MAFFEI STRUCTURAL ENGINEERING

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

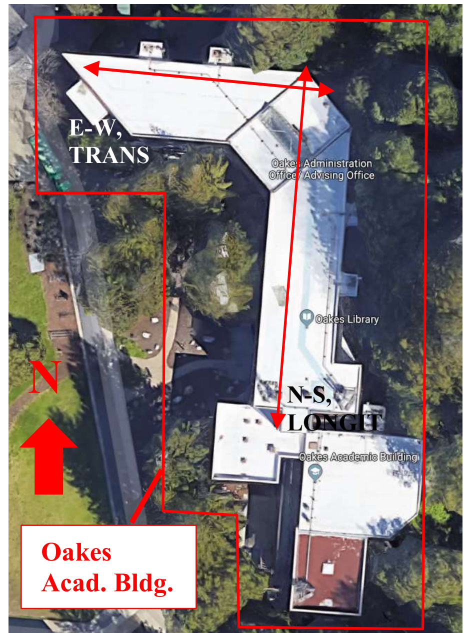
UC Santa Cruz building seismic ratings

Oakes Academic Building

CAAN #7416
233 Oakes Road
UCSC Campus: Main Campus

REGISTERED PROFESSIONAL ENGINEER JOSEPH RICHARD MAFFEI No. 3694 STRUCTURAL STATE OF CALIFORNIA

DATE: 2019-06-30



Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	V (Poor)	
Rating basis	Tier 1	ASCE 41-17[1]
Date of rating	2019	
Recommended UC Santa Cruz priority 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)	
Is 2018-2019 rating required by UCOP?	Yes	
Further evaluation recommended?	Yes	Tier 3 evaluation of steel moment frames

[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 life-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

- Structural drawings by Forell Elsesser, “College 7, University of California Santa Cruz” as-built date 15 July 1974.
- Architectural drawings by McCue Boone Tomsick Architects, “College 7, University of California Santa Cruz” as-built date 15 July 1974.
- Structural drawings by J.D. Raggett & Associates, “Oakes College Humanities Graduate Student Annex” as-built date 18 May 1992.
- Seismic Survey document by Rutherford & Chekene date 2 March 1998 (Information of construction type of 1984 addition from this document
- University of California Facilities Link building database information, “7416” provided by José Sanchez (UCSC) on 2019-05-30.

Additional building information known to exist

- Wood Academic Building addition (west side of lecture hall) built in 1984

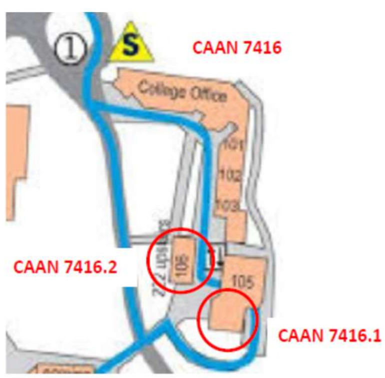
Scope for completing this form

We reviewed the structural drawings for the original construction and carried out a site visit to verify that the existing drawings matched the existing structure to the best of our knowledge. An ASCE 41-17 Tier 1 evaluation was completed. We did not perform an ASCE 41 Tier 1 nonstructural evaluation, but we looked for potentially hazardous nonstructural components during our site visit.

Brief description of structure

The Oakes Academic Building is at the Oakes College (formerly College 7) at the UCSC campus. The building was designed in 1974 by the architectural office of McCue, Boone and Tomsick and the structural office of Forell Elsesser. The building consists of six buildings separated by expansion joints. As shown in the layout plan below, for the purpose of this seismic evaluation, the buildings will be sub-divided into three CAANs as follows:

CAAN	Building Name
7416	Oakes Academic (consisting of north office wing, south office wing, entry hall, and lecture hall)
7416.1	Oakes Academic Lecture Hall addition
7416.2	Oakes Academic Graduate Student Annex



The Oakes Academic building is three stories tall, L-shaped in plan and composed of two office wings linked by an entry hall. The office wings are trapezoidal in plan while the entry hall is rectangular. The building is approximately 24,000 square feet with a regular column grid at 10’ on center. The roof is flat and measures 32 feet in height from the 1st floor level. The exterior walls are non-bearing and consist of 2x wood studs and the interior walls are non-bearing, consisting of metal stud framing.

The Oakes Academic building also includes a one-story lecture hall that is square in plan with a chamfered corner, located at the southeast corner of the office wing. It is roughly 2000s square feet and is 43’x43’ in plan with columns spaced at least 10'-6" apart. The roof is flat and aligns in elevation with the main building roof. It is composed of a wood space frame assembly that spans over the hall and bears on perimeter W14 columns.

An addition to the lecture hall was constructed at the southwest corner of the office wing in 1984 (structural drawings were not available at the time this report was written). The building is conventionally framed with wood framed floors and bearing walls.

Another addition called the Graduate Student Annex was constructed south of the lecture hall in 1992. The building was designed in 1992 by the architectural office of Russ Haisley AIA and the structural office of J.D. Raggett & Associates. It is two stories tall and measures 34'-9"x31'-2" in plan with roughly 2100 square feet of occupiable space. The foundation consists of concrete grade beams. A slab-on-grade spans the southern half of the first floor while the northern half is framed with 5/8" plywood sheathing over 2x14 joists. The roof framing consists of ½" plywood sheathing over 2x10 roof joists. The floor framing consists of 5/8" plywood sheathing over 2x14 wood joists. The typical joists span to 2x wood stud walls or glulam beams. The glulam beams span over large interior spaces and are supported by 6x6 wood posts. A bridge connects the 1992 addition to the lecture hall and office wing.

