UCSF

Rating form completed by:

ARUP

Evaluator: BT, JW, KN

Date: 2/11/2020

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

DATE: 2020-02-11

UCSF building seismic ratings

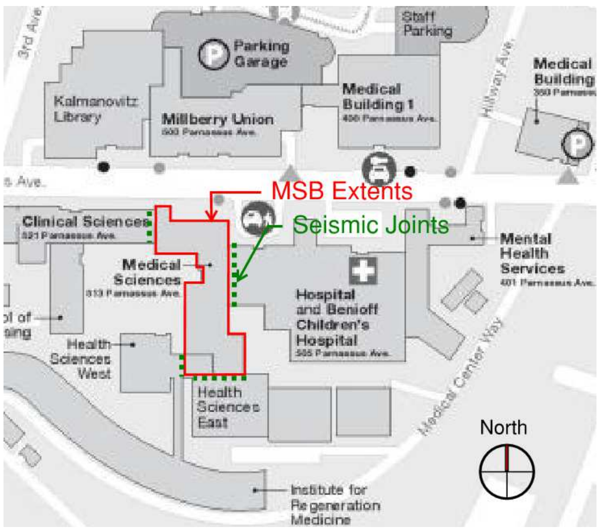
Medical Sciences Building

CAAN #2252

513 Parnassus Avenue, San Francisco, CA 94143

UCSF Campus: Parnassus

Plan



North elevation (photo credit: Google)

Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	V	
Rating basis	Tier 2	Evaluation included site visit and linear dynamic analysis using a three-dimensional structural model. More detailed nonlinear analysis is recommended.
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 project cost to retrofit to IV rating	iv	(i) Low: less than $50 per sf (ii) Medium: greater than $50 per sf and less than $200 per sf (iii) High: greater than $200 per sf and less than $400 per sf (iv) Very High: greater than $400 per sf
Is 2018-2019 rating required by UCOP?	Yes	

Further evaluation recommended?	Yes	See later discussion.

Building information used in this evaluation

- Original design drawings: “Medical Sciences Building Increment No 1,” dated September 22, 1950. Architects Blanchard & Maher and civil engineers Huber & Knapik.
- Original design drawings: “Medical Sciences Building Increment No 2,” dated March 1, 1955. Architects Blanchard & Maher and civil engineers Huber & Knapik.
- Original design drawings showing south addition to MSB: “Health Sciences Instruction and Research Unit 1,” dated July 25, 1962. Reid Rockwell Banwell & Tarics Architects and Engineers.
- Seismic improvement design drawings: “UCSF MSB Improvements Phase 2: BP1A – Seismic Upgrade,” dated October 2, 2006. Architect The Design Partnership; Engineer Degenkolb.
- Previous seismic evaluation calculations: “UCSF Medical Science Building MSB Improvements Phase II,” dated February 7, 2005. Degenkolb Engineers.
- Previous seismic study for Moffitt and MSB: “UCSF Medical Center: Moffitt/MSB Seismic Study,” dated July 2003. Degenkolb Engineers.
- Bedrock elevations: “Campus of the University of California at San Francisco Showing Bedrock Contours,” based on report by Chester Marliave dated November 22, 1948.
- Seismic hazard report: “Health Science Instruction + Research: Seismic Improvements,” dated December 21, 2018. Maffei Structural Engineering.
- Geotechnical characteristics and geohazards: letter from John Egan dated June 26, 2018 and updated July 25, 2019, project number 1024, subject “UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, San Francisco, California”. Note that the response spectrum used for this analysis is not from this document but from the site-specific study by Maffei Structural Engineering above.
- Comparison of response spectra: “Comparison of Earthquake Response Spectra, UCSF Parnassus Campus, San Francisco, California,” technical memorandum by John Egan dated December 6, 2019.
- Post-Loma Prieta inspection report: “Performance of UCSF Buildings During the October 17, 1989 Loma Prieta Earthquake,” report number 01-3690-1787 Revision A, by Impell Corporation, dated November 17, 1989.

Additional building information known to exist

- Laboratory test results for in situ structural concrete. Such testing is referenced in the Degenkolb calculations but the results could not be located.

Scope for completing this form

Original structural design drawings were reviewed, as well as subsequent studies as referenced. A site visit was made on 15 April 2019, during which general observation of the interior and exterior configuration were made and non-structural falling hazards were reviewed. An ASCE 41-17 Tier 1 evaluation was made utilizing the prescribed structural checklists, and subsequently a three-dimensional analysis model was constructed for carrying out limited linear dynamic analysis. That supplemental analysis is considered a Tier 2 evaluation in this report.

Brief description of structure

The MSB was designed and constructed in two phases during the 1950s, with a small addition constructed in 1962. All three of these increments are seismically joined. It is a concrete and steel structure, L-shaped in plan, with 13 stories above grade and one basement level. The building is situated immediately adjacent to other buildings on three sides: Moffit Hospital to the east, the Clinical Sciences Building to the west, and two Health Sciences towers to the south. The building fronts Parnassus Avenue to the north.

Identification of levels: The building levels are designated as follows: Basement (El. 384’-6”), 1st Floor (El. 397’-6”), 2nd to 13th Floor (El. 411’-0” to 554’-0”), 14th Floor and Roof (El. 567’-0”). Above the main roof are a penthouse and mechanical room, whose roof is at elevation 600’-7”. The surrounding grade slopes downward to the north and east, with elevations ranging from approximately 397 feet to 424 feet.

