UCSF

Rating form completed by:

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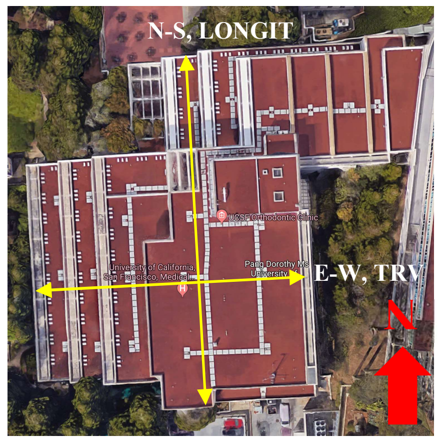
Text in green is to be part of UCSF building database and may be part of UCOP database

UCSF building seismic ratings

School of Dentistry, University of California San Francisco

CAAN #2412
707 Parnassus Ave, San Francisco, CA 94131
UCSF Campus: Parnassus

DATE: 2020-06-26



Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	V	Pending planned Tier 2 analysis
Rating basis	Tier 1	ASCE 41-17[1]
Date of rating	2019	
Recommended UCSF priority category for retrofit	Priority B	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application for modification
Ballpark total construction cost to retrofit to IV rating[2]	Very High (> $400/sf)	See recommendations on further evaluation and retrofit.
Is 2018-2019 rating required by UCOP?	Yes	Building previously rated IV but does not have a fully documented previous review
Further evaluation recommended?	Tier 3 NLRHA	Start with Tier 2 linear, proceed to Tier 3 if beneficial.

[1] We translate this Tier 1 evaluation to a Seismic Performance Level rating using professional judgment. Non-compliant items in the Tier 1 evaluation do not automatically put a building into a particular rating category, but we evaluate such items along with the combination of building features and potential deficiencies, focused on the potential for collapse or serious damage to the gravity supporting structure that may threaten occupant safety. See Section III B of the UC Seismic Policy and Method B of Section 321 of the 2016 California Existing Building Code.

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

Tier 3 linear evaluation

Aspects of this report will be superseded by the Tier 3 linear evaluation in progress by MSE. The Seismic Performance Level Rating remains V. The Tier 3 findings show that the building can achieve a rating of IV from the retrofitting of selected columns. Thus, the cost to retrofit to a rating of IV will be lower than indicated in this report. The Tier 3 findings will provide revised information on the significance of the potential deficiencies.

Building information used in this evaluation

- Structural drawings by Isador Thompson and Assoc., "School of Dentistry Building, University of California San Francisco Campus," dated June 15, 1976 (59 sheets).
- Architectural Drawings by John Funk and Assoc., "School of Dentistry Building, University of California San Francisco Campus," dated June 15, 1976 (90 sheets).
- UCSF Group 2 Buildings – Assessment of Geotechnical Characteristics and Geohazards, 5/17/19 (draft) by SRC member, John Egan.

Additional building information known to exist

- Specifications by John Funk and Assoc., "School of Dentistry Building, University of California San Francisco Campus," dated June 1976

Scope for completing this form

Reviewed structural drawings for original construction and carried out ASCE 41-17 Tier 1 evaluation. Conducted limited review of architectural drawings to inform load takeoff. Made brief site visit to visually observe for conformance with drawings, building condition, and review by spot check for nonstructural seismic hazards.

Brief description of structure

The building, constructed on a sloping hillside site, has an area of approximately 135,000 square feet. It was designed in the mid-1970s by architect John Funk and Associates and structural engineer Isador Thompson with drawings dated 1976. Construction was completed in 1979.

The building has four stories above grade plus a relatively small penthouse. A basement mechanical level underlies a portion of ground level, and a crawl space underlies a portion of the remainder. The building is terraced into the hillside, in particular at the North Wing, as can be seen in Figure 3. Upper floor levels step back to follow the hillside as can be seen in Figures 1, 2 and 3 and the aerial photos.

Identification of levels: A partial basement, used as a mechanical room, is designated the Basement level (El +353') on original drawings. The Basement level daylights at west side, but with a full-length concrete wall that is free of openings. A crawl space occurs below an additional portion of Level 1 at a higher elevation than Basement (Elevation varies). This area has a 2" mud slab floor on ground and presumably allows for plumbing and other utility distribution services. There are four main floor levels above the basement: referred to as Level 1 (El +367), 2 (El +382), 3 (El +396), and 4 (El +410); and a flat Roof level (El +410). Each level has a somewhat different plan shape due to the terracing of the building.

Foundation system: The building is terraced into the hillside, with retaining walls on the east side typically retaining 1 level of soil. At the north end of the building (North Wing) portions of Levels 2, 3, 4 and 5 are constructed on grade to accommodate slope at the north end of the building. Portions of Level 1 are constructed on grade at the South Wing. The entirety of the building is supported on cast-in-place concrete piers that are typically 4 feet in diameter. The piers are connected by a substantial system of orthogonal and diagonal grade beams.

