1 evaluation.
Previous ratings		
Most recent rating	III (Good)	
Date of most recent rating	1998	
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

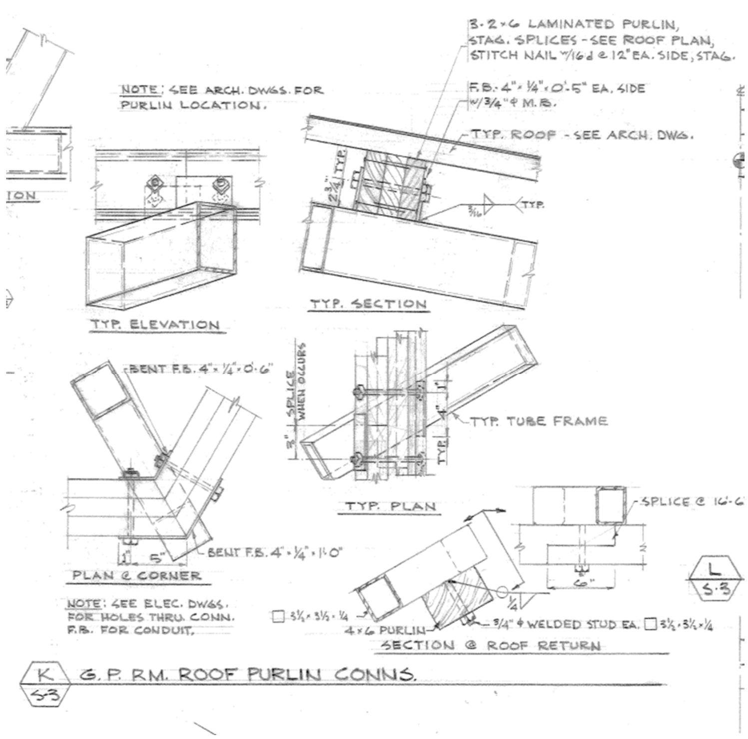
Color Coded Floor Plan

[redacted]

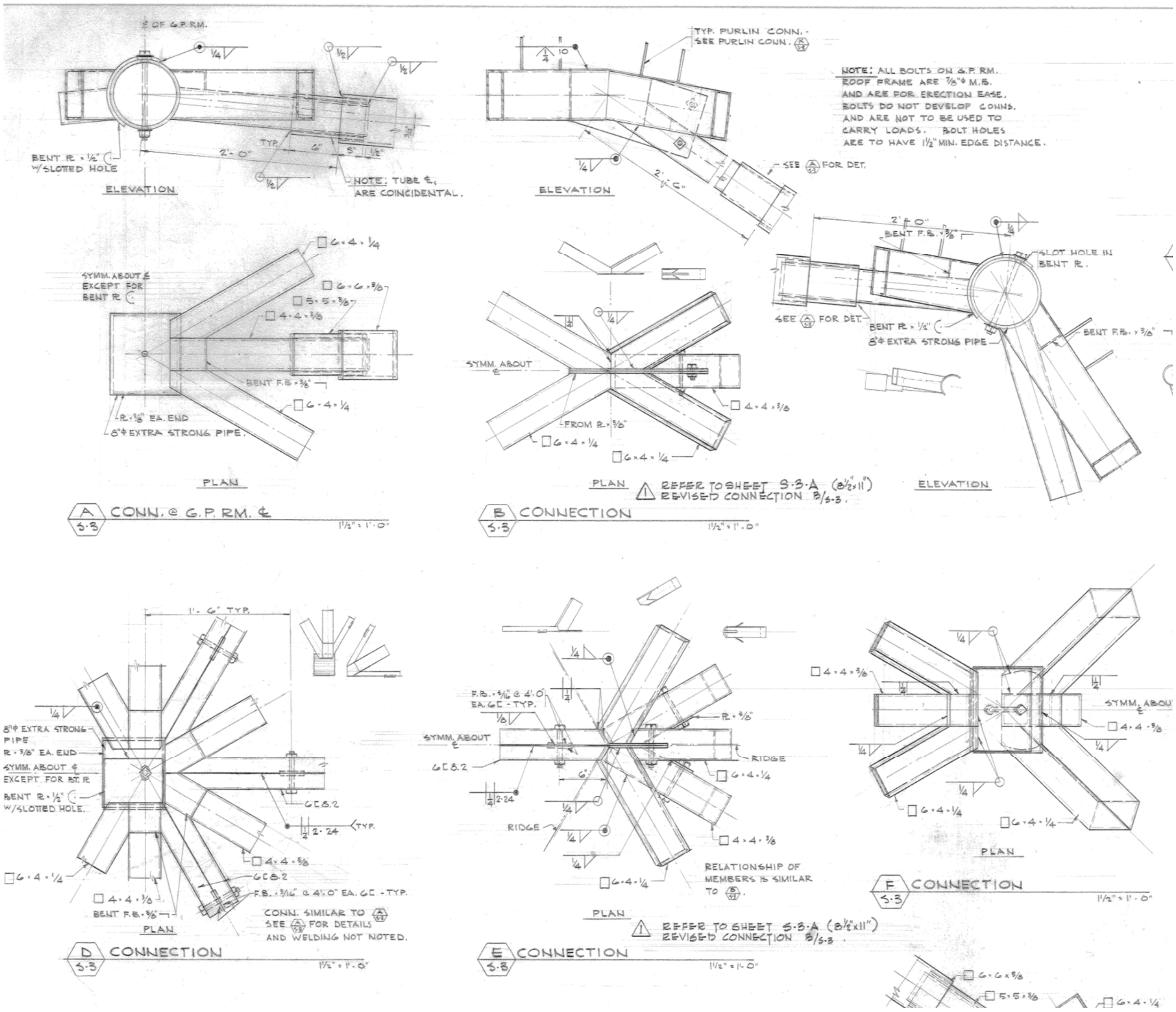
Roof Plan over General Purpose Room from Sheet S-3