Both the 1984 addition and the 1992 Graduate Student Annex were constructed after the 1976 UBC which qualifies them as benchmark buildings per ASCE 41, section 3.3. Thus, a full Tier 1 check is not required for these buildings and Form 1 will be filed for each.

Identification of levels

North and south office wings, entry hall: Three levels. 1st Floor at grade with top of slab on grade elevation 609.0, 2nd Floor, 3rd Floor, Roof.

Lecture Hall: One level. 1st Floor 1st Floor at grade with top of slab on grade elevation 609.0 and stepping down to 604.0, Roof.

Foundation system: The foundation consists of concrete grade beams in a grid layout with steel WF columns at the footing intersections. A slab-on-grade spans the entire first floor area. At the lecture hall, the slab on grade steps toward the southeast to make up the benched seating.

Structural system for vertical (gravity) load: The roof framing of the main building consists of 3-1/4" concrete fill over 3" metal deck supported by W16 beams. The beams are supported by W10 columns. The 3rd Floor and 2nd Floor are very similar in construction to the roof, with 3-1/4" concrete fill over 3" metal deck supported by W18 beams. The beams frame into the same W10 columns along the perimeter of the building. The stairs consist of steel stringers topped with concrete over metal deck. There are two outboard mechanical shafts at each office wing that are supported by 4x headers bearing on wood posts.

The lecture hall roof consists of ½" plywood supported by 2x8 joists at 16" on center. The joists are supported by a wood framed space truss system spanning over the hall. The trusses are composed of double 3x12 chords with double 3x8 diagonals, joined together by a steel pipe and shear tab assembly.

Structural system for lateral forces: At the main building, concrete over metal decking at the roof and floor acts as a rigid diaphragm to transfer lateral inertial forces into steel moment frames. The frame columns are oriented in the strong direction to resist transverse lateral loads through single bay frames at 10ft on center. The columns are oriented in the weak direction for resisting longitudinal lateral loads through multiple bays at the building perimeter.

The lecture hall consists of two-bay moment frames, one story tall, at each perimeter wall. The typical moment frames are anchored into the foundation system with 1" diameter anchor bolts and 1" bearing plates. A 2" expansion joint is located between the main building portions and lecture hall.

The two outboard mechanical wells are sheathed on the short transverse walls, and a single longitudinal wall. However, the longitudinal wall has a full wall width penetration to accommodate a louver system. At the floor levels, the joists are framed with ½" plywood acting as a diaphragm to transfer forces into the perimeter W18 beam or W16 roof beam. The transverse edge beam is a wood 4x8 on each side of the vent that is bolted to the steel perimeter beam.

The stair at the southeast corner of the office wing adjacent to the lecture hall has two steel braced frames oriented to resist lateral forces in the north-south direction. The frames consist of 2-1/2” std pipe columns and braces, and W8 beams.

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

- The steel moment frames exceed the Tier 1 checklist limit for drift which will negatively impact the non-structural components and finishes.
- Moment frame beam-column connections are designed as full penetration welds which was typical for pre-Northridge connections. The column panel zones do not have adequate shear capacity to develop the strength of the beams framing in. We expect a brittle failure mechanism under a major seismic event at beam-column connections.
- Strong column-weak beam requirements are not satisfied. We would expect yielding mechanisms to occur in the columns rather than the beams.
- Because of the expected drift, the existing 2” seismic gaps between buildings are not adequate. We expect pounding and significant damage between buildings after a major seismic event.

At southeast stair braced frame:

- The 2-1/2” pipe columns do not have adequate strength to resist overturning forces. The surrounding perimeter walls may act as bearing supports to prevent collapse in this area.
- The connection brace to gusset connection strength is not adequate to develop the strength of the brace. We expect a brittle failure mechanism under a major seismic event at brace-gusset connections.

Structural deficiency	Affects rating?	Structural deficiency	Affects rating?
Lateral system stress check (wall shear, column shear or flexure, or brace axial as applicable)	Y	Openings at shear walls (concrete or masonry)	N
Load path	Y	Liquefaction	N
Adjacent buildings	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	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 non-structural life-safety concerns, including at exit routes.[3]

- Above the main entrance doors there are large pendant lights. It is unknown what their connection to the ceilings are and may present a falling hazard in a seismic event.
- Sloping glass over the entry, may be susceptible to breaking because of excessive building movement. Overhead panels appear to be wired glass which may limit the safety concerns because of this falling hazard.
- Within the main entrance building study hall there were large multi-story, single pane windows. These large windows did not appear to be detailed to accommodate lateral drift or have a protective film in case of cracking and may present a falling hazard in a seismic event. Panels are attached to slender secondary framing elements which are in turn attached to the main frame, which may not provide adequate stiffness to protect the glazing.

[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 non-structural hazards may occur.

- The stair connections do not have any deformation capability because of their rigid connection to the structure at the top and bottom. As a result, these stairs may act as bracing elements and attract seismic loads for which they are not adequate. We expect that the stair and connections to structure will experience damage/non-linear behavior in a seismic event because of this.
- The outboard mechanical vents do not have a well-defined load path, either to the structure or to the foundation. While we expect a significant amount of movement and subsequent damage in these areas, we do not expect these framing elements to pose a collapse hazard.