Foundation system: The MSB rests on shallow foundations over a relatively dense sand near the foot of Mount Sutro. Interior columns bear on isolated spread footings and perimeter walls on narrow continuous spread footings, with enlargements at perimeter columns. Footings have pedestals indicated in the design drawings as poured monolithically with the ground slab; however, the presence of a positive connection between footings and the ground slab could not be confirmed. Bedrock exists at a moderate depth, but the bedrock slopes more steeply than the surface grade.

Structural system for vertical (gravity) load: The gravity system consists of steel columns and beams integrated with concrete slabs and beams. Columns are steel wide flange sections with lightly-reinforced concrete encasement for fireproofing. Perimeter columns are embedded in more substantially-reinforced pilasters in the basement walls. Primary girders in the floor framing are concrete-encased steel wide flange sections. The floor slabs are formed one-way-spanning concrete slabs supported by concrete beams, which in turn span to the composite steel girders. The perimeter walls and portions of the floor slabs are supported by steel angle trusses embedded in concrete beams.

Structural system for lateral forces: The seismic system likewise integrates steel and concrete components. Reinforced concrete floor slabs act as diaphragms which distribute inertial forces to the vertical elements of the lateral force-resisting system. The vertical elements consist of two primary systems: steel brace frames encased in concrete shear walls, and a concrete-encased steel spandrel truss system which wraps around most of the exterior facades. Though the spandrel system has a role in supporting a portion of the floor weight, it also acts in flexure to couple the shear walls and to engage the perimeter columns in resisting horizontal shear forces. The perimeter columns adjacent to shear walls are engaged as outrigger columns to share overturning axial forces with the shear walls. The spandrels engage perimeter columns further from shear walls in the manner of a Vierendeel truss, and these columns carry substantial bending and shear forces.

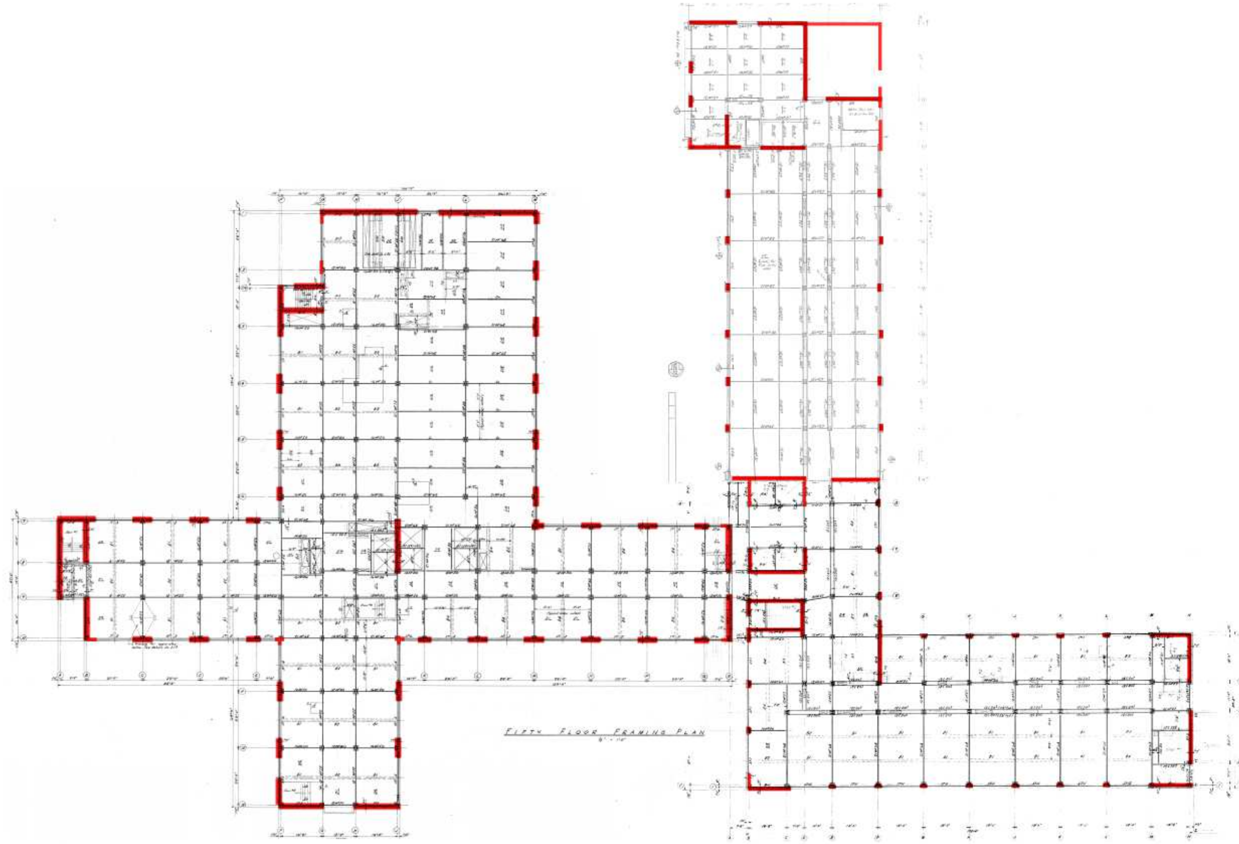
The steel braces are far less stiff than the concrete walls in which they are embedded, and carry a relatively small proportion of the total shear demand. By contrast, the steel columns embedded in the shear walls serve as the primary tension elements resisting overturning forces. Similarly, the steel truss angles embedded in the perimeter concrete spandrel beams serve as the primary chord reinforcement for those elements.