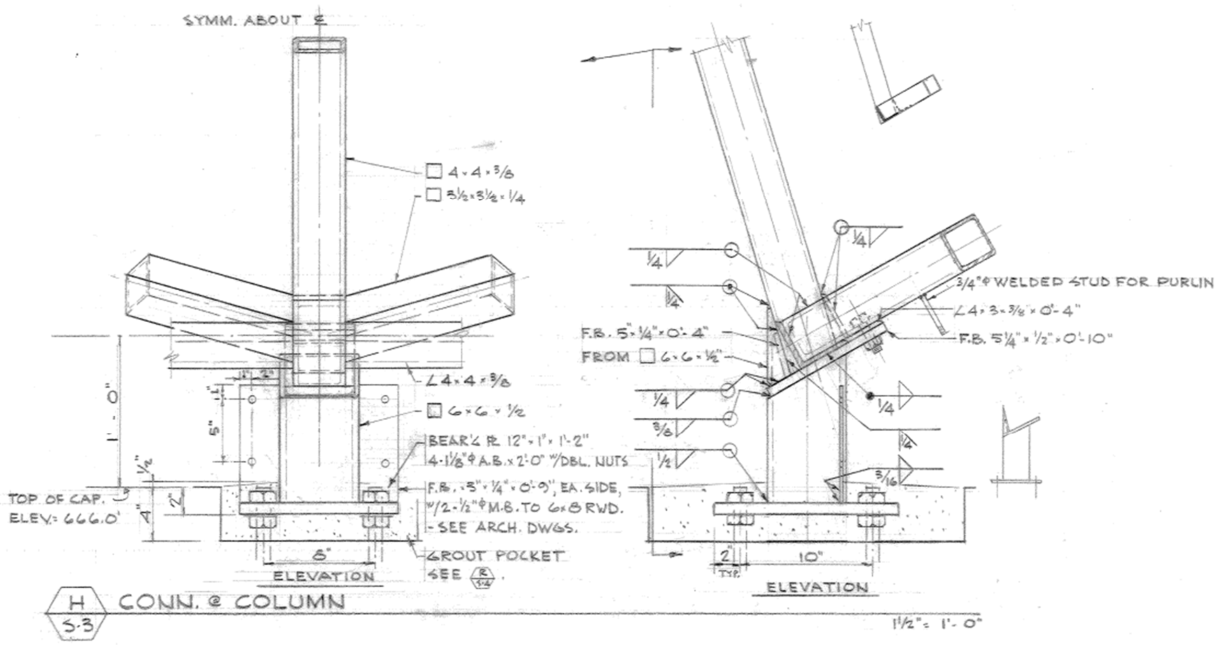
General Purpose Room Roof Details for Connection from Steel Truss to Wood Purlins



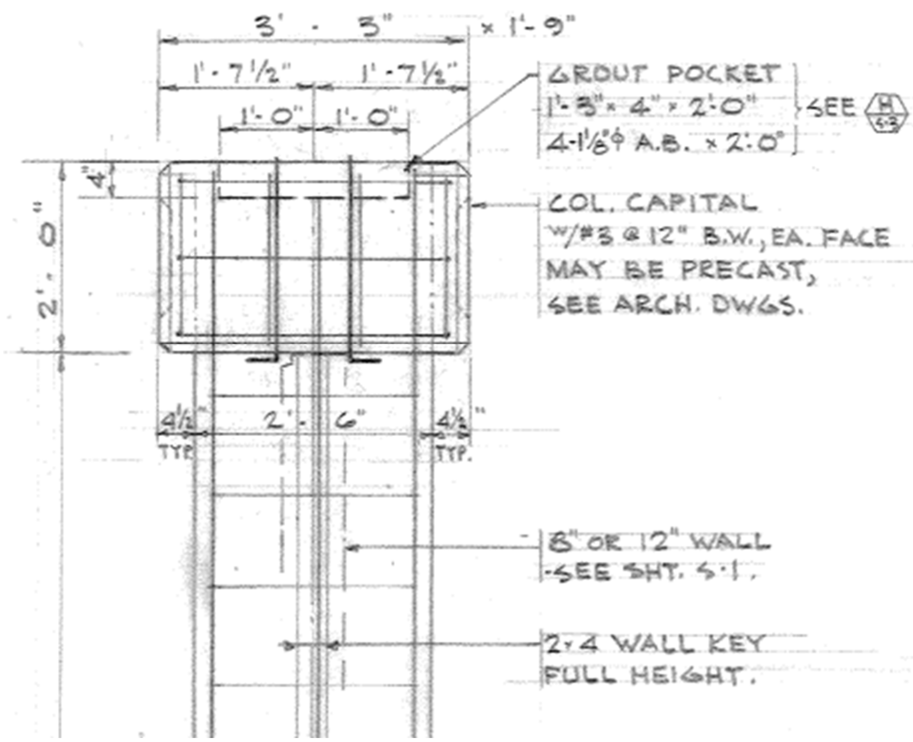
General Purpose Room Roof Truss Connection Details



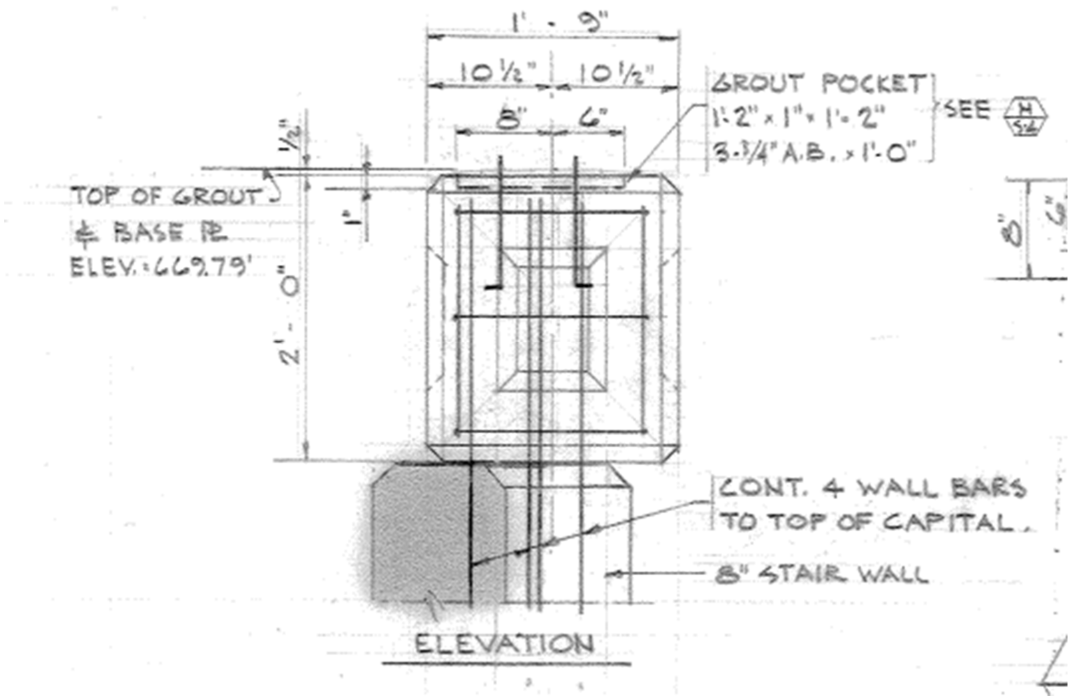
General Purpose Room Roof Truss Connection to Top of Concrete Column



Capital at Top of Concrete Column (from Sheet S-4)



Capital on top of Column Supporting Steel Roof Truss (Detail L/S-4)