Structural system for vertical (gravity) load: The elevated floors and roof are framed using a one-way, lightweight concrete joisted slab consisting of a 5-1/2 inch thick slab and 10" wide by 16-1/2" (net) deep ribs at 6'-8" spacing. Joists span in the north-south direction to 6 foot wide girders of same depth (22" overall), spaced at 30 foot centers. Girders span to reinforced concrete columns, which are typically 18 inches square or smaller, and constructed with 5,000 psi normal weight concrete. Reinforced concrete walls serve to support floors for gravity loads, where they occur.

Structural system for lateral forces: Lateral seismic forces are typically resisted by reinforced concrete walls. Referring to Figure 1, walls shown solid extend full height from the roof to foundations. Walls shown dotted typically extend 1 level and typically serve to retain 1 story of lateral earth pressure. At the North Wing, lateral forces in the east-west direction are resisted by sloped concrete grade beams that extend from the base to Level 4 and act much like braced frames.

Walls are typically 12 inches in thickness. Wall reinforcement is variable. In general, walls are characterized by lighter horizontal reinforcement and heavier boundary reinforcement than would be provided currently. Detailing of walls, and associated collectors and foundations is thorough, indicating that substantial effort was devoted to sizing the seismic lateral force resisting system (presumably optimizing to the code-required lateral strength).

A heavily reinforced 16” thick wall is provided at the western end of the north wing. The performance of this wall will be key to satisfactory performance of the building.

Building code: The building code used for design is not identified on the original structural or architectural drawings. The drawings are dated 1976. The code used for seismic design was probably the 1973 Uniform Building Code (UBC), based on the dates of design and construction.

Building condition: The condition of the structure is good, including exposed-to-view concrete at the building façade. We observed no evidence of distress or deterioration on site.

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:

- Columns are shear-governed because of wide tie spacing. Tier 2 analysis might be able to conclude that columns are adequate at some locations based on actual imposed drift demands.
- Various of the columns are shortened by spandrel and sill elements, with increased risk of shear failure at these locations.
- The building is expected to have plan-torsion response to earthquake ground shaking in north-south direction, because of the more substantial extent of wall on upslope side for purpose of earth retention. Torsion may be less pronounced in Tier 2 or Tier 3 a 3-dimensional linear model, based on the stepping back of upper levels.
- Given the low horizontal reinforcement ratio of the walls, they may exhibit shear-critical behavior.
- Shear walls also serve to support the floors for gravity loads, increasing the risk of partial collapse in the event of wall shear failure.
- The interaction of the building with the hillside should be expected to alter seismic response.
- Some concrete walls at the penthouse are discontinuous.

Potential deficiencies may compromise seismic performance. Columns and shear walls vulnerable to shear failure may pose an unacceptable risk of partial collapse.

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

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

Structure and façade are all of reinforced concrete cast-in-place construction. No falling hazards were observed. There is a seismic shutoff on natural gas supply and no gas-fired major equipment. Medical gas cylinders were observed to be restrained.

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

Discussion of rating

We assign a rating of V because of the vulnerability of columns and walls to shear failure. Because of plan-torsion eccentricity, the highest vulnerability is in the western part of the building under north-south seismic motion.

Recommendations for further evaluation or retrofit

A more detailed seismic evaluation is planned. Some insight into the plan-torsion response should be gained from linear dynamic analysis, if such analysis is able to model the hillside condition and the potential for the structure to ratchet away from the hillside**.** Further linear analysis is not likely to change the rating, but it could help identify the required retrofitting. If linear analysis results indicate that nonlinear analysis would be beneficial, NLRHA could better model the effect of wall shear failure, plan torsion, and the hillside condition.

Applicable retrofit measures may include fiber wrapping of columns to address shear failure, and adding concrete walls, in particular in the north-south direction.

Peer review comments on rating

Structural members of the UCSF Seismic Review Committee (SRC), reviewed a preliminary presentation of this evaluation on 25 June 2019. Three structural members of the SRC (Phipps, Lizundia, Moore) also reviewed the final report on 29 July 2019. The panel generally agreed that a Seismic Rating of V is appropriate.

Additional building data	Entry	Notes
Latitude	37.7617	
Longitude	-122.4610	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	4	The partial basement daylights on the west side, but reviewer considers Level 1 to be "Base"
Number of stories (basements) below lowest perimeter grade	1	Partial basement and partial crawl space below Level 1
Building occupiable area (OGSF)	135,951	From UCOP spreadsheet
Risk Category per 2016 CBC 1604.5	III	Occupant load > 500 and contains educational occupancy above 12th grade.
Building structural height, h_n	57 ft	Structural height defined per ASCE 7-16 Section 11.2, measured from Level 1.