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	Potential Hazards 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

Discussion of rating

The rating of V takes into account that the lateral force resisting system of the building relies on pre-Northridge steel moment frame connections which have demonstrated brittle, non-ductile behavior under a major seismic events. While the building itself has a well-defined load path and lateral elements, the large expected lateral drift and brittleness of the non-structural elements (e.g. exterior glazing, stair connections) are a potential hazard. It is unclear, without more in-depth structural analysis, if this building has a collapse mechanism when subjected to code level seismic forces.

Recommendations for further evaluation or retrofit

We recommend that the Campus perform a more detailed review of the steel moment frame system with a focus on the beam-to-column joints. While we observed that the previous earthquakes have not resulted in observable damage, our experience suggests that there may be damage to the existing moment frame joints that may not be readily observed. We recommend the joints be inspected to verify their integrity and capacity to resist future ground shaking.

We also recommend performing a Tier 3 evaluation to obtain a more refined quantification of frame demands and interstory drifts. We recommend that nonlinear analyses be performed to better quantify flexural demands and capacities at the beam-column connections. Nonlinear analyses will also contribute to better understand the magnitude of the problem with the flexural strength of the frame columns. Only by performing nonlinear analysis can one get better confidence in the absence of a potential story mechanism.

We also recommend a more detailed evaluation of the braced frame at the southeast stair framing and its ability to resist lateral forces in tandem with the moment frame system. We put the building on Priority Category B, as the above items should be done if there are any plans for modifying the building.

Peer review of rating

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

Additional building data	Entry	Notes
Latitude	36.989977	
Longitude	-122.062792	
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	0	
Building occupiable area (OGSF)	30,204	
Risk Category per 2016 CBC Table 1604.5	II	Offices at Level 2 & Dining Hall at Level 1
Building structural height, h_n	38 ft	Structural height defined per ASCE 7-16 Section 11.2
Coefficient for period, C_t	0.035	Estimated using ASCE 41-17 equation 4-4 and 7-18
Coefficient for period, β	0.8	Estimated using ASCE 41-17 equation 4-4 and 7-18
Estimated fundamental period	0.64 sec	Estimated using ASCE 41-17 equation 4-4 and 7-18
Site data		
975 yr hazard parameters S_s, S_1	1.278, 0.483	
Site class	D	
Site class basis[4]	Geotech	See footnote below
Site parameters F_a, F_v[5]	1, 1.817	
Ground motion parameters S_{cs}, S_{c1}	1.278, 0.878	
S_a at building period	1.37	
Site V_{s30}	1500 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	Orig. Built: 1974 Code: 1970 UBC	Code on drawings

[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

[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.

Applicable code for partial retrofit	None	No partial retrofit
Applicable code for full retrofit	None	No full retrofit
Model building data		
Model building type North-South	S1 S1a W2	Original office wings and entrance Original lecture hall
Model building type East-West	S1 S1a	Original office wings and entrance Original lecture hall
FEMA P-154 score	N/A	Not included here because we performed ASCE 41 Tier 1 evaluation.
Previous ratings		
Most recent rating	none	
Date of most recent rating	-	
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

Annotated Foundation Partial Plan - South:

[redacted]

Annotated Foundation Partial Plan - North:

[redacted]

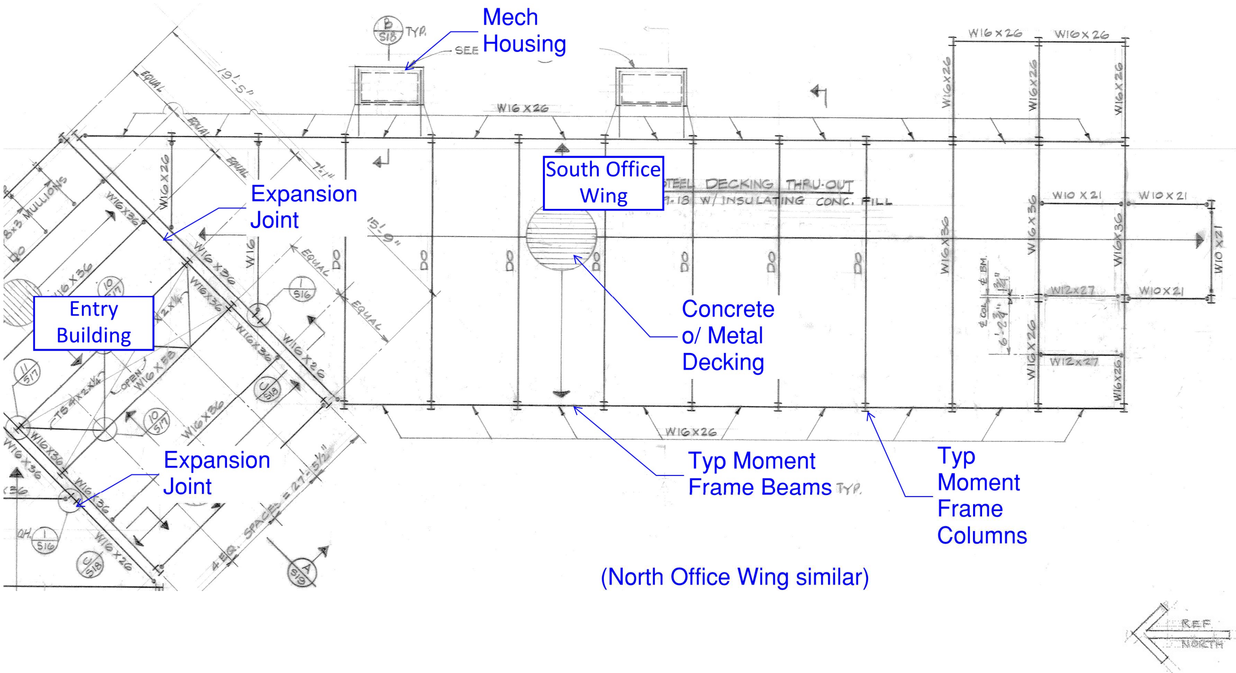
Annotated Level 2 Partial Plan:

[redacted]

Annotated Third Floor & Lecture Hall Roof Plan:

[redacted]

Annotated Roof Plan:



Windows at study hall

Lecture hall roof framing

Table 17-2. Collapse Prevention Basic Configuration Checklist

Status **Evaluation Statement**

Low Seismicity

Building System—General

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.

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.

2” expansion joint < .015 x 38ft = 7in

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.

No Mezzanine

Building System—Building Configuration

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.

Even distribution of moment frames at each level.

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.

C NC N/A U VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.

C 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.

C 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.

C NC N/A U TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.

Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)

Geologic Site Hazards

C 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 (15.2 m) under the building.

C NC N/A U SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.

C NC N/A U SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.

High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)

Foundation Configuration

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 $0.6S_a$.

18ft/33ft = 0.58 < 0.6*1.28g

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.

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-8. Collapse Prevention Structural Checklist for Building Types S1 and S1a

Status	Evaluation Statement	
Low Seismicity		
Seismic-Force-Resisting System		
C NC N/A U	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2.	
C NC N/A U	DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.4.3.1, is less than 0.030.	Per calculations
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than $0.30F_y$.	
C NC N/A U	FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.4.3.9, is less than F_y. Columns need not be checked if the strong column–weak beam checklist item is compliant.	
Connections		
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames.	
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation.	A.B. w/ PL washer
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)		
Seismic-Force-Resisting System		
C NC N/A U	REDUNDANCY: The number of bays of moment frames in each line is greater than or equal to 2.	
C NC N/A U	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements.	
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections can develop the strength of the adjoining members based on the specified minimum yield stress of steel.	FP bevel weld provided.
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	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel in accordance with AISC 341, Section A3.2.	pre-Northridge Connection
C NC N/A U	PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column.	
C NC N/A U	COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web.	Typical splice detail not provided
C NC N/A U	STRONG COLUMN—WEAK BEAM: The percentage of strong column–weak beam joints in each story of each line of moment frames is greater than 50%.	
C NC N/A U	COMPACT MEMBERS: All frame elements meet section requirements in accordance with AISC 341, Table D1.1, for moderately ductile members.	
Diaphragms (Stiff or Flexible)		
C NC N/A U	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length.	N/A at class laboratory since width of diaphragm same as frame
Flexible Diaphragms		
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	Diaphragm at Lecture Hall Welded pipe and gussets at truss intersections w/ ½" plywood roof sheathing.
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	

continues

Table 17-8 *(Continued)*. Collapse Prevention Structural Checklist for Building Types S1 and S1a

Status	Evaluation Statement
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-10. Collapse Prevention Structural Checklist for Building Types S2 and S2a

Exit stair adjacent to Lecture Hall

Status	Evaluation Statement
Low Seismicity	
Seismic-Force-Resisting System	
C NC N/A U	REDUNDANCY: The number of lines of braced frames in each principal direction is greater than or equal to 2.
C NC N/A U	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than $0.30F_y$.
C NC N/A U	BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.4.3.4, is less than $0.50F_y$.
Connections	
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames.
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation.
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)	
Seismic-Force-Resisting System	
C NC N/A U	REDUNDANCY: The number of braced bays in each line is greater than 2.
C NC N/A U	CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals.
C NC N/A U	COMPACT MEMBERS: All brace elements meet compact section requirements in accordance with AISC 360, Table B4.1.
C NC N/A U	K-BRACING: The bracing system does not include K-braced bays.
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	COLUMN SPLICES: All column splice details located in braced frames develop 50% of the tensile strength of the column.
C NC N/A U	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have Kl/r ratios less than 200.
C NC N/A U	CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals.
C NC N/A U	COMPACT MEMBERS: All brace elements meet section requirements in accordance with AISC 341, Table D1.1, for moderately ductile members.
C NC N/A U	CHEVRON BRACING: Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs.
C NC N/A U	CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces frame into the beam–column joints concentrically.
Diaphragms (Stiff or Flexible)	
C NC N/A U	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 25% of the frame length.
Flexible Diaphragms	
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.

¼” Gusset yields

Table 17-6. Collapse Prevention Structural Checklist for Building Type W2

Status	Evaluation Statement
Low and Moderate Seismicity	
Seismic-Force-Resisting System	
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing 1,000 lb/ft Diagonal sheathing 700 lb/ft Straight sheathing 100 lb/ft All other conditions 100 lb/ft
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building.
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an ... through the floor.
C NC N/A U	... one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1.
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.
C NC N/A U	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.
Connections	
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)	
Connections	
C NC N/A U	WOOD SILL BOLTS: Sill bolts are spaced at 6 ft (1.8 m) or less with acceptable edge and end distance provided for wood and concrete.
Diaphragms	
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.