APPENDIX A

Additional Photos

South Elevation of Clerestory Windows at General Purpose Room
(Looking Southwest from South Roof Terrace)

Northwest Corner of General Purpose Room
(Looking Southeast on West High Grade Level)

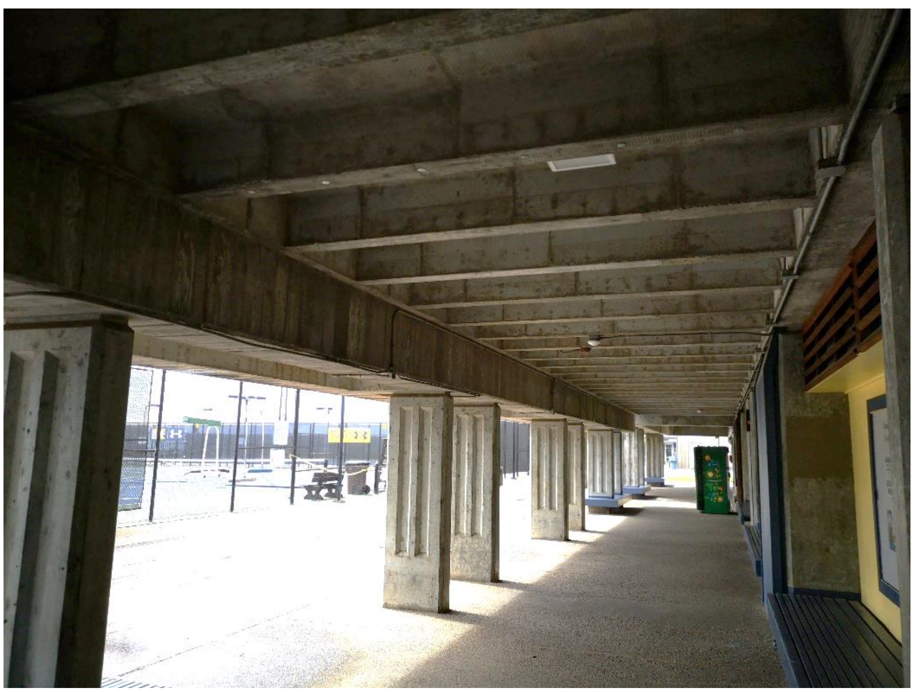
Northeast Corner of General Purpose Room
(Looking West From East Lower Grade Level)

Southeast Corner of General Purpose Room
(Looking West from East Lower Grade Level)

East Elevation of South Rectangular Section
(Looking Northwest)

Added Shear Wall in 1988 Seismic Upgrade
(Looking Northwest)

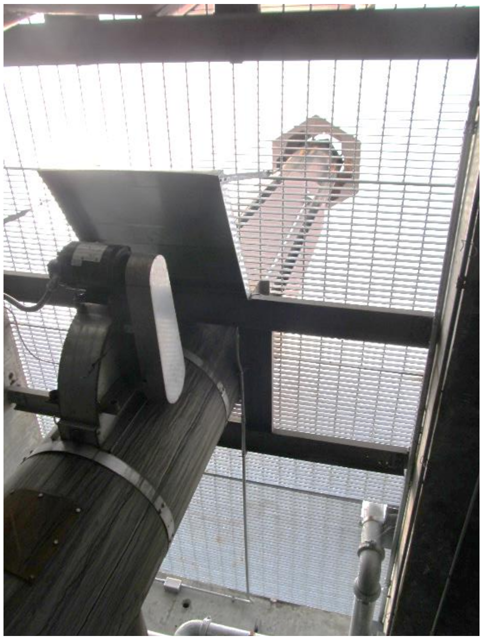
Steel Space Truss in General Purpose Room



Under South Covered Walkway in South Rectangular Section (Looking Southeast)

Dugout Feature along the East Edge of Roof Terrace
(Looking Southeast from North Roof Terrace)

Steel Flues and Stairwell Canopy
(Looking North from South Roof Terrace)




Steel Flues Extending Through Metal Grating and Base Connection

APPENDIX B

ASCE 41-17 Tier 1 Checklists (Structural)

UC Campus:	Santa Cruz		Date:	06/28/2019	
Building CAAN:	7119	Auxiliary CAAN:	By Firm:	Rutherford + Chekene	
Building Name:	East Field House North Building		Initials:	JY	Checked: WAL/BL
Building Address:	Santa Cruz, CA 95064		Page:	1 of 3	

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

LOW SEISMICITY

BUILDING SYSTEMS - *GENERAL*

	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:** Steel space frame roof structure delivers loads to concrete capitals and concrete shear walls founded on concrete caissons in general purpose room. In the north and south rectangular sections, concrete slabs deliver loads to concrete shear walls.
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:** A building addition was built to the south of the field house. Structural drawings for the addition indicate there is a 1” seismic expansion joint which his less than 12’0” x 0.015 =2.2” as required for high seismicity. However, the buildings align vertically at the terrace level concrete slabs.
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:** There are no mezzanines.

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

	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:** Single story structure.
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:** Single story structure.
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:** All shear walls are continuous to the foundation.

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

UC Campus:	Santa Cruz			Date:	06/28/2019	
Building CAAN:	7119	Auxiliary CAAN:		By Firm:	Rutherford + Chekene	
Building Name:	East Field House North Building			Initials:	JY	Checked: WAL/BL
Building Address:	Santa Cruz, CA 95064			Page:	2 of 3	

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

C NC N/A U	
NC	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:** Although the building is programmatically a single-story structure, the general purpose room continues to rise up from the terrace room, creating a vertical irregularity. The steel framed roof over the general purpose room is supported by concrete shear walls at the first story without any discontinuities
C	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:** Single story structure. Weight of the high roof over the general purpose room is well below a 50% of the terrace roof.
C	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 shear walls of N-S direction in the south rectangular section are concentrated to the west side, rendering the center of the rigidity closer to the west exterior wall line – 28 feet from centerline which exceeds the limit of 20% of the building width 25'-0". However, in the general purpose room, with the symmetrical wall layout, the center of rigidity is at the center. In the north rectangular section, a new wall is added to the east exterior wall line to help move the center of rigidity closer to the centerline. When all walls are combined, the center of rigidity of the whole building is expected to be within the 25 feet limit.