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

Coefficient for period, C_t	0.02	Estimated using ASCE 41-17 equation 4-4 and 7-18
Coefficient for period, β	0.75	Estimated using ASCE 41-17 equation 4-4 and 7-18
Estimated fundamental period	0.41 sec	Estimated using ASCE 41-17 equation 4-4 and 7-18
Site data		
975-yr hazard parameters S_s, S_1	1.56, 0.616	
Site class	C	
Site class basis	Geotech Parameters	UCSF Group 2 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
Site parameters F_a, F_v	1.2, 1.4	Per ASCE 7-16, Tables 11.4-1 and 11.4-2
Ground motion parameters S_{cs}, S_{c1}	1.872, 0.862	UCSF Group 2 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
S_a at building period	1.87	
Site V_{s30}	680 m/s	
V_{s30} basis	Estimated	UCSF Group 2 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
Liquefaction potential	No	
Liquefaction assessment basis	Study	UCSF Group 2 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
Landslide potential	No	
Landslide assessment basis	Study	UCSF Group 2 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
Active fault-rupture identified at site?	No	
Fault rupture assessment basis	Study	UCSF Group 2 Buildings –Tier 1 Geotechnical Assessment, Egan (2019)
Site-specific ground motion study?	No	
Applicable code		
Applicable code or approx. date of original construction	Built: 1976 Code: 1973 UBC	Code not identified on drawings, assumed based on date
Applicable code for partial retrofit	None	No partial retrofit known
Applicable code for full retrofit	None	No full retrofit known
Model building data		
Model building type North-South	C2 Conc. wall	
Model building type East-West	C2 Conc. wall	
FEMA P-154 score	N/A	Not included here because we performed ASCE 41 Tier 1 evaluation.
Previous ratings		
Most recent rating	IV	In spreadsheet. Basis for rating is unknown
Date of most recent rating	-	Rating date is unknown
2nd most recent rating	Good	In spreadsheet. Basis for rating is unknown
Date of 2nd most recent rating	-	Rating date is unknown

3rd most recent rating	-	
Date of 3rd most recent rating	-	
Appendices		
ASCE 41 Tier 1 checklist included here?	Yes	Refer to attached checklist file