1984 & 1992 wood additions (W2) benchmark buildings per ASCE-41, Section 3.3

Table 17-6 *(Continued)*. Collapse Prevention Structural Checklist for Building Type W2

Status	Evaluation Statement
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally [illegible] have horizontal spans less than 40 ft (12.2 m) and have aspect ratios less than or equal to [illegible]
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

1984 & 1992 wood additions (W2) are benchmark buildings per ASCE-41, Section 3.3

ASCE 41-17 - Tier 1 Calculations

Oakes Academic Building

Building Properties:

Seismic Parameters:

Risk Category: II 2016 CBC table 1604.5

Site Class: D Assumed

Probability: 5% in 50 years

$S_{XS} = 1.533$ for BSE-2E hazard level

$S_{X1} = 0.878$ for BSE-2E hazard level

Seismic Forces:

$T = C_t \cdot h_n^{\beta} \rightarrow 0.64$ sec (ASCE 41-17 Eqn. 4-4)

$C_t = 0.035$ steel moment frame systems

$\beta = 0.8$ steel moment frame systems

$h_n = 38$ ft

$S_a = min\left(\frac{S_{X1}}{T}, S_{XS}\right) \rightarrow 1.37$ g (ASCE 41-17 Eqn. 4-3)

$V_{base} = C_{base} \cdot S_a \cdot W_{total} \rightarrow 1{,}706\,kips$ (ASCE 41-17 Eqn. 4-1)

$C_{base} = 1$ (ASCE41-17 Table 4-7) 3 Story Moment Frame

Building Weight:

$A_{floor} = 4{,}242\,ft^2$

$floor_{unit,weight} = \Sigma\left(tbl_{floor\,Unit_{weight}}\right) \rightarrow 80\,psf$

$floor_{weight} = \Sigma\left(tbl_{floor\,Weight_{floor}}\right) + ExtWall_{weight} + IntWall_{weight} \rightarrow 474\,kips$

$roof_{unit,weight} = \Sigma\left(tbl_{roof\,Unit_{weight}}\right) \rightarrow 60\,psf$

$roof_{weight} = \Sigma\left(tbl_{roof\,Weight_{roof}}\right) + 0.5 \cdot ExtWall_{weight} + 0.5 \cdot IntWall_{weight} \rightarrow 300\,kips$

$A_{roof} = 3{,}880\,ft^2$

$W_{total} = floor_{weight} + floor_{weight} + roof_{weight} \rightarrow 1{,}248\,kips$

$A_{ExtWall} \rightarrow 4{,}680\,ft^2$

$ExtWall_{unit,weight} = \Sigma\left(tblExtWall_{Unit_{weight}}\right) \rightarrow 18\,psf$

$ExtWall_{weight} = \Sigma\left(tblExtWall_{Weight_{ExtWall}}\right) \rightarrow 84.2\,kips$

$A_{IntWall} \rightarrow 4{,}212\,ft^2$

$IntWall_{unit,weight} = \Sigma\left(tblIntWall_{Unit_{weight}}\right) \rightarrow 12\,psf$

$IntWall_{weight} = \Sigma\left(tblIntWall_{Weight_{IntWall}}\right) \rightarrow 50.5\,kips$

ASCE 41-17 - Tier 1 Calculations

Oakes Academic Building

Load Distribution:

tblFloorShear

$Floor$	W	h	M'	C_{story}	V_{story}	V_{total}
Roof	$300\,kips$	$38\,ft$	$11{,}407{,}296\,lbs \cdot ft$	0.38	$651\,kips$	$651\,kips$
Third	$474\,kips$	$26\,ft$	$12{,}327{,}744\,lbs \cdot ft$	0.41	$703\,kips$	$1{,}354\,kips$
Second	$474\,kips$	$13\,ft$	$6{,}163{,}872\,lbs \cdot ft$	0.21	$352\,kips$	$1{,}706\,kips$

$$M' = W \cdot h$$

$$C_{story} = \frac{M'}{\Sigma\left(tblFloorShear_{M'}\right)}$$

$$V_{story} = C_{story} \cdot V_{base}$$

Roof Level - S1 Drift Check: ASCE 41-17 Sec. 4.4.3.1

$$D_r = \left(\frac{k_b + k_c}{k_b k_c}\right) \cdot \left(\frac{h}{12E}\right) \cdot V_c \text{ Interstory drift}$$

$$k_b = \frac{I_{beam,x}}{L} \qquad E = 29{,}000\,ksi$$

$$k_c = \frac{I_{col}}{h} \qquad L = 10\,ft \qquad h = 13\,ft$$

East-West direction - Third Floor

$N_{col} = 24$ Number of columns resisting load

Member Properties

W18x35 Beam	W10x49 Col	
$I_{beam,x} = 510in^4$	$I_{col,x} = 272in^4$	
	$I_{col,y} = 93.4in^4$	
$Z_b = 66.5in^3$	$Z_{cx} = 60.4in^3$	$Z_{cy} = 28.3in^3$
$d_b = 17.7\,in$	$d_c = 10\,in$	
$t_f = 0.425\,in$	$t_{cf} = 0.56\,in$	
	$t_{cw} = 0.34\,in$	
	$b_{cf} = 10\,in$	

$F_y = 36\,ksi$

$R_y = 1.5$

$V_c = \frac{V_3}{N_{col}} \rightarrow 56{,}429\,lbs$ 3rd Floor Load Demand to each column