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

GEOLOGIC SITE HAZARD

C NC N/A U	Description
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:** Per 2009 County map at https://gis.santacruzcounty.us/mapgallery/Emergency%20Management/Hazard%20Mitigation/LiquifactionMap2009.pdf
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 2009 County map at https://gis.santacruzcounty.us/mapgallery/Emergency%20Management/Hazard%20Mitigation/LandslideMap2009.pdf

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

UC Campus:	Santa Cruz		Date:	06/28/2019
Building CAAN:	7119	Auxiliary CAAN:	By Firm:	Rutherford + Chekene
Building Name:	East Field House North Building		Initials:	JY / Checked: WAL/BL
Building Address:	Santa Cruz, CA 95064		Page:	3 of 3

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	
○	○	○	●	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 2009 County map at https://gis.santacruzcounty.us/mapgallery/Emergency%20Management/Hazard%20Mitigation/FaultZoneMap2009.pdf

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

FOUNDATION CONFIGURATION

C	NC	N/A	U	Description
●	○	○	○	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:** Building width B = 51.5', Building Height is H = 32', B/H = 1.6 Sa = 1.54g per ATC at BSE-2E 0.6 x Sa = 0.924 B/H > 0.6 Sa
●	○	○	○	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:** Site D is assumed. Concrete grade beams ties together caissons in the general purpose room. In the north and south rectangular sections, the concrete slab-on-grade spans between concrete strip footings.

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

UC Campus:	Santa Cruz			Date:	6/28/2019	
Building CAAN:	7119	Auxiliary CAAN:		By Firm:	RUTHERFORD + CHEKENE	
Building Name:	East Field House North Building			Initials:	JY	Checked: WAL/BL
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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type C2-C2A

Low And Moderate Seismicity

Seismic-Force-Resisting System

C / NC / N/A / U	Description
NC	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5.2.5.1) **Comments:** Loads from the space frame are supported by concrete capitals and columns in the general purpose room. In the rectangular sections, the north and south exterior walls are load-bearing. They do not have embedded columns in the walls at the beam locations with additional vertical reinforcing and column ties per the typical column details.
C	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:** There are more than 2 lines of walls in each direction.
C	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1) **Comments:** Calculated wall stresses are less the ASCE 41 limit of 123 psi for f'c = 3,750 psi – wall average shear stresses in the transverse direction are 21 psi (E-W direction) and 17 psi (N-S direction).
C	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3) **Comments:** Per wall section details in Sheet No. S-1, at a minimum, ρ= 0.0025 (#4 @ 10" o.c., e.w. in 8" thick walls, #4 @ 18" o.c., e.w., e.f. in 8" thick walls, and #4 @ 12" o.c., e.w., e.f. in 12" thick walls).

Connections

C / NC / N/A / U	Description
N/A	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1) **Comments:** The building has rigid diaphragms.
C	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:** Per Sheet S-2, edge beams are typically cast on top of shear walls. The slab/joist reinforcing passes through the edge beams into the wall with hooks oriented in the vertical direction. These hooks are located at the exterior face of the exterior walls. Also 6" typical wide continuous shear keys are provided at the beam/wall joint.

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

UC Campus:	Santa Cruz		Date:	6/28/2019	
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Building Address:	Santa Cruz, CA 95064		Page:	2	of 3

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

C NC N/A U	
(C selected)	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing directly above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4) **Comments:** Per Details A, B, C & D in Sheet S-1, dowels extend up out of the strip footings and caissons match vertical wall reinforcing.

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
(C selected)	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:** All columns have the shear capacity to develop their flexural strength, except one which is on the margin between being shear and flexurally critical, and which has the capacity to drift significantly more than ½" using conservative fixed-fixed end conditions before reaching its shear capacity
(N/A selected)	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:** There are no flat slabs.
(N/A selected)	COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1) **Comments:** There are no coupling beams.

Diaphragms (Stiff Or Flexible)

C NC N/A U	Description
(C selected)	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:** There are no split levels.
(NC selected)	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:** There is a stair opening in the roof diaphragm in the south rectangular section. The stairwell is built with full-height concrete shear walls that are tied to the roof diaphragm.

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

Flexible Diaphragms

C NC N/A U	Description
N/A	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) **Comments:** The terrace level has rigid concrete diaphragms, and the high roof has a complete steel space frame..
N/A	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 terrace level has rigid concrete diaphragms, and the high roof has tongue and groove sheathing spanning short distances to the steel space frame.
N/A	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 terrace level has rigid concrete diaphragms, and the wood at the high roof spans less than 24 feet between the steel space truss.
N/A	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 terrace level has rigid diaphragms, and the high roof has tongue groove sheathing.
N/A	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 diaphragms in the building are either concrete, wood, or steel bracing.

Connections

C NC N/A U	Description
N/A	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:** Shear walls sit directly on and are anchored to the top of the caissons. There are no pile caps.

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

APPENDIX C

UCOP Seismic Safety Policy Falling Hazards Assessment Summary

UC Campus:	Santa Cruz			Date:	06/28/2019
Building CAAN:	7119	Auxiliary CAAN:		By Firm:	Rutherford + Chekene
Building Name:	East Field House North Building			Initials:	JY
				Checked:	WAL/BL
Building Address:	418 Hager Drive, Santa Cruz, CA 95064			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:** There are no heavy ceilings, features or ornamentation above the general purpose room. The structure is exposed.
P ☐ N/A ☒	**Heavy masonry or stone veneer above exit ways or public access areas** **Comments:** There heavy masonry or stone veneer above exit ways or public access areas. The structure is exposed above the covered walkways and general purpose room.
P ☐ N/A ☒	**Unbraced masonry parapets, cornices, or other ornamentation above exit ways or public access areas** **Comments:** There are no URM parapets, cornices or other ornamentation above exit ways or public access areas. The structure is exposed above the covered walkways and general purpose room.
P ☐ N/A ☒	**Unrestrained hazardous material storage** **Comments:** No hazardous material storage was observed.
P ☐ N/A ☒	**Masonry chimneys** **Comments:** There are no masonry chimneys.
P ☐ N/A ☒	**Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.** **Comments:** Unknown.

Falling Hazards Risk: Low

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

APPENDIX D

Quick Check Calculations

Rating form completed by:
RUTHERFORD + CHEKENE
ruthchek.com
Evaluator: JY/WAL/BL
Date: 06/28/2019

Unit Weights:

Flat Load Tables

Rectangular Sections	Seismic Weight psf	Dead Load psf	Remarks
Slab	50	50	Exposed 4"(average depth) slab
Joists and Beams	35	35	CIP Joists and Beam -
Sprinklers, lighting and misc.	2	2	MEP hung from underside of floor slab
Columns	5	0	RC columns
Total-typical roof	92	87	
Total-dugout region	142	137	8" thick slab

General Purpose Room	Seismic Weight psf	Dead Load psf	Remarks
Metal Roof	1	1	copper roof per ASCE 7-16
Rigid Insulation	2	2	2" rigid insulation
Laminated purlins	12	12	(3) 2x6 purlins @3'-6" on center
Beams/girders	21	21	Steel wide flange girders below 6" slab
Sprinklers, lighting and misc.	2	2	MEP hung from underside of floor slab
Columns	5	0	Steel wide flange column
Total	42	38	

Rating form completed by:

RUTHERFORD + CHEKENE
ruthchek.com

Evaluator: JY/WAL/BL

Date: 06/28/2019

Story Weights:

wconcrete = 150 psf

Floor Levels	Floor Area (ft^2) [1,2] Rectangular Sections	Floor Area (ft^2) [1,2] General Purpose Room	Floor Weight (psf) Rectangular Sections	Floor Weight (psf) General Purpose Room	Wall Weight: Wall height below floor level (ft)	Wall Weight: Wall height tributary to each floor level (ft)	Wall Weight: Wall Area below (ft^2)	Wall Weight: Wall Seismic Weight (kips)	Additional Weight (kips) [3]	Total Seismic Weight (kips)
Roof	6,753	6,963	92	42	12.00	6.00	516	464	88	1,470
1st Floor	6,753	6,963	92	42	0.00	6.00	0	464	0	1,382

Notes:
1 - Seismic base is set at the 1st floor. Soil-structure interaction is ignored for ASCE 41-17 Tier 1.
2 - Calculation for the wall seismic weight at the roof is provided below:

Wall ID	Rectangular Sections: Thickness (in)	Rectangular Sections: Length (ft)	Rectangular Sections: Area (ft^2)	General Purpose Room: Thickness (in)	General Purpose Room: Length (ft)	General Purpose Room: Area (ft^2)
1F - 1X	8	43.6	0.0	0	0	0.0
1F - 2X	8	12.375	55.6	8	83.375	0.0
1F - 3X	8	8.25	52.8	8	79.25	0.0
1F - 4X	8	19.33	0.0	0	0	0.0
1F - 5X	8	30.02	0.0	0	0	0.0
1F - 6X	12	8.33	0.0	0	0	0.0
1F - 7X	8	19.02	0.0	0	0	0.0
1F - 8X	8	42	0.0	0	0	0.0
1F - 1Y	12	23.25	23.3	0	0	0.0
1F - 2Y	9	12	9.0	0	0	0.0
1F - 3Y	0	0	0.0	12	66	66.0
1F - 4Y	0	0	0.0	12	66	66.0
1F - 5Y	12	25.67	25.7	0	0	0.0
1F - 6Y	12	21	21.0	0	0	0.0
1F - 7Y	8	21	14.0	0	0	0.0
1F - 8Y	12	15.33	15.3	0	0	0.0
1F - 9Y	8	32	21.3	0	0	0.0
		Σ =	238.0		Σ =	132.0

Wall height above roof = 0.00 ft
Wall height below roof = 12.00 ft

Wall area above roof = 0 ft^2
Wall area below roof = 370.0 ft^2

$w_{concrete}$ = 0.15 kcf

$$Wall\ seismic\ weight\ at\ roof = w_{concrete} \times \left(Area_{below} \times \frac{Height_{below}}{2} + Area_{above} \times \frac{Height_{above}}{2} \right)$$

Wall seismic weight = 333 kips

3 - Additional weight includes the following items:
- Concrete parapet: 8" wide x 2'-0" high concrete curbs around the perimeters of roof for rectangular section

Floor Levels	Concrete parapet: Thickness (in)	Concrete parapet: Length (ft)	Concrete parapet: Weight (plf)	Concrete parapet: Weight (kips)	Total (kips)
Roof	8	350.6	250	87.6	87.6
1st Floor					0.0

Rating form completed by:

RUTHERFORD + CHEKENE
ruthchek.com

Evaluator: JY/WAL/BL

Date: 06/28/2019

Period:

C_t=	0.02
h_n (ft)=	14.00
B=	0.75

T=	0.14	sec

Notes:
1- The period calculated per ASCE 41-17 Equation 4-4.

$$T = C_t \cdot h_n^{B}$$

2- Ct and B are for "all other framing system" per ASCE 41-17 Section 4.4.2.4.
3- The building height is taken from the 1st floor to the roof.

Rating form completed by:

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Evaluator: JY/WAL/BL

Date: 06/28/2019

BSE-2E Site Parameters:

OSHPD

7119

Latitude, Longitude: 36.994539, -122.061018

Date	5/30/2019, 3:44:43 PM
Design Code Reference Document	ASCE41-17
Custom Probability	
Site Class	D - Stiff Soil

Type	Description	Value
Hazard Level		BSE-2N
S_S	spectral response (0.2 s)	1.631
S_1	spectral response (1.0 s)	0.625
S_{XS}	site-modified spectral response (0.2 s)	1.631
S_{X1}	site-modified spectral response (1.0 s)	1.063
F_a	site amplification factor (0.2 s)	1
F_v	site amplification factor (1.0 s)	1.7
ssuh	max direction uniform hazard (0.2 s)	1.754
crs	coefficient of risk (0.2 s)	0.93
ssrt	risk-targeted hazard (0.2 s)	1.631
ssd	deterministic hazard (0.2 s)	3.017
s1uh	max direction uniform hazard (1.0 s)	0.686
cr1	coefficient of risk (1.0 s)	0.911
s1rt	risk-targeted hazard (1.0 s)	0.625
s1d	deterministic hazard (1.0 s)	1.027

Type	Description	Value
Hazard Level		BSE-1N
S_{XS}	site-modified spectral response (0.2 s)	1.087
S_{X1}	site-modified spectral response (1.0 s)	0.708

Type	Description	Value
Hazard Level		BSE-2E
S_S	spectral response (0.2 s)	1.281
S_1	spectral response (1.0 s)	0.485
S_{XS}	site-modified spectral response (0.2 s)	1.281
S_{X1}	site-modified spectral response (1.0 s)	0.88
f_a	site amplification factor (0.2 s)	1
f_v	site amplification factor (1.0 s)	1.815

Type	Description	Value
Hazard Level		BSE-1E
S_S	spectral response (0.2 s)	0.689
S_1	spectral response (1.0 s)	0.242
S_{XS}	site-modified spectral response (0.2 s)	0.86
S_{X1}	site-modified spectral response (1.0 s)	0.513
F_a	site amplification factor (0.2 s)	1.249
F_v	site amplification factor (1.0 s)	2.115

Type	Description	Value
Hazard Level		T-Sub-L Data
T-Sub-L	Long-period transition period in seconds	12

Rating form completed by:

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Evaluator: JY/WAL/BL

Date: 06/28/2019

Story Shears

Sa=	1.28	
W=	1,470	kips
C=	1.4	Per ASCE 41-17 Table 4-7

V=	2,636	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	12.00	12.00	1,470	17,636	1.00	2,636	2,636
1st Floor	0.00	0.00	1,382	0	0.00	0	2,636
				17,636	1	2,636	

Notes:

1- The base of building is assumed to be at the 1st floor.

2- 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.