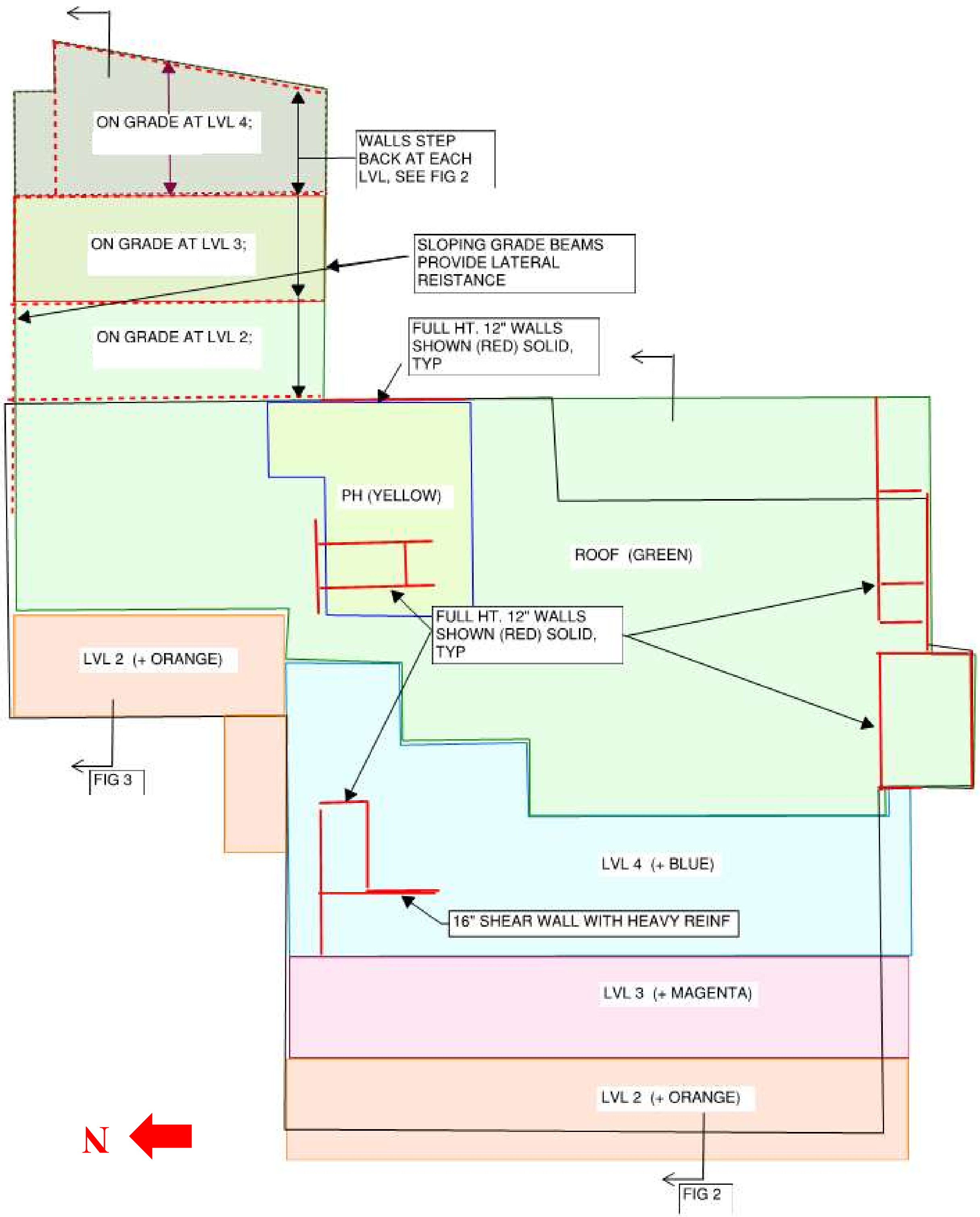


Figure 1: Diagrammatic Plan

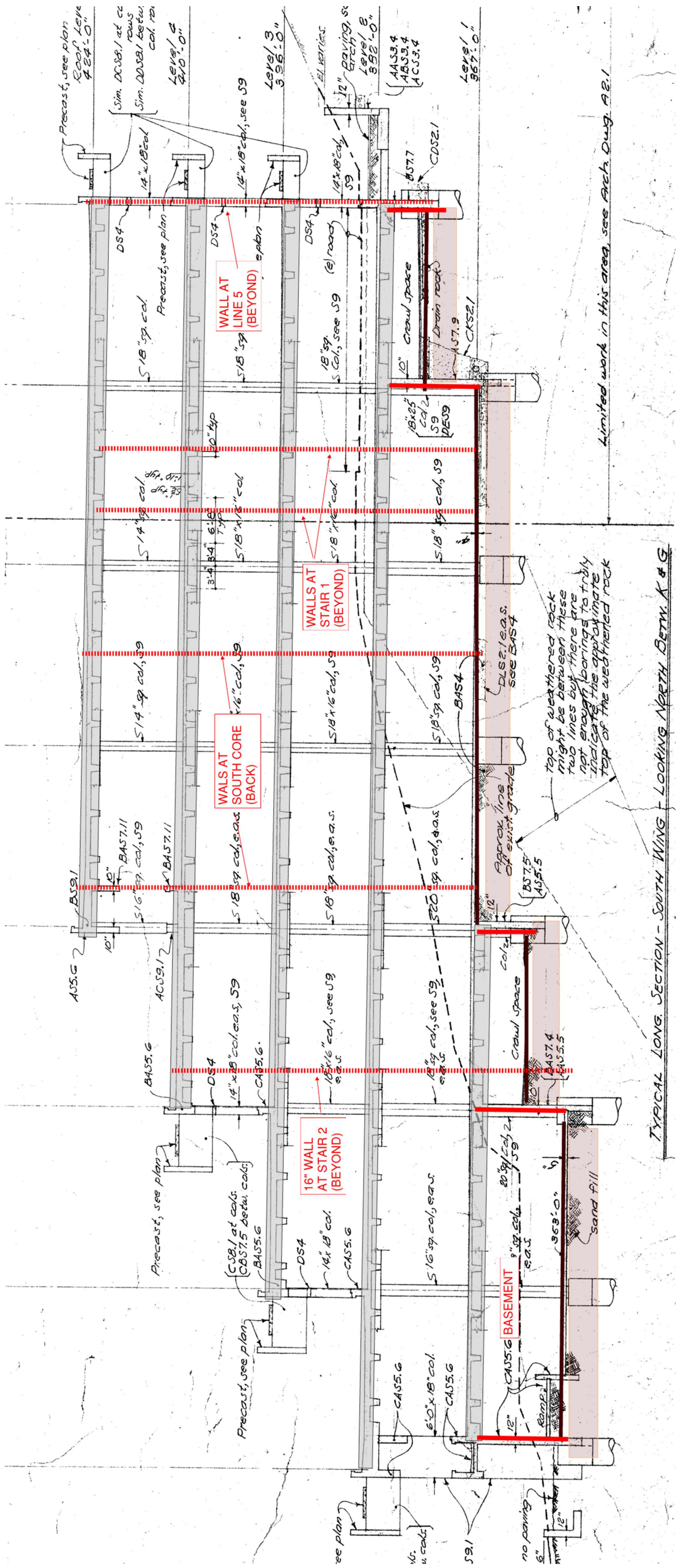


Figure 2: Section at South Wing

Figure 3: Section at North Wing

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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:**
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:**
C ● NC ○ N/A ○ U ○	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3) **Comments:**

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

	Description
C ● NC ○ N/A ○ U ○	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1) **Comments:**
C ● NC ○ N/A ○ U ○	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2) **Comments:**
C ○ NC ● N/A ○ U ○	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3) **Comments:** Generally true, with exception of wall elements at Penthouse. Closely spaced ties are added at columns supporting discontinuous wall elements.