A concrete retaining wall exists around much of the building perimeter, which distributes lateral and overturning forces more uniformly to foundation elements.

Adjacent structures and prior evaluations: Originally the MSB was joined to the Moffitt Hospital building to the east, constructed at about the same time. In 2003 a seismic evaluation was conducted for the joined structure, and it was determined that both buildings would perform better if they were separated. Furthermore, there was a need to remove the MSB from OSHPD jurisdiction. In 2009 a project was carried out to separate MSB from Moffitt Hospital, and also to execute some nominal strengthening of the MSB seismic system. Though the MSB was removed from OSHPD jurisdiction, the stairwell on the east side of the building serves as an exitway for Moffitt, and the strengthening was required by OSHPD in order to permit that use.

Strengthening included the removal of coupling beams on the west face (grid N); infill of openings and strengthening of the south wall (grid 18); strengthening of the chord connections at the re-entrant corner at grid F/4 at multiple floors; and diaphragm chord strengthening along grids 1 and 4 at the 14th floor. Upon completion of those improvements, the building was deemed to have achieved a rating of "good" according to the UCOP rating system at that time.

It should be noted that though Moffitt Hospital and the MSB were constructed at the same time, these two buildings are structurally different. Any deficiencies that exist in one building may or may not be present in the other. The figure below shows a comparison of the shear wall layouts (marked in red) for the two buildings at a typical floor. In the figure, Moffitt Hospital is the building to the left. As can be seen, the two buildings differ in their overall plan geometries as well as in the dimensions and distribution of shear wall piers.



The seismic separation between this building and the Health Science towers to the south is inconsistent. Seismic joints were not clearly visible on all floors during the Arup site visit, and utilities crossing the joints do not respect the required relative movement tolerance. There is currently a separate project underway to develop a remedy to this condition. This report assumes that those towers are seismically separated from the MSB.

The seven-story Clinical Sciences Building (CSB) to the west of MSB is currently undergoing a seismic upgrade. The original drawings indicated gaps between these two buildings of two inches up to the second story, and six inches above that level. Furthermore, the floor levels for these buildings do not align, meaning that impact between them during a seismic event could cause damage to columns. The CSB retrofit design thus incorporates some remedial measures to address the potentially insufficient seismic gap. The first is the removal of some material from the seventh floor and roof of the CSB, such that the gap at those floors is widened to nine inches. The second is that the CSB has been designed to be very stiff in the east-west direction, limiting roof displacement to five inches under the BSE-C hazard level. It is expected that these measures will have substantially reduced the risk of pounding between these two buildings.

<u>Building code</u>: The complete structure was built in three increments, each separated by several years. Available drawings for the first increment do not reference a governing building code and do not carry a building official's stamp, but the drawings are dated September 22, 1950. This structure may have been subject to the 1949 Uniform Building Code. Available drawings for the second increment are likewise not stamped by a building official but carry a date of March 1, 1955. This increment should have been subject to the 1952 Uniform Building Code, which was the next edition after 1949. The third increment, for which available drawings are dated May 25, 1962, also does not reference a governing code or carry a building official's stamp. It is surmised that this increment would have been subject to the 1961 Uniform Building Code, which was three editions after the 1952 code.

These building codes would all pre-date the benchmark standards for steel or concrete systems as given in ASCE 41, and though limited retrofit work has been carried out according to a recent standard, this structure is assumed not benchmarked for the purpose of this evaluation. For the purpose of this evaluation, this structure should be considered not benchmarked, and an ASCE 41 Tier 1 evaluation is necessary.

A partial retrofit of the structure occurred in 2009, which was carried out according to the 2001 California Building Code.

Building condition: Good. During the site visit the building was observed to be well maintained, with no evidence of deterioration.

Building response in 1989 Loma Prieta Earthquake: No evidence of damage due to this earthquake was observed during the Arup site visit. An inspection of the UCSF campus conducted shortly after the earthquake by the Impell Corporation revealed very little damage. Most of the damage was to interior partitions and the façade (presumably the ceramic cladding). There was, however, some superficial damage (fallen facade tiles) due to pounding against the CSB to the west. The building was judged safe for immediate occupancy.

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

The following is a compilation of deficiencies identified through the Tier 1 evaluation and the supplemental Tier 2 analysis. Most Tier 1 deficiencies were not found to be mitigated through the Tier 2 analysis, however that analysis better established the relative significance of Tier 1 deficiencies toward the building's overall response characteristics. Some of the Tier 1 deficiencies were found to be not governing.