$k_b \rightarrow 4.25in^3$

$I_{col} = I_{col,y} \rightarrow 93.4in^4$

$k_c \rightarrow 0.6in^3$

$$D_r = \left(\frac{k_b + k_c}{k_b k_c}\right) \cdot \left(\frac{h}{12E}\right) \cdot V_c \text{ Interstory drift}$$

$D_{r,3} = D_r \rightarrow 0.048$ Interstory drift at 3rd Floor

$dcr\left(D_{r,3}, 0.03\right) \rightarrow$ NG (1.61)

Therefore, interstory drift criteria not met

ASCE 41-17 - Tier 1 Calculations

Oakes Academic Building

S1 Axial Stress Check: ASCE 41-17 Sec. 4.4.3.6

Check loading in North-South direction for overturning

$$p_{ot} = \frac{1}{M_s} \cdot \left(\frac{2}{3}\right) \cdot \left(\frac{V \cdot h_n \cdot ft}{L_{frame} n_f}\right) \cdot \left(\frac{1}{A_{col}}\right) \qquad \text{Axial Demand}$$

Tension Demand

$$p_{ot} = \frac{1}{M_s} \cdot \left(\frac{2}{3}\right) \cdot \left(\frac{V \cdot h_n \cdot ft}{L_{frame} n_f}\right) \cdot \left(\frac{1}{A_{col}}\right) \qquad p_{ot} \rightarrow 4{,}168\, psi \quad \text{Axial Demand}$$

$V = V_2 \rightarrow 1{,}706{,}008\, lbs$ Total shear at 2nd Floor

$L_{frame} = 32\, ft$

$M_s = 2.5$ Buildings evaluated to Collapse Prevention Performance Level

$n_f = 9$

$A_{col} = 14.4 in^2$

$h_n \rightarrow 38$ height above base to roof level

$\sigma_{axial,allow} = 0.3 \cdot F_y \rightarrow 10.8\, ksi$

$dcr\left(p_{ot}, \sigma_{axial,allow}\right) \rightarrow$ OK (0.39)

Check loading in East-West direction for max overturning

$$p_{ot} = \frac{1}{M_s} \cdot \left(\frac{2}{3}\right) \cdot \left(\frac{V \cdot h_n \cdot ft}{L_{frame} n_f}\right) \cdot \left(\frac{1}{A_{col}}\right) \qquad p_{ot} \rightarrow 6{,}670\, psi \quad \text{Axial Demand}$$

$V = V_2 \rightarrow 1{,}706{,}008\, lbs$ Total shear at 2nd Floor

$L_{frame} = 90\, ft$

$M_s = 2.5$ Buildings evaluated to Collapse Prevention Performance Level

$n_f = 2$

$A_{col} = 14.4 in^2$

$h_n \rightarrow 38$ height above base to roof level

$\sigma_{axial,allow} = 0.3 \cdot F_y \rightarrow 10.8\, ksi$

$dcr\left(p_{ot}, \sigma_{axial,allow}\right) \rightarrow$ OK (0.62)

ASCE 41-17 - Tier 1 Calculations

Oakes Academic Building

Flexural Stress in Columns and Beams

East-West Loading Direction

$$f_{j,avg,beams} = V_j \cdot \left(\frac{1}{M_s}\right) \cdot \left(\frac{n_c}{n_c - n_f}\right) \cdot \left(\frac{h}{2}\right) \cdot \left(\frac{1}{Z_{bms}}\right)$$

$V_j = V_2 \rightarrow 1{,}706{,}008\ lbs$ Total shear at 2nd Floor

$M_s = 9$ Buildings evaluated to Collapse Prevention Performance Level

$n_c = 21$ Number of Frame Columns

$n_b = 18$ Number of Frame Beams at 2nd Floor

$n_f = 2$ Number lines of resistance

$h \rightarrow 13\ ft$ story height

$Z_{bms} = 2 \cdot n_b \cdot Z_b \rightarrow 2{,}394\ in^3$

$f_{j,avg,beams} \rightarrow 6.83\ ksi$ Average Beam Flexural Stress Demand

$dcr\left(f_{j,avg,beams}, F_y\right) \rightarrow \text{OK (0.19)}$

$$f_{j,avg,cols} = V_j \cdot \left(\frac{1}{M_s}\right) \cdot \left(\frac{n_c}{n_c - n_f}\right) \cdot \left(\frac{h}{2}\right) \cdot \left(\frac{1}{Z_{cols}}\right)$$

$Z_{cols} = n_c \cdot Z_{cy} \rightarrow 594\ in^3$

$f_{j,avg,cols} \rightarrow 27.5\ ksi$ Average Column Flexural Stress Demand

$dcr\left(f_{j,avg,cols}, F_y\right) \rightarrow \text{OK (0.76)}$

North-South Loading Direction

$$f_{j,avg,beams} = V_j \cdot \left(\frac{1}{M_s}\right) \cdot \left(\frac{n_c}{n_c - n_f}\right) \cdot \left(\frac{h}{2}\right) \cdot \left(\frac{1}{Z_{bms}}\right)$$