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

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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:** There are many stiffness irregularities in the lateral force resisting system of this hillside structure that warrant the use of a dynamic analysis. However, the analysis model will be quite complicated and will involve consideration of how best to treat tie to on-grade elements at upper levels. The typical Tier I stress check of walls is poorly suited to this building.
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:** There is change of mass (less than 50%) at most levels, due to changes in floor area of terraced building. There is change of mass exceeding 50% between Level 1 and Level 2 due to reduction in floor area (not on grade), however Level 1 is judged to be the "Base" of the building by reviewer.
NC	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:** Due to the terracing of the building into the hillside and presence of retaining walls and foundations at grade at upper levels, the center of rigidity is pushed to the rear of the building, in particular when assessed using a story by story approach. Expect much less torsional when assessed by 3d dynamic analysis and considering torsional resistance of orthogonal elements..

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

GEOLOGIC SITE HAZARD

C NC N/A U	Description
C	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:** Although site is mapped as moderate liquefaction potential, Egan identifies actual potential is very low.
C	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:** Moderate slope (<15°); steeper upward to east. Building and associated retaining well founded on drilled piers and tied foundations.
C	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:** 5-1/4 miles to San Andreas, which is closest fault.

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

FOUNDATION CONFIGURATION

C NC N/A U	Description
C ○ NC ● N/A ○ U ○	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3) **Comments:** Noncompliant at some wall locations. However, it is obvious from review of drawings that substantial attention was devoted to providing vertical trim reinforcing at wall ends and developing the reinforcing into a substantial foundation of grade beams and large diameter drilled piers. Reviewer does not expect overturning to be a substantial deficiency.
C ● NC ○ N/A ○ U ○	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4) **Comments:** Much better than average attention to foundation ties, typical of Isador Thompson designed structures, and perhaps more important than average for this hillside structure.

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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:** Although there is a generally complete frame, walls do support gravity loads at several locations.
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:**
NC	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:** As discussed in basic checklist, this building is too complex to form an opinion with an average degree of confidence based on this check. Unusual features include grade beams that slope with the hillside to form pseudo-braced-frames. A Tier 2 level check is planned and considered necessary to rate building with an average degree of confidence. See calculations for some checks intended to be an indicator of compliance.
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:**

Connections

C NC N/A U	Description
NC	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:** Penthouse is framed with concrete walls with steel deck roof. Where deck frames strongway to wall, ledger angle is attached with 1/2" drilled anchors of an older vintage. Hazard is low as roof and fan room are not typically occupied and wall span out of plane between end walls is moderate.
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:** Drawings evidence substantial attention to provision of collectors into shear walls. Although it is expected that collectors are noncompliant with present standards, they are considered to be much better than average and those collector types and details that have led to collapse in past earthquakes.

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Collapse Prevention Structural Checklist For Building Type C2-C2A

C NC N/A U	
● C ○ NC ○ N/A ○ U	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:** Drawings evidence substantial attention to detailing of foundation to grade beam and wall connections.

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

Seismic-Force-Resisting System

	Description
○ C ● NC ○ N/A ○ U	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:** Concrete columns typically have widely spaced #3 ties (16"o.c.) with a few ties at 8" spacing top and botltom. There is a wide assortment of column sizes and reinforccment. Columns checked using ASCE 41 Eqn 10-3 (including expected strength for shear capacity and u = 4.5 at ends and 2 at midheight), or using ACI 318 are noncompliant. Columns may be okay using Krolicki 2011 model for column shear. Perform Tier 2 analysis.
● C ○ NC ○ N/A ○ U	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3) **Comments:**
● C ○ NC ○ N/A ○ U	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:**

Diaphragms (Stiff Or Flexible)

	Description
● C ○ NC ○ N/A ○ U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1) **Comments:**
○ C ● NC ○ N/A ○ U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3) **Comments:** Diaphragm openings occur adjacent to walls at stair and elevator shafts. Wall n line 6.7 between E and F is most compromised. Drawings show attention to the provision of collectors at such walls.