- Seismic separations between this building and the three other adjacent buildings do not meet the Tier 1 criterion. This criterion stipulates a clear separation of 1.5% of the height to any given floor. At roof level this would equate to a gap of 30.5 inches, as compared to actual gaps that vary from 16 to 21 inches depending on the interface. The Tier 2 analysis suggests that MSB drifts are likely less than 1% at the 975-year event. If adjacent structure drifts are similar, the required gap may be close to the 1.5% criterion. In addition, the seismic gaps between this building and the health sciences towers to the south do not appear to exist below third level upon inspection. Where these structures are detached, the seismic joints are still bridged by utilities which are not detailed for differential displacement. A separate project is examining this issue. A concurrent seismic retrofit project for the CSB to the west has increased the available gap and stiffened that structure in order to address the adjacency issue along that interface. The separation provided at Moffitt to the east, though non-compliant with the 1.5% criterion, was installed in a recent project and based upon analysis of both buildings. Given recent or concurrent attention to this issue at all interfaces, the seismic separation deficiency is presumed to be addressed in this evaluation.
- Steel columns and braces surpass the Tier 1 permissible axial stress by up to approximately 300% and 150%, respectively. The Tier 2 analysis indicated that neither of these failure modes was governing. Column axial demand is primarily induced by overturning moment in the shear walls in which they are embedded, but shear wall capacities tend to be governed by shear rather than flexure. Braces are found to carry a far smaller percentage of the seismic shear than indicated by the Tier 1 analysis, and braces did not fail the Tier 2 checks.
- Concrete shear stresses exceed the Tier 1 permissible shear stress by approximately 200%, and this finding is approximately borne out by the Tier 2 analysis.
- Steel element capacities are governed by non-ductile bolt or rivet shear rupture at end connections. Bolted/riveted column splices cannot develop 50% of the column tensile strength as required by the Tier 1 criteria, and the Tier 2 analysis finds that column splice capacity may govern global building response due to overturning demands at embedded columns. Spandrel chord elements likewise are shown by the Tier 2 analysis to govern spandrel flexural response due to their end connections. Brace connections do not have sufficient capacity to develop brace buckling as required by the Tier 1 criteria; however, the Tier 2 analysis indicates that brace demands are not high enough to cause failure of those connections.

- Perimeter columns are subjected to a combination of axial, bending, shear and torsion where they are joined to stiff spandrel beams which are connected eccentrically to the columns. The Tier 2 analysis determined that the torsion may be at least in part mitigated by the concrete encasement around these nodes, and that the combination of axial, bending and shear forces on the column splices may govern outside the zone of possible torsion.
- K-brace configurations exist in some brace frames, which is not permitted by the Tier 1 evaluation criteria. Also, chevron brace configurations exist where the supporting beam does not meet the Tier 1 strength criteria. Both of these conditions would be a concern if braces were allowed to buckle, as that would lead to a net shear demand on the beam or column. In this building, these frames are embedded in concrete which should effectively restrain the braces against buckling. A brace configuration issue that remains a concern is the eccentricity that is permitted by the design drawings between brace workpoints and the beam-column joints of all brace frames. This will induce shear in columns and is only partially mitigated by the concrete encasement. The design drawings indicate that this eccentricity is to be minimized, but do not place a limit on it. In addition, spandrel chord elements connect into boundary columns at significant eccentricities above and below floor levels, which likewise may affect column performance.
- Some braces do not meet the Tier 1 slenderness criterion. As noted previously, this issue is considered mitigated by the concrete encasement.
- Coupling beams are not specially detailed to maintain strength and stiffness at large rotation demands, and walls adjacent to coupling beams are not detailed with additional reinforcement to support overturning forces acting as non-coupled walls, as required by Tier 1 criteria. In general, the Tier 2 evaluation determined that concrete walls have insufficient overturning capacity whether or not they are coupled, but also that wall segments may be governed by steel column splice failure or by shear failure.
- Significant torsional response is expected to exist due to the building's L shape in plan and asymmetric distribution of lateral stiffness and capacity. The magnitude of the building's torsional response is within the criterion for the Tier 1 check, yet the Tier 2 analysis indicates that torsion should not be discounted as a contributor to overall response.

The table below summarizes the Tier 1 deficiencies and whether they are considered to affect the structure's seismic rating. Deficiencies are considered to not affect the rating if they were found to be mitigated or not governing through the Tier 2 analysis, or as in the case of the adjacent building deficiency, there are current projects underway intended to mitigate them.

Structural deficiency	**Affects rating?**	**Structural deficiency**	**Affects rating?**
Building System: Adjacent Buildings	N	Steel Seismic Force-Resisting System: K-Bracing	N
Concrete Seismic Force-Resisting System: Shear Stress Check	Y	Steel Seismic Force-Resisting System: Column Splices	Y
Concrete Seismic Force-Resisting System: Coupling Beams	Y	Steel Seismic Force-Resisting System: Slenderness of Diagonals	N
Steel Seismic Force Resisting System: Column Axial Stress Check	N	Steel Seismic Force-Resisting System: Chevron Bracing	N
Steel Seismic Force Resisting System: Brace Axial Stress Check	N	Steel Seismic Force-Resisting System: Concentrically Braced Frame Joints	Y
Steel Seismic Force-Resisting System: Connection Strength	Y		

Summary of review of nonstructural life-safety concerns, including at exit routes. [1]

General observations:

- Equipment was typically anchored and/or restrained in the mechanical rooms.

[1] 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.

- Finishes around lobbies and exit corridors did not appear to be heavy or particularly susceptible to dislodging and creating a falling hazard
- The bracing of the gas lines is unknown.
- Much of the building's exterior façade consists of a heavy ceramic tile system, which did not appear to be jointed sufficiently to allow for significant building movement in a large seismic event. It is uncertain how the tiles are anchored to the structure. Passersby could potentially be exposed to tiles falling from great height.
- Laboratory spaces contained many tall objects such as refrigerators and high storage shelves. Evidence of attention to falling hazards was present: high shelves generally had rails and a strapping system existed for restraining refrigerators. However, compliance was inconsistent: straps were not attached to some refrigerators, and anchorage of some restraints appeared to be ad hoc. Some large countertop equipment did not appear to be anchored.
- Hazardous material treatment also appeared somewhat lax. Some liquid nitrogen tanks were observed to be unstrapped and resting on wheeled carts, with hoses attached to nearby stationary equipment. If these tanks were to move and rupture the hoses, a dangerous condition could result. Also, some low-level radioactive material was stored in a way that it could slide off of shelves.
- Falling hazards appeared to be low in most offices. Furniture and bookshelves had a low profile, with some exceptions.
- Classrooms and meeting rooms typically featured large wall-mounted monitors which appeared to be anchored with a robust system. However, it could not be determined whether they are anchored to engineered structure within the wall.