$V_j = V_2 \rightarrow 1{,}706{,}008\ lbs$ Total shear at 2nd Floor

$M_s = 9$ Buildings evaluated to Collapse Prevention Performance Level

$n_c = 18$ Number of Frame Columns

$n_b = 9$ Number of Frame Beams at 2nd Floor

$n_f = 9$ Number lines of resistance

$h \rightarrow 13\ ft$ story height

$Z_{bms} = 2 \cdot n_b \cdot Z_b \rightarrow 1{,}197\ in^3$

$f_{j,avg,beams} \rightarrow 24.7\ ksi$ Average Beam Flexural Stress Demand

$dcr\left(f_{j,avg,beams}, F_y\right) \rightarrow \text{OK (0.69)}$

$$f_{j,avg,cols} = V_j \cdot \left(\frac{1}{M_s}\right) \cdot \left(\frac{n_c}{n_c - n_f}\right) \cdot \left(\frac{h}{2}\right) \cdot \left(\frac{1}{Z_{cols}}\right)$$

$Z_{cols} = n_c \cdot Z_{cx} \rightarrow 1{,}087\ in^3$

$f_{j,avg,cols} \rightarrow 27.2\ ksi$ Average Column Flexural Stress Demand

$dcr\left(f_{j,avg,cols}, F_y\right) \rightarrow \text{OK (0.76)}$

ASCE 41-17 - Tier 1 Calculations

Oakes Academic Building

$$\lambda_{md} = \frac{b}{t}$$

$$bt_{ratio} = 8.93$$

$$ht_{w,ratio} = 23.1$$

$$dcr\left(bt_{ratio}\ ,\ 0.38 \cdot \sqrt{\frac{E}{F_y}}\right) \rightarrow \text{OK (0.83)}\ \ \text{Checking Limit for Moderately Ductile Members}$$

$$dcr\left(ht_{w,ratio}\ ,\ 1.49 \cdot \sqrt{\frac{E}{F_y}}\right) \rightarrow \text{OK (0.55)}\ \ \text{Checking Limit for Moderately Ductile Members}$$

<u>Check Panel Zone Strength</u>

$$M_{pr} = 1.1 \cdot R_y \cdot Z_b \cdot F_y \rightarrow 3{,}950\, kip \cdot in\ \ \text{Plastic Moment demand}$$

$$R_u = \frac{0.8 \cdot 2 \cdot M_{pr}}{d_b - t_f} \rightarrow 366\, kips$$

Find Panel Zone capacity. From AISC Specifications equation J10-11 for W10x49 column:

$$\phi R_n = 0.6 \cdot F_y \cdot d_c \cdot t_{cw} \cdot \left(1 + \frac{3 \cdot b_{cf} \cdot t_{cf}^{\ 2}}{d_b d_c t_{cw}}\right) \rightarrow 84.9\, kips$$

$$dcr\left(R_u\ ,\ \phi R_n\right) \rightarrow \text{NG (4.31)}$$

Therefore, existing W10x49 column panel zone is not adequate.

<u>Strong Column-Weak Beam Check</u>

$Z_cF_{yc} > 1.1\ R_yZ_bF_{yb}$ n.g. by inspection

W10x49 Column W18x35 Beam

$Z_{cx} \rightarrow 60.4 in^3$ $Z_b \rightarrow 66.5 in^3$

ASCE 41-17 - Tier 1 Calculations

Oakes Academic Building - Stair near Lecture Hall

Seismic Forces:

$T = C_t \cdot h_n^{\beta} \rightarrow 0.23\ \text{sec}$ (ASCE 41-17 Eqn. 4-4)

$C_t = 0.02$ for all other framing systems

$\beta = 0.75$ for all other framing systems

$h_n = 26\ \text{ft}$

$S_a = min\left(\frac{S_{X1}}{T}, S_{XS}\right) \rightarrow 1.53\ \text{g}$ (ASCE 41-17 Eqn. 4-3)

$C_{base} = 1.2$ (ASCE41-17 Table 4-7) 2 Story Braced Frame

Building Weight:

$A_{roof} = 126 ft^2$

$A_{floor} = 244 ft^2$

$A_{ExtWall} = 17\,ft \cdot 13\,ft \rightarrow 221 ft^2$

$roof_{unit,weight} = \Sigma\left(tbl_{roof\,Unit_{weight}}\right) \rightarrow 60\,psf$

$floor_{unit,weight} = \Sigma\left(tbl_{floor\,Unit_{weight}}\right) \rightarrow 80\,psf$

$ExtWall_{unit,weight} = \Sigma\left(tblExtWall_{Unit_{weight}}\right) \rightarrow 18\,psf$

$roof_{weight} = A_{roof} \cdot roof_{unit,weight} + 0.5 \cdot ExtWall_{unit,weight} \cdot A_{ExtWall} \rightarrow 9.55\,kips$

$floor_{weight} = A_{floor} \cdot floor_{unit,weight} + ExtWall_{unit,weight} \cdot A_{ExtWall} \rightarrow 23.5\,kips$

$W_{total} = floor_{weight} + roof_{weight} \rightarrow 33\,kips$

$V_{base} = C_{base} \cdot S_a \cdot W_{total} \rightarrow 60.8\,kips$ (ASCE 41-17 Eqn. 4-1)

Load Distribution:

tblBrFrmFloorShear

Floor	*W*	*h*	*M'*	C_{story}	V_{story}	V_{total}
Third	9.55 *kips*	26 *ft*	248,274 *lbs·ft*	0.45	27.3 *kips*	27.3 *kips*
Second	23.5 *kips*	13 *ft*	305,474 *lbs·ft*	0.55	33.5 *kips*	60.8 *kips*