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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:**
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:**
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:**
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:**
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:**

Connections

C NC N/A U	Description
C	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:** Large diameter cast-in-place concrete piers well anchored to well reinforced foundation grade beams

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

SEISMIC EVALUATION OF EXISTING BUILDINGS - TIER 1 SCREENING

ASCE 41-17 Chapter 4

General

Architect	John Funk & Associates Architects		
Structural Engineer	Isador Thompson and Associates		
Location	707 Parnassus Ave, San Francisco, CA 94131		
Design date	1976		
Latitude	37.76167		
Longitude	-122.46100		
Stories above grade	4		

Seismic parameters

Risk Category	III	(CBC 2013 Table 1604.5)
Site Class	C	Egan, 06/2019
Liquefaction hazard	Very Low	Egan, 06/2019
S_{CS}	1.87	Egan, 06/2019
S_{C1}	0.86	Egan, 06/2019
S_{rS}	0.907	Egan, 06/2019
S_{r1}	0.409	Egan, 06/2019

Scope

Performance level	See Table 2-1
Seismic hazard level	BSE-C
Level of seismicity	High
Building type	C2: Concrete shear walls with stiff diaphragms

Material properties

Concrete	f'_c	Specified stength on Sheet S1		
		4,000	Ltwt	Floors, precast
		4,000	NWC	Walls, GB, Piers
		5,000	NWC	Columns, typ; Walls, as noted
Reinf.	f_y	40	ksi	Typical, U.O.N.
		60	ksi	As noted; typ for wall boundaries

Checklists

Benchmark building	No
Checklist(s) req'd	ASCE 41-17 Collapse Prevention Structural Checklist for Building Type (
	ASCE 41-17 Collapse Prevention Basic Configuration
	UCOP SEISMIC SAFETY POLICY Falling Hazard Assessment Summary

Seismic forces

V	50657	kip	$V = Cs_a W$	= 1.87W
W	27089	kip	building weight	
C	1.0			
S_a	1.87	g	$S_a = S_{x1}/T \le S_{XS}$	
T	0.41	sec	$T = C_t h_n^{\beta}$	
C_t	0.020			
β	0.75			
h_n	57	ft	building height	

Story Forces

Story	w	story ht	h	wh^k	F_{story}	(4-3a) F_{story}	(4-3b) V_{story}
	kip	ft	ft			kip	kip
Roof & PH	5623		57	320533	0.34	17305	
4	6939	14.0	43	298356	0.32	16108	17305
3	7248	14.0	29	210206	0.22	11349	33413
2	7279	14.0	15	109186	0.12	5895	44762
1		15.0	0				50657
Total	27089			938282	1.0	50657	

k 1.00 k = 1.0 for T < 0.5, 2.0 for T > 2.5, linear interpolation between

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

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

Shear stress in shear walls

Story	$A_{w\,N\text{-}S}$	$A_{w\,E\text{-}W}$	(4-9) v_{NS}^{avg}	(4-9) v_{EW}^{avg}	D/C_{NS}	D/C_{EW}
	in^2	in^2	psi	psi		
Roof & PH						
4	17250	28000	502	309	3.5	2.2
3	42000	39672	398	421	2.8	3.0
2	62900	44000	356	509	2.5	3.6
1	66800	57120	379	443	2.7	3.1
Total						

M_s 2.0 (Table 4-9)

v_{limit} 141 psi $v_{limit} = 2\sqrt{f_c'} \ge 100$ psi, f'c is spec'd strength

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

Notes:

1. Building is too irregular for meaningful Tier 1 analysis. However, magnitude of DCR's indicates that building fails "Quick Checks" regardless of precision.

2. N-S Wall area between L4 and Roof adjusted to reflect that wall at south end is overweighted. 2/3 of shear is assigned to walls norths of Line G.

3. Accumulation of shear (V_{story}) does not recognize that some story shear from upper levels above does not continue to base due to terracing at rear of North Wing. Wall area from levels above is added to levels below to consider.

Column H7 at L2

Input

$d_{bs} := 0.375\ in$	Diameter of tie rebar
$legs := 3$	No. of legs of ties
$A_v := legs \cdot \frac{\pi}{4} \cdot d_{bs}^2 = 0.331\ in^2$	Area of shear reinforcement
$s_1 := 16\ in$	Spacing of ties at middle of the column
$s_2 := 8\ in$	Spacing of ties at end of the column
$d_b := 1.27\ in$	Diameter of longitudinal rebar
$A_s := 10.16\ in^2$	Area of longitudinal reinforcement
$f_y := 1.25 \cdot 40\ ksi = 50\ ksi$	Strength of steel
$f'_c := 1.5 \cdot 5\ ksi = 7.5\ ksi$	Strength of concrete
$b := 16\ in$	Width of the column
$h := 18\ in$	Depth of the column
$H := 12.167\ ft$	Height of the column
$cc := 1.5\ in$	Clear cover to ties
$c := 6.46\ in$	Depth of NA
$\lambda := 1$	For normal weight concrete
$N_u := 423\ kip$	Axial force
$M_u := 405.8\ kip \cdot ft$	Design moment
$\mu_{\Delta 1} := 2$	Displacement ductility at the middle of the column where tie spacing is
$\mu_{\Delta 2} := 4.5$	Displacement ductility at ends of the column where tie spacing is small
$Curvature := \text{“Double”}$	**Single** or **Double** curvature