UCOP non-structural checklist item	Life safety hazard?	UCOP non-structural checklist item	Life safety hazard?
Heavy ceilings, feature or ornamentation above large lecture halls, auditoriums, lobbies or other areas where large numbers of people congregate	None observed	Unrestrained hazardous materials storage	Yes
Heavy masonry or stone veneer above exit ways and public access areas	Potentially	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

Basis of Seismic Performance Level rating

The MSB has a clear load path with no significant horizontal or vertical offsets in the lateral system. There is no obvious weak or soft story. The existence of multiple lateral systems (steel brace frames, concrete shear walls, perimeter spandrel system) provides redundancy. These strengths are offset by a few severe deficiencies: the dominance of brittle bolted connection failure mechanisms in the column splices and spandrel truss end connections; and the tendency of concrete shear walls to be lightly reinforced and governed by shear failure.

These deficiencies suggest a structure which may do well in smaller seismic events, but may not possess sufficient ductility to meet the desired performance characteristics in larger events. Preliminary studies suggest that though individual components may fail suddenly when subject to high demands, the structure's redundant systems can serve to re-distribute demands and potentially avoid collapse.

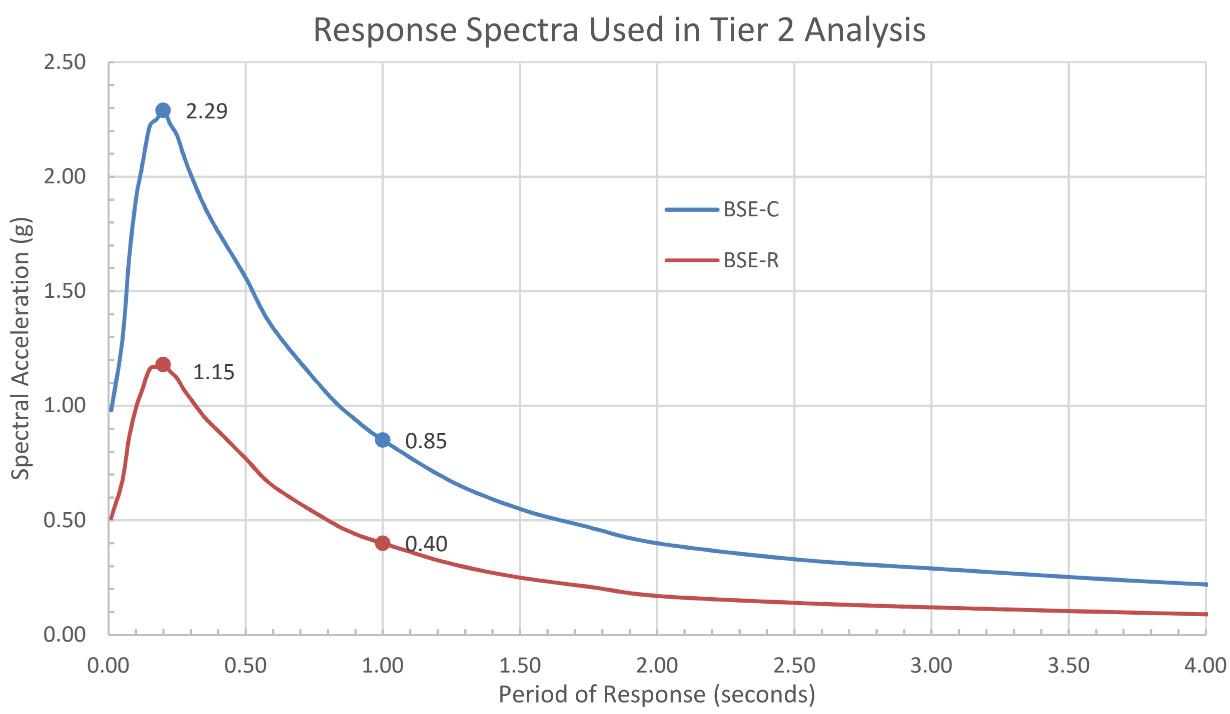
Given the large ductility demands under requisite hazard levels and the brittle nature of failure mechanisms, the evaluators could not justify a IV rating for this building. But the lack of an obvious collapse scenario suggests that a rating of VI may not be warranted. Hence a rating of V has been agreed with the Seismic Review Committee. It is further recommended that a Tier 3 nonlinear analysis be carried out to more completely evaluate the post-yield behavior of this building.

Recommendations for further evaluation or retrofit

Seismic retrofit would be recommended if a rating of IV is desired for this building. Such retrofit should consider as a minimum the selective strengthening of column splices and the strengthening of some concrete shear walls. The full extent of retrofit cannot be determined from analysis conducted to date; however, the analysis does suggest that strengthening should be considered for not just a few but many elements distributed around the building. In addition, an efficient retrofit design should consider the implementation of a demand reduction system such as added damping.

Analyses by linear methods have been carried out to date, along with subcomponent studies to understand some aspects of post-yield behavior. The evaluators feel that more comprehensive nonlinear analysis is warranted. There is likely to be a complex interaction between failure modes, such that the failure of one component substantially alters the subsequent global behavior and load distribution to other components. This interaction is likely to be dynamic in addition to nonlinear, as some failure modes may exert different effects on different modal responses. Hence a nonlinear response history analysis should be part of either a further refinement to the seismic rating or a retrofit effort. Such an analysis is not expected to be able to justify a IV rating, however it may reduce the extent and magnitude of strengthening, as well as enabling the application of non-conventional, dynamics-dependent retrofit methods such as added damping.