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Evaluator: JY/WAL/BL
Date: 06/28/2019

Average Shear Stress in Concrete Shear Wall:

Average Stresses

Ms = 4.5
f'c = 3750 psi

Based upon General Structural Notes on Sheet S-4

Longitudinal (E-W direction)					
Story	Story Shear	Wall Area	Average Shear Stress	Tier 1 Shear Stress Limit	Wall OK?
	(kips)	(in^2)	(psi)	(psi)	
Roof - 1st Floor	2,636	33,573	17	122	OK

Transverse (N-S direction)					
Story	Story Shear	Wall Area	Average Shear Stress	Tier 1 Shear Stress Limit	Wall OK?
	(kips)	(in^2)	(psi)	(psi)	
Roof - 1st Floor	2,636	40,740	14	122	OK

Notes:
1 - Ms factor per ASCE 41-17 Table 4-8.

Table 4-8. M_s Factors for Shear Walls

	Level of Performance		
Wall Type	**CP**[a]	**LS**[a]	**IO**[a]
Reinforced concrete, precast concrete, wood, reinforced masonry, and cold-formed steel	4.5	3.0	1.5
Unreinforced masonry	1.75	1.25	1.0

[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.

Check of Column Capital

Seismic Force

$W = 38\,PSF \times 7235\,SF = 275\text{ kips}$

$V = Sa \cdot C \cdot W = 1.28 \times 1.4 \times 275^k = 492.8^k$

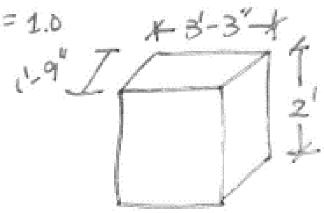
seismic force due to general purpose room roof weight into the capitals.

<u>Longitudinal (E-W) Direction</u>

Per Capital $\quad 492.8^k/12 = \underline{41.1^k}$

$\phi V_n = V_c + V_s \qquad V_c = 2\left(1 + \frac{N_u}{2000 A_g}\right) \cdot \lambda \sqrt{f'_c}\, b w d$

$\phi = 1.0$



$A_g = (1'\text{-}9") \times (3'\text{-}3") = 819\text{ in}^2$

$N_u = 275^k/24 = 11.5^k \sim 12^k$

$b_w = 39"$

$d = 24" - 3" = 21"$

$f'_c = 3750\text{ psi}$

$\therefore V_c = 101^k > 41.1^k$ ok.

No shear reinf. req'd

<u>Transverse (N-S) Direction.</u>

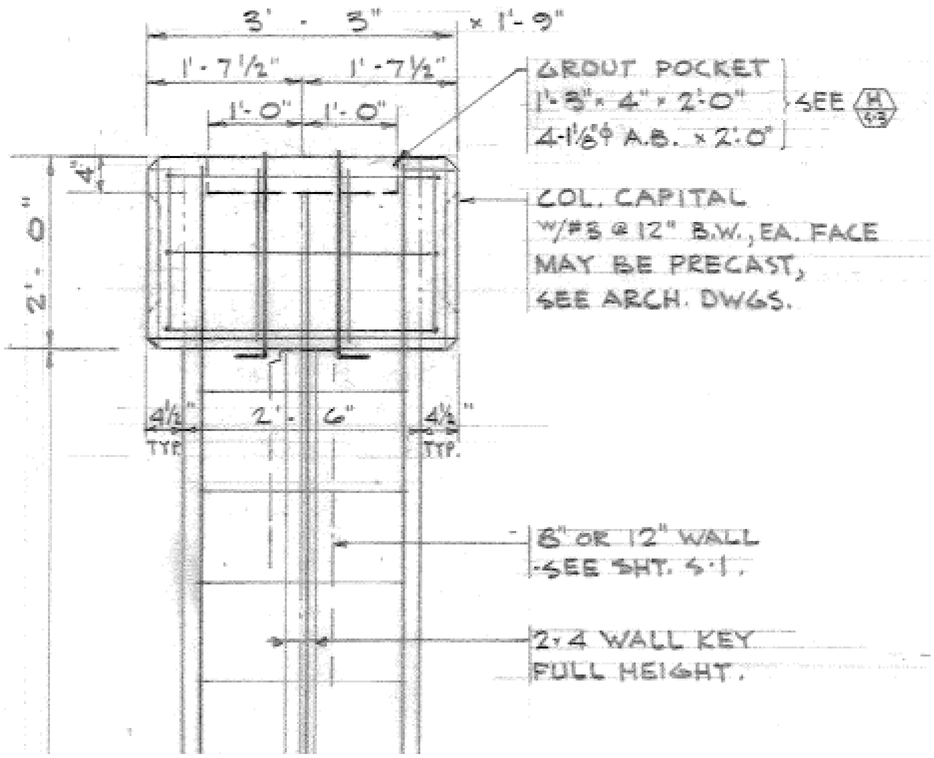
Per Capital $\quad 492.8^k/12 \times \sqrt{2} = 58^k$

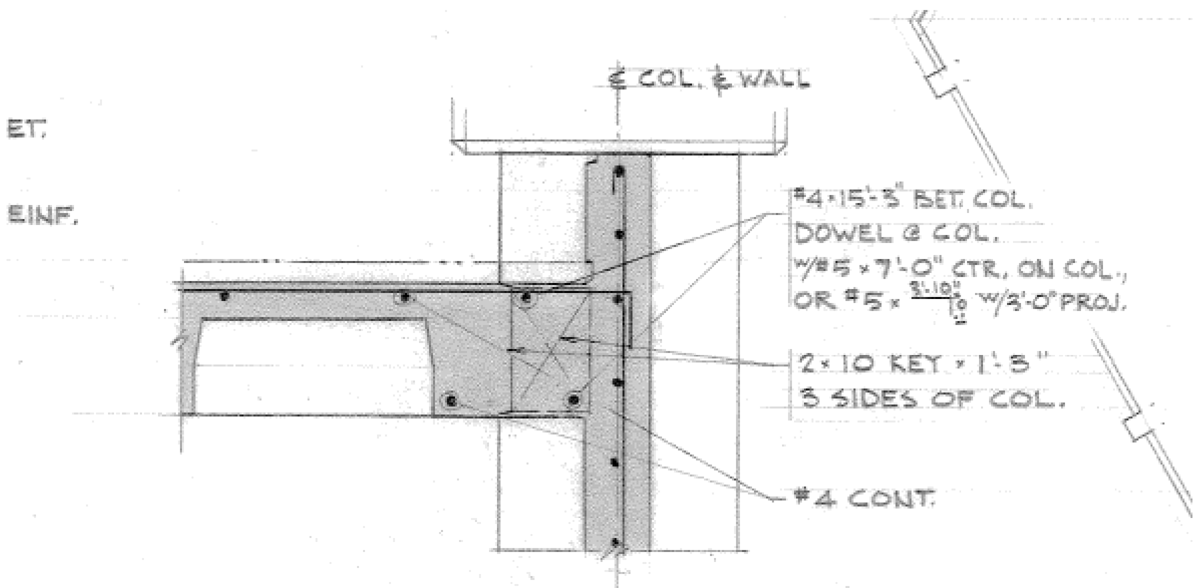
$b_w = 21"$

$d = 39" - 3" = 36"$

$\therefore V_c = 93.2^k > 58^k$ ok.