Column H7 at L2

ASCE 41-17 Eq. 10-3

$d := h - cc - d_{bs} - \frac{d_b}{2} = 15.49\ in$ Effective depth

$A_g := b \cdot h = 288\ in^2$ Cross sectional area

$V_u := \frac{2 \cdot M_u}{H} = 66.705\ kip$ Design shear

$$k_{nl1} := if\left(\mu_{\Delta 1} \le 2, 1, if\left(\mu_{\Delta 1} \ge 6, 0.7, 1 + \left(\frac{0.7-1}{6-2} \cdot \left(\mu_{\Delta 1} - 2\right)\right)\right)\right)$$

Shear strength degradation factor based on ductility

$k_{nl1} = 1$

$$\alpha_{col1} := if\left(\frac{s_1}{d} \le 0.75, 1, if\left(\frac{s_1}{d} \ge 1, 0, 1 + \left(\frac{0-1}{1-0.75} \cdot \left(\frac{s_1}{d} - 0.75\right)\right)\right)\right)$$

Dimension parameter for effectiveness of transverse reinforcement

$\frac{s_1}{d} = 1.033$ $\alpha_{col1} = 0$

$$V_{col1} := k_{nl1} \cdot \left(\left(\alpha_{col1} \cdot \left(\frac{A_v \cdot f_y \cdot d}{s_1}\right)\right) + \left(\lambda \cdot \left(\frac{6\sqrt{f'_c \cdot psi}}{\frac{M_u}{V_u \cdot d}} \cdot \sqrt{1 + \frac{N_u}{6 \cdot A_g \cdot \sqrt{f'_c \cdot psi}}}\right) \cdot 0.8\ A_g\right)\right)$$

Shear strength of th columns in middle

$V_{col1} = 49.692\ kip$

$$k_{nl2} := if\left(\mu_{\Delta 2} \le 2, 1, if\left(\mu_{\Delta 2} \ge 6, 0.7, 1 + \left(\frac{0.7-1}{6-2} \cdot \left(\mu_{\Delta 2} - 2\right)\right)\right)\right)$$

Shear strength degradation factor ba ductility

$k_{nl2} = 0.813$

$$\alpha_{col2} := if\left(\frac{s_2}{d} \le 0.75, 1, if\left(\frac{s_2}{d} \ge 1, 0, 1 + \left(\frac{0-1}{1-0.75} \cdot \left(\frac{s_2}{d} - 0.75\right)\right)\right)\right)$$

Dimension parameter for effectiveness of transverse reinforcement

$\frac{s_2}{d} = 0.516$ $\alpha_{col2} = 1$

$$V_{col2} := k_{nl2} \cdot \left(\left(\alpha_{col2} \cdot \left(\frac{A_v \cdot f_y \cdot d}{s_2}\right)\right) + \left(\lambda \cdot \left(\frac{6\sqrt{f'_c \cdot psi}}{\frac{M_u}{V_u \cdot d}} \cdot \sqrt{1 + \frac{N_u}{6 \cdot A_g \cdot \sqrt{f'_c \cdot psi}}}\right) \cdot 0.8\ A_g\right)\right)$$

Shear strength c in end zones

$V_{col2} = 66.438\ kip$

Column H7 at L2

Krolicki Model (Krolicki et al (2011) "Shear Strength of Walls Subjected to Cyclic Loading" Journal of Earthquake Engineering)

$l_w := h = 18\ in \qquad h_w := H$

$\rho_g := \dfrac{A_s}{b \cdot h} = 0.035$ Longitudinal reinforcement ratio

$\alpha_P := max\left(3 - \dfrac{M_u}{V_u \cdot l_w}, 1\right) = 1$ Shear coefficient to account for span ratio

$\beta := min\left(0.5 + 20 \cdot \rho_g, 1\right) = 1$ Shear coefficient to account for longitudinal reinforcement

$\gamma_{p1} := if\left(\mu_{\Delta 1} \le 2, 3.5, if\left(\mu_{\Delta 1} \ge 6, 0.6, 3.5 + \left(\dfrac{0.6 - 3.5}{6 - 2} \cdot \left(\mu_{\Delta 1} - 2\right)\right)\right)\right) \qquad \gamma_{p1} = 3.5$

$\gamma_{p2} := if\left(\mu_{\Delta 2} \le 2, 3.5, if\left(\mu_{\Delta 2} \ge 6, 0.6, 3.5 + \left(\dfrac{0.6 - 3.5}{6 - 2} \cdot \left(\mu_{\Delta 2} - 2\right)\right)\right)\right) \qquad \gamma_{p2} = 1.688$

$V_{c1} := \alpha_P \cdot \beta \cdot \gamma_{p1} \cdot \sqrt{f'_c \cdot psi} \cdot \left(0.8 \cdot A_g\right)$ Shear strength of concrete in middle zone

$V_{c1} = 69.836\ kip$

$V_{c2} := \alpha_P \cdot \beta \cdot \gamma_{p2} \cdot \sqrt{f'_c \cdot psi} \cdot \left(0.8 \cdot A_g\right)$ Shear strength of concrete in end zone