It should be further noted that the evaluation of this building was occurring during the time that a more accurate site-specific seismic hazard was under development. Among other things the new hazard included a shift from the older California fault rupture model known as UCERF 2 to the current version UCERF 3. The seismic hazard spectra used for this analysis was substantially more severe than the current understanding of the hazard. See figure below. The Tier 2 analysis indicated demand-capacity ratios sufficiently large that the main findings of this report would likely persist even under the lower hazard levels. Nevertheless, the recommendation for further refinement to the analysis is strengthened by the need to update the seismic hazard spectra.



Many of the non-structural concerns are associated with inconsistency in the application of existing procedures rather than inadequacy of the procedures themselves. More frequent audits are recommended to elicit better compliance, particularly with bracing of hazardous materials. The exterior cladding system may warrant more detailed study to quantify the risk of injury due to falling cladding tiles.

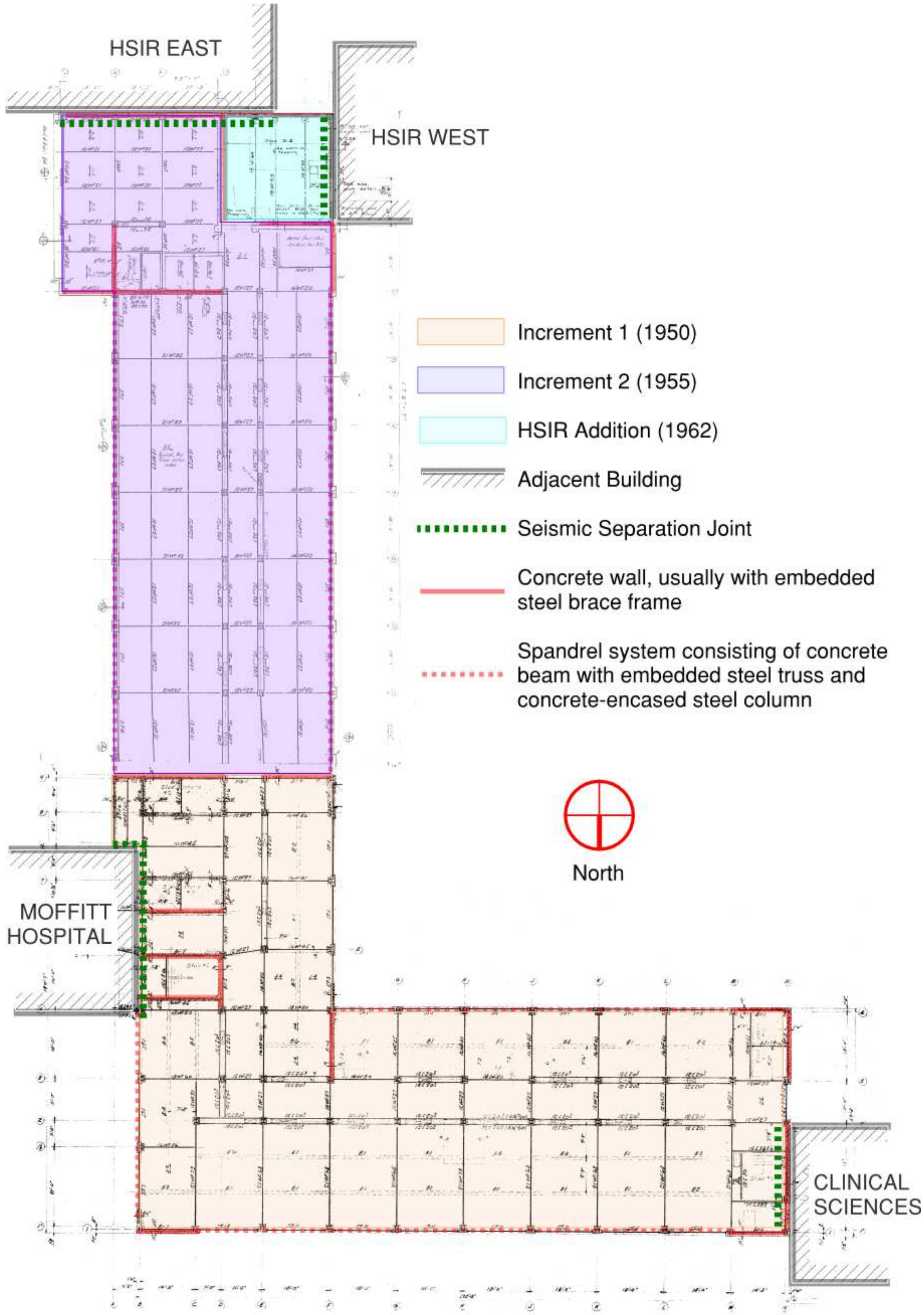
Peer review comments on rating

Four structural members of the UCSF Seismic Review Committee (Lizundia, Moore, Maffei and Phipps) reviewed the evaluation and on July 29, 2019 were unanimous that the Seismic Performance Level Rating is Level V and that further study is required.