No shear reinf. req'd





P
S-2
SECTION
½" = 1'-0"

Rating form completed by:

RUTHERFORD + CHEKENE
ruthchek.com
Evaluator: JY/WAL/BL

Date: 06/28/2019

COLUMN DEFORMATION COMPATIBILTY (0.9DL)

Material properties

- Concrete	f'_c	3.75	ksi	Based upon General notes, Sheet S-4 in 1964 Structural drawings
- Steel rebar, longitudinal rebar	f_y	40	ksi	Based upon General notes, Sheet S-4 in 1964 Structural drawings

Other parameters

- Flexural ductility	k_{nl}	0.7
- Normal weight	λ	1.0

spColumn Model	Col Location (Column Type)[6]	Level	SIZE b (in)	SIZE h (in)	SIZE A_G (in²)	LONGITUDINAL n-#	LONGITUDINAL D (in)	LONGITUDINAL A_s (in²)	TRANVERSE n-#-s (in)	TRANVERSE D (in)	TRANVERSE A_v (in²)	TRANVERSE $f_{y,t}$ (ksi)
#F.7-6 (X-Dir)-1F-6#7	F.7-6 (X-Dir)	1	10	28	280.0	6 #7	0.875	3.6	3.0 #3 10	0.375	0.33	40
#E-5 (X-Dir)-1F-6#6	E-5 (X-Dir)	1	8	28	224.0	6 #6	0.75	2.7	3.0 #3 8	0.375	0.33	40
#F-5 (X-Dir)-1F-4#5	F-5 (X-Dir)	1	12	8	96.0	4 #5	0.625	1.2	3.0 #3 8	0.375	0.33	40
#F.7-6 (Y-Dir)-1F-6#7	F.7-6 (Y-Dir)	1	28	10	280.0	6 #7	0.875	3.6	3.0 #3 10	0.375	0.33	40
#E-5 (Y-Dir)-1F-6#6	E-5 (Y-Dir)	1	28	8	224.0	6 #6	0.75	2.7	3.0 #3 8	0.375	0.33	40
#F-5 (Y-Dir)-1F-4#5	F-5 (Y-Dir)	1	8	12	96.0	4 #5	0.625	1.2	3.0 #3 8	0.375	0.33	40

spColumn Model	DIMENSION d (in)[1]	DIMENSION h_b (in)	DIMENSION L (ft)	DIMENSION L_n (in)	CONFINEMENT s/d	CONFINEMENT a_{col}	FLEX. YIELD $L_n/2d_c$	FLEX. YIELD M_u/V_ud	FLEX. YIELD $6f^{0.5}/(M/Vd)$	AXIAL N_{UG} (k)[2]	AXIAL x (ft)	AXIAL y (ft)	AXIAL A_{trib} (ft²)	AXIAL $(1+N/6Af^{0.5})^{0.5}$
#F.7-6 (X-Dir)-1F-6#7	22.40	12.0	12.00	132.0	0.45	1.0	2.9	2.9	0.12	24	16.3	10.5	170.6	1.1
#E-5 (X-Dir)-1F-6#6	22.40	12.0	12.00	132.0	0.36	1.0	2.9	2.9	0.12	36	19.0	14.3	272.3	1.2
#F-5 (X-Dir)-1F-4#5	9.60	12.0	12.00	132.0	0.83	0.7	6.9	4.0	0.09	36	19.0	15.0	285.0	1.4
#F.7-6 (Y-Dir)-1F-6#7	8.00	12.0	12.00	132.0	1.25	0.0	8.3	4.0	0.09	24	16.3	10.5	170.6	1.1
#E-5 (Y-Dir)-1F-6#6	6.40	12.0	12.00	132.0	1.25	0.0	10.3	4.0	0.09	36	19.0	14.3	272.3	1.2
#F-5 (Y-Dir)-1F-4#5	6.40	12.0	12.00	132.0	1.25	0.0	10.3	4.0	0.09	36	19.0	15.0	285.0	1.4

spColumn Model	SHEAR V_S (k)[3]	SHEAR V_C (k)	SHEAR V_{col} (k)[4]	FLEXURE M_{P-0} (k-ft)[5]	FLEXURE M (k-ft)[5]	FLEXURE 2M/L (k)	SHEAR/FLEX. CONTROL	Vp/Vcol	ADDITIONAL CHECK (LIMITED BY DRIFT) E (ksi)	I_g (in⁴)	Δ (in)	L (in)	Vprob (k)[6]	Acceptance criteria[7]
#F.7-6 (X-Dir)-1F-6#7	29.7	31.1	42.5	189.4	218.2	39.7	Flexure	0.93	3490.5	9,366.2	0.5	144.0	32.8	-
#E-5 (X-Dir)-1F-6#6	37.1	26.8	44.7	140.0	182.5	33.2	Flexure	0.74	3490.5	7,492.9	0.5	144.0	26.3	-
#F-5 (X-Dir)-1F-4#5	10.6	10.1	14.5	17.6	26.7	4.9	Flexure	0.34	3490.5	884.7	0.5	144.0	3.1	-
#F.7-6 (Y-Dir)-1F-6#7	0.0	22.9	16.0	63.2	71.4	13.0	Flexure	0.81	3490.5	1,194.7	0.5	144.0	4.2	-
#E-5 (Y-Dir)-1F-6#6	0.0	19.8	13.8	38.4	47.4	8.6	Flexure	0.62	3490.5	611.7	0.5	144.0	2.1	-
#F-5 (Y-Dir)-1F-4#5	0.0	10.1	7.0	26.5	42.2	7.7	Shear	1.09	3490.5	174.8	0.5	144.0	0.6	OK

Notes:

1 - Effective depth, d, is computed as 0.8h, where h is the dimension of the column in the direction of shear.

2 - Based on 0.9DL.

3 - Per 10.3.4/ASCE 41-17, lap-spliced reinforcement in tied columns is assumed to be ineffective in regions of high ductility demand.

4 - Shear capacity of column is based on ASCE 41-17 Eq 10-3 using nominal material strengths with ϕ =1.0.

$$V_{col} = k_{nl} \cdot \left[\alpha_{col} \cdot \left(\frac{A_v \cdot f_{ytLE} \cdot d}{s} \right) + \lambda \cdot \left[\sqrt{1 + \frac{N_{UG}}{6 \cdot Ag \cdot \sqrt{f_{cLE}}}} \cdot \frac{6 \cdot (f_{cLE})}{\left(\frac{M_{UD}}{V_{UD} \cdot d} \right)} \right] \cdot 0.8 \cdot A_g \right]$$

α_{col} = 1.0 for s/d <= 0.75, 0.0 for s/d >=1.0

5 - Plastic moment capacity of the column is based upon expected flexural strength using 1.5 f'c and 1.25 fy.