$M' = W \cdot h$

$C_{story} = \frac{M'}{\Sigma\left(tblBrFrmFloorShear_{M'}\right)}$

$V_{story} = C_{story} \cdot V_{base}$

ASCE 41-17 - Tier 1 Calculations

Oakes Academic Building - Stair near Lecture Hall

S2 Column Axial Stress Check: ASCE 41-17 Sec. 4.4.3.6

Check loading in East-West direction for overturning

Tension Demand

$$p_{ot} = \frac{1}{M_s} \cdot \left(\frac{2}{3}\right) \cdot \left(\frac{V \cdot h_n \cdot ft}{L_{frame} n_f}\right) \cdot \left(\frac{1}{A_{col}}\right) \qquad p_{ot} \rightarrow 30{,}993\,psi \text{ Axial Demand}$$

$V = querycol(tblBrFrmFloorShear, Floor, Second, V_{total}) \rightarrow 60{,}793\,lbs$ Total shear at 2nd Floor

$L_{frame} = 4\,ft$

$M_s = 2.5$ Buildings evaluated to Collapse Prevention Performance Level

$n_f = 2$

$A_{col} = 1.7\,in^2$ 2.5" STD Pipe

$h_n \rightarrow 26$ height above base to roof level

$F_y = 36\,ksi$

$\sigma_{axial,allow} = 0.3 \cdot F_y \rightarrow 10.8\,ksi$

$dcr(p_{ot}, \sigma_{axial,allow}) \rightarrow$ NG (2.87)

S2 Brace Axial Stress Check: ASCE 41-17 Sec. 4.4.3.4

Axial demand in brace

$$f_{j,avg,brace} = \left(\frac{1}{M_{s,br}}\right) \cdot \boxed{\left(\frac{V_j}{s \cdot N_{br}}\right) \cdot \left(\frac{L_{br}}{A_{br}}\right)} \rightarrow 1.74\,ksi$$

$dt_{ratio} = 14.2$

$F_{ye} = 1.25 \cdot F_y \rightarrow 45\,ksi$

$\frac{1{,}500\,ksi}{F_{ye}} \rightarrow 33.3$

$M_{s,br} = 70$ For d/t < 1500/F_ye

$V_j = V \rightarrow 60{,}793\,lbs$

$s = 4\,ft$ Horiz span

$N_{br} = 1$

$A_{br} = A_{col} \rightarrow 1.7\,in^2$

$L_{br} = 13.6\,ft$

$\sigma_{brace,allow} = 0.5 \cdot F_y \rightarrow 18\,ksi$

$dcr(f_{j,avg,brace}, \sigma_{brace,allow}) \rightarrow$ OK (0.096)

ASCE 41-17 - Tier 1 Calculations

Oakes Academic Building - Stair near Lecture Hall

S2 Compact Members per AISC 360, Table B4.1

$$\lambda_{r9} = \frac{0.11 \cdot E}{F_y} \rightarrow 88.6$$

$dcr\left(dt_{ratio}, \lambda_{r9}\right) \rightarrow$ OK (0.16) Columns and Braces

$$\lambda_{r1} = 0.56 \cdot \sqrt{\frac{E}{F_y}} \rightarrow 15.9 \qquad \lambda_{r5} = 1.49 \cdot \sqrt{\frac{E}{F_y}} \rightarrow 42.3$$

$bt_{ratio} = 7.95$

$ht_{wRatio} = 29.9$

$dcr\left(bt_{ratio}, \lambda_{r1}\right) \rightarrow$ OK (0.5) Columns and Braces

$dcr\left(ht_{wRatio}, \lambda_{r5}\right) \rightarrow$ OK (0.71) Columns and Braces

S2 Slenderness Check

$k = 1$

$$\frac{k \cdot L_{br}}{r} \rightarrow 172 \qquad dcr\left(\frac{k \cdot L_{br}}{r}, 200\right) \rightarrow \text{OK (0.86)}$$

$r = 0.947\ in$

S2 Connection Strength

$\phi R_{n,tension} = \phi \cdot F_y \cdot t_{pl} \cdot w_p \rightarrow 39.5\ kips$ Gr 36 Gusset capacity

$\phi = 0.9$ For Yield

$t_{pl} = 0.25\ in$

$w_p = 2.88\ in + 1\ in + 1\ in \rightarrow 4.88\ in$ Whitmore width as width of brace plus 1" ea side

$P_{U,tension} = R_y \cdot F_{y,br} \cdot A_{br} \rightarrow 97.9\ kips$ Expected Brace Strength in Tension

$R_y = 1.6$

$F_{y,br} = F_y \rightarrow 36\ ksi$

$dcr\left(P_{U,tension}, \phi R_{n,tension}\right) \rightarrow$ NG (2.48)

S2 Compact Members per AISC 341, Table D1.1 Moderately Ductile

$\lambda_{md,br} = \frac{0.044 \cdot E}{F_y} \rightarrow 35.4$ $dcr\left(dt_{ratio}, \lambda_{md,br}\right) \rightarrow$ OK (0.4) Columns and Braces

$\lambda_{md,bm,fl} = 0.38 \cdot \sqrt{\frac{E}{F_y}} \rightarrow 10.8$ $bt_{ratio} \rightarrow 7.95$ $dcr\left(bt_{ratio}, \lambda_{md,bm,fl}\right) \rightarrow$ OK (0.74) Columns and Braces

$\lambda_{md,wb} = 1.49 \cdot \sqrt{\frac{E}{F_y}} \rightarrow 42.3$ $ht_{wRatio} \rightarrow 29.9$ $dcr\left(ht_{wRatio}, \lambda_{md,wb}\right) \rightarrow$ OK (0.71) Columns and Braces