Additional building data	Entry	Notes
Latitude	37.76300°	Based on letter from John Egan dated June 26, 2018 and updated July 25, 2019, project number 1024, subject "UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, San Francisco, California"
Longitude	-122.45828°	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	13	Plus penthouse
Number of stories (basements) below lowest perimeter grade	1	
Building occupiable area (OGSF)	350,000	Estimated
Risk Category per 2016 CBC 1604.5	II	Category III may be warranted due to the presence of classrooms which may exceed 250 in total occupancy. The largest auditorium is in an adjacent structure that is seismically isolated from this one. It is not clear whether any of the biological or radiological materials contained in this building would also qualify for a Category III rating
Building structural height, h_n	182.5 feet	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
Estimated fundamental period	1.5 seconds	From modal analysis of elastic three-dimensional model.
Site data		
975-year hazard parameters S_s, S_1	1.544g, 0.609g	Based on UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019). Note that these parameters were under development at the time that this building evaluation was being conducted, and hence these values were not used in the analysis. This evaluation was based on an earlier site-specific response spectrum obtained for the adjacent HSIR site, having a peak spectral acceleration of 2.29g and 1-second spectral acceleration of 0.85g.See plot in previous section.
Site class	C	
Site class basis	.	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Site parameters F_a, F_v	1.2, 1.4	Applied Technology Council website. Note that these factors were not used in analysis. Site specific response spectrum used.

Ground motion parameters S_{cs}, S_{c1}	1.852g, 0.852g	These parameters not used in analysis, see notes above.
S_a at building period	0.55g	First mode period only, using HSIR site-specific response spectrum.
Site V_{s30}	385 m/s	
V_{s30} basis	Estimated	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Liquefaction potential/basis	No	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Landslide potential/basis	No	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Active fault-rupture hazard identified at site?	No	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Site-specific ground motion study?	Yes	From Maffei (2018)
Applicable code		
Applicable code or approx. date of original construction	Built: 1950, 1955, 1962 Code: 1949, 1952, 1961 UBC	Dates represent first, second and third increments, respectively. Codes assumed based on dates of design drawings.
Applicable code for partial retrofit	2001 CBC	
Applicable code for full retrofit	None	No full retrofit known
Model building data		
Model building type North-South	C2 – Concrete shear walls with stiff diaphragms	This is the primary building type assumed. Type S2 – steel brace frames with rigid diaphragms – also exists, but based on elastic stiffness the braces carry much less of the base shear than the concrete walls.
Model building type East-West	C2 – Concrete shear walls with stiff diaphragms	This is the primary building type assumed. Type S2 – steel brace frames with rigid diaphragms – also exists, but based on elastic stiffness the braces carry much less of the base shear than the concrete walls.
FEMA P-154 score	N/A	FEMA P-154 not carried out.
Previous ratings		
Most recent rating	Good	
Date of most recent rating	2009	
2nd most recent rating		Previous ratings not known.
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

Typical floor plan



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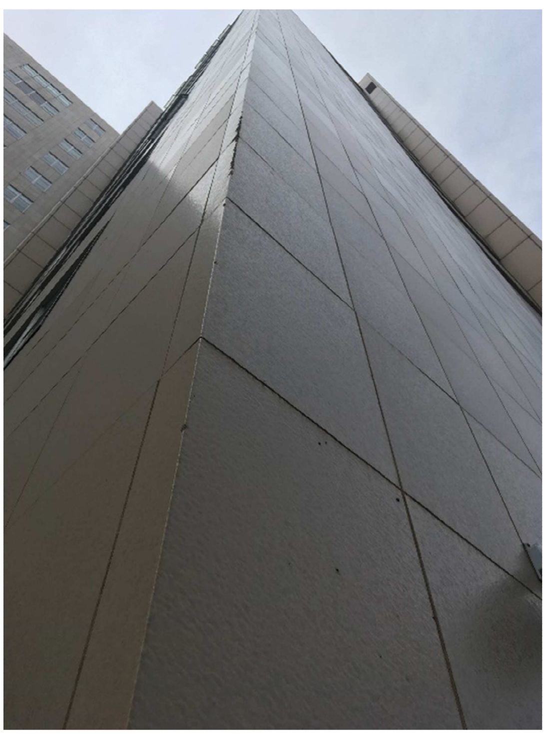
APPENDIX A

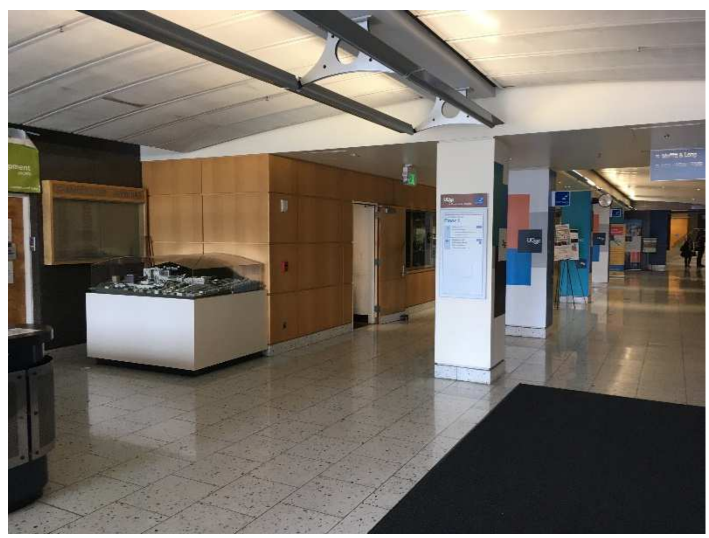
Additional Images

Photograph 1 Exterior view from Parnassus Avenue. Moffitt Hospital is to the left.

Photograph 2 Exterior view from southwest. Note deep spandrels along exterior faces are a significant part of the seismic system.



Photograph 3 Typical ceramic tile cladding system.



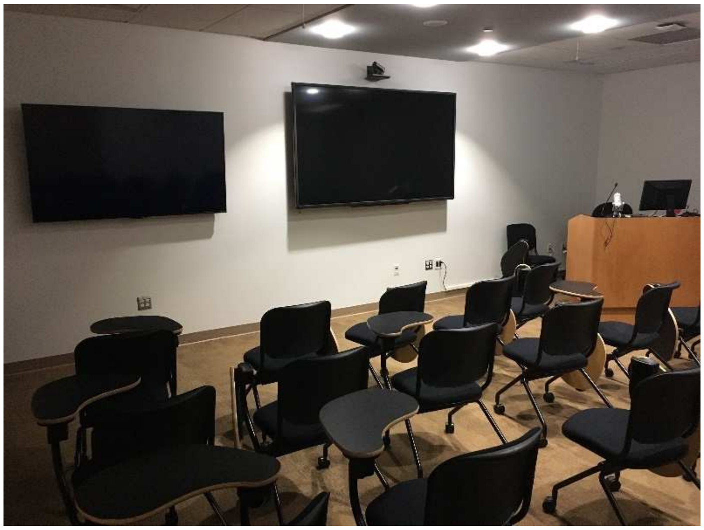
Photograph 4 Interior view of main entry lobby.

Photograph 5 Typical interior hallway.

Photograph 6 Typical office space, furniture mostly low.

Photograph 7 Tall, non-anchored bookshelves in some offices.

Photograph 8 Typical meeting room. Heavy monitors are anchored to walls using a standard system.



Photograph 9 Typical classroom. Heavy monitors anchored to wall using a standard system.

Photograph 10 Typical laboratory space. Rails on high shelves, some heavy items not anchored.

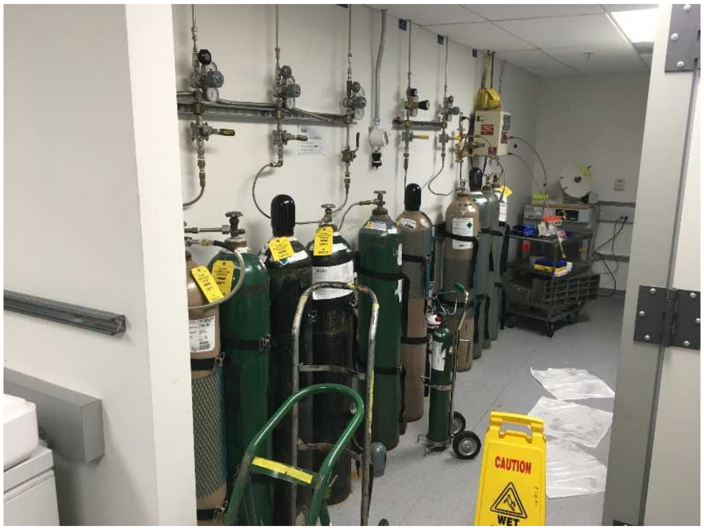
Photograph 11 Laboratory space having high shelving without rails.

Photograph 12 Tall refrigerators often anchored by an engineered system.

Photograph 13 Evidence of equipment anchorage not consistently utilized.

Photograph 14 Hazardous materials tanks with engineered bracing system.

Photograph 15 Evidence of hazardous materials tanks not properly braced, hoses that may rupture with movement.



Photograph 16 Hazardous materials tanks with engineered anchorage system.

Photograph 17 Some tank anchorage does not appear to be engineered.

Photograph 18 Blocked exitway.

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APPENDIX B

ASCE 41-17 Tier 1 Checklists (Structural)

UC Campus:	**San Francisco Parnassus**		Date:	**10/25/2019**	
Building CAAN:	**2252**	Auxiliary CAAN:	By Firm:	**Arup**	
Building Name:	**Medical Sciences Building**		Initials:	**ML**	Checked: **BT**
Building Address:	**513 Parnassus Avenue, San Francisco, CA 94143**		Page:	**1**	of **3**

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

LOW SEISMICITY

BUILDING SYSTEMS - *GENERAL*

	Description
C NC N/A U (C selected)	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:** Gravity load paths are well-defined. No discontinuous columns. Lateral load paths appear to cover all sections of the building.

C NC N/A U (NC selected)

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: The required 1.5% separation would equate to 30.5 inches at the MSB roof. Though seismic separation exists to varying degrees at the three adjacent structures, none of the gaps would comply with that requirement. More detailed analysis would probably suggest that such large separations are not necessary for these stiff buildings. The following table compares actual seismic gaps with the 1.5% Tier 1 criterion:

Adjacent Building	MSB Story Level Corresponding to Roof	Gap Provided	Tier 1 Criterion
CSB (west)	8th	6 inches	16.5 inches
Moffitt (east)	Roof	21 inches	30.5 inches
Health Sciences (south)	Roof	16-19 inches	30.5 inches

Additional notes: Structure has been separated from Moffitt Hospital (east), though these were initially conjoined upon construction in the 1950s. Does not appear to connect structurally to adjacent clinic (west). Clear distance shown in drawings appears to be 4", which is insufficient; however this building is currently under construction for a seismic upgrade, and it is expected that this will be resolved. After visiting the site, it is still not completely clear whether the MSB is consistently separated from the health sciences buildings to the south. A seismic joint exists above grade within the health sciences buildings, but it is not located immediately on the border of the original MSB building and the health sciences building. It appears that a 15-story portion of building was added to the southern tip of MSB, and the joints were placed between this addition and the health sciences buildings. At levels 3 and 4, the seismic joint on the southwest border