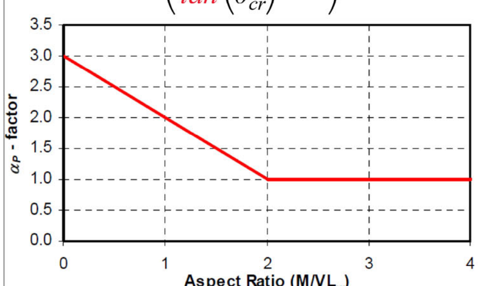
$V_{c2} = 33.671\ kip$

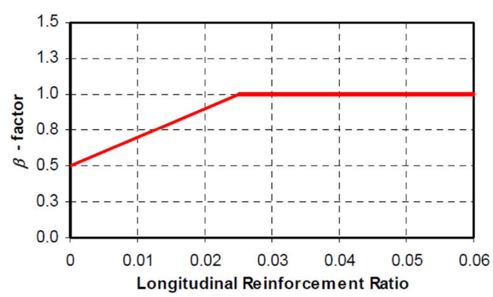
$\theta_{cr} := max\left(\left(\dfrac{35° - 45°}{2}\right) \cdot \left(\dfrac{M_u}{V_u \cdot l_w}\right) + 45°, 35°\right) = 35°$

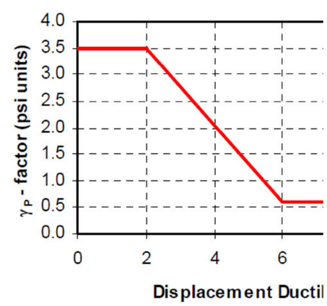
$c_o := cc - d_{bs} = 1.125\ in$ Cover to the main rebars

$l' := l_w - c - c_o = 10.415\ in$ Horizontal length of the crack

$h_{cr} := min\left(\dfrac{l'}{tan\left(\theta_{cr}\right)}, h_w\right) = 14.874\ in$ Vertical height of the inclined crack







Column H7 at L2

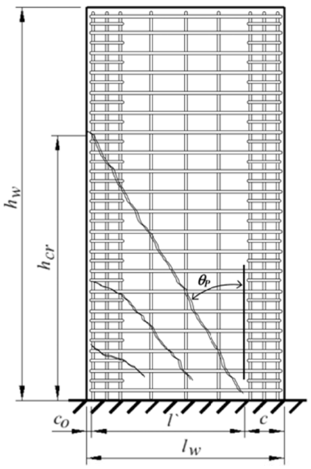
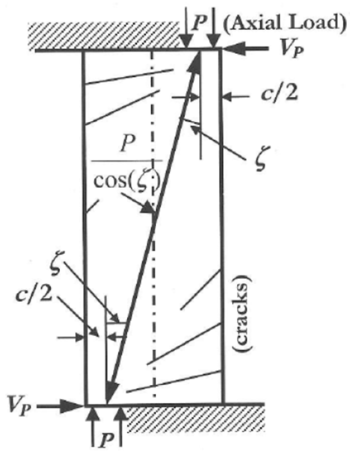
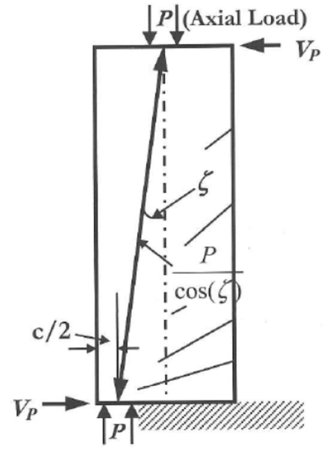


Figure 11.4. Average crack angle, θ



(a) Double Curvature



(b) Single Curvature

$V_{s1} := \dfrac{A_v \cdot f_y \cdot h_{cr}}{s_1} = 15.401 \ kip$ Shear strength of ties in middle zone

$V_{s2} := \dfrac{A_v \cdot f_y \cdot h_{cr}}{s_2} = 30.803 \ kip$ Shear strength of ties in end zone

$$\zeta := if\left(Curvature = \text{“Single”}, atan\left(\frac{\frac{l_w - c}{2}}{H}\right), atan\left(\frac{l_w - c}{H}\right)\right) = 4.519 \ °$$

$V_p := N_u \cdot tan(\zeta) = 33.433 \ kip$

$V_{n1} := V_{c1} + V_{s1} + V_p$ Shear strength of column in middle zone

$V_{n1} = 118.671 \ kip$

$V_{n2} := V_{c2} + V_{s2} + V_p$ Shear strength of column in end zone

$V_{n2} = 97.907 \ kip$

Comparison between ASCE 41-17 & Krolicki model

$\dfrac{V_{col1}}{V_{n1}} = 0.419$ Comparison at the middle of the column

$\dfrac{V_{col2}}{V_{n2}} = 0.679$ Comparison at the end of the column