6 - Shear induced due to drift of a fixed-fixed column.

$$V_{prob} = \frac{12E \cdot 0.5 \cdot I_g \cdot \Delta}{L^3}$$

7- Vprob is compared to Vcol. If Vprob <Vcol, Shear failure is not likely to occur.

COLUMN DEFORMATION COMPATIBILTY (1.1DL+0.275LL)

Material properties

- Concrete	f'_c	3.75	ksi	Based upon General notes, Sheet S-4 in 1964 Structural drawings
- Steel rebar, longitudinal rebar	f_y	40	ksi	Based upon General notes, Sheet S-4 in 1964 Structural drawings

Other parameters

- Flexural ductility	k_{nl}	0.7
- Normal weight	λ	1.0

spColumn Model	Col Location (Column Type)[6]	Level	SIZE			LONGITUDINAL				TRANVERSE						DIMENSION				CONFINEMENT	
			b (in)	h (in)	A_G (in^2)	n-#		D (in)	A_S (in^2)	n-#-s (in)			D (in)	A_v (in^2)	$f_{y,t}$ (ksi)	d (in)[1]	h_b (in)	L (ft)	L_u (in)	s/d	a_{col}
#F.7-6 (X-Dir)-1F-6#7	F.7-6 (X-Dir)	1	10	28	280.0	6	#7	0.875	3.6	3.0	#3	10	0.375	0.33	40	22.40	12.0	12.00	132.0	0.45	1.0
#E-5 (X-Dir)-1F-6#6	E-5 (X-Dir)	1	8	28	224.0	6	#6	0.75	2.7	3.0	#3	8	0.375	0.33	40	22.40	12.0	12.00	132.0	0.36	1.0
#F-5 (X-Dir)-1F-4#5	F-5 (X-Dir)	1	12	8	96.0	4	#5	0.625	1.2	3.0	#3	8	0.375	0.33	40	9.60	12.0	12.00	132.0	0.83	0.7
#F.7-6 (Y-Dir)-1F-6#7	F.7-6 (Y-Dir)	1	28	10	280.0	6	#7	0.875	3.6	3.0	#3	10	0.375	0.33	40	8.00	12.0	12.00	132.0	1.25	0.0
#E-5 (Y-Dir)-1F-6#6	E-5 (Y-Dir)	1	28	8	224.0	6	#6	0.75	2.7	3.0	#3	8	0.375	0.33	40	6.40	12.0	12.00	132.0	1.25	0.0
#F-5 (Y-Dir)-1F-4#5	F-5 (Y-Dir)	1	8	12	96.0	4	#5	0.625	1.2	3.0	#3	8	0.375	0.33	40	6.40	12.0	12.00	132.0	1.25	0.0

spColumn Model	FLEX. YIELD			AXIAL					SHEAR			FLEXURE			SHEAR/FLEX. CONTROL	Vp/Vcol	ADDITIONAL CHECK (LIMITED BY DRIFT)					
	$L_u/2d_c$	M_u/V_ud	$6f^{0.5}/(M/Vd)$	N_{UG} (k)[2]	x (ft)	y (ft)	A_{trib} (ft^2)	$(1+N/6Af^{0.5})^{0.5}$	V_s (k)[3]	V_c (k)	V_{col} (k)[4]	M_{p-0} (k-ft)[5]	M (k-ft)[5]	2M/L (k)			E (ksi)	I_g (in^4)	Δ (in)	L (in)	Vprob (k)[6]	Acceptance criteria[7]
#F.7-6 (X-Dir)-1F-6#7	2.9	2.9	0.12	32	16.3	10.5	170.6	1.1	29.7	31.9	43.1	189.4	218.2	39.7	Flexure	0.92	3490.5	9,366.2	0.5	144.0	32.8	-
#E-5 (X-Dir)-1F-6#6	2.9	2.9	0.12	47	19.0	14.3	272.3	1.3	37.1	28.0	45.6	140.0	182.5	33.2	Flexure	0.73	3490.5	7,492.9	0.5	144.0	26.3	-
#F-5 (X-Dir)-1F-4#5	6.9	4.0	0.09	48	19.0	15.0	285.0	1.5	10.6	10.8	15.0	17.6	26.7	4.9	Flexure	0.32	3490.5	884.7	0.5	144.0	3.1	-
#F.7-6 (Y-Dir)-1F-6#7	8.3	4.0	0.09	32	16.3	10.5	170.6	1.1	0.0	23.5	16.5	63.2	71.4	13.0	Flexure	0.79	3490.5	1,194.7	0.5	144.0	4.2	-
#E-5 (Y-Dir)-1F-6#6	10.3	4.0	0.09	47	19.0	14.3	272.3	1.3	0.0	20.7	14.5	38.4	47.4	8.6	Flexure	0.60	3490.5	611.7	0.5	144.0	2.1	-
#F-5 (Y-Dir)-1F-4#5	10.3	4.0	0.09	48	19.0	15.0	285.0	1.5	0.0	10.8	7.6	26.5	42.2	7.7	Shear	1.01	3490.5	174.8	0.5	144.0	0.6	OK

Notes:

1 - Effective depth, d, is computed as 0.8h, where h is the dimension of the column in the direction of shear.

2 - Based on 1.1DL+0.275LL.

3 - Per 10.3.4/ASCE 41-17, lap-spliced reinforcement in tied columns is assumed to be ineffective in regions of high ductility demand.

4 - Shear capacity of column is based on ASCE 41-17 Eq 10-3 using nominal material strengths with $\phi = 1.0$.

$$V_{col} = k_{nl}\left[\alpha_{col}\left(\frac{A_v f_{ytLE} d}{s}\right) + \lambda\left[\sqrt{1 + \frac{N_{UG}}{6 Ag \sqrt{f_{cLE}}}} \cdot \frac{6(f_{cLE})}{\left(\frac{M_{UD}}{V_{UD} d}\right)}\right] 0.8 A_g\right]$$

$\alpha_{col} = 1.0$ for $s/d <= 0.75$, 0.0 for $s/d >= 1.0$

5 - Plastic moment capacity of the column is based upon expected flexural strength using 1.5 f'c and 1.25 fy.

6 - Shear induced due to drift of a fixed-fixed column.

$$V_{prob} = \frac{12E \cdot 0.5 \cdot I_g \cdot \Delta}{L^3}$$

7- Vprob is compared to Vcol. If Vprob <Vcol, Shear failure is not likely to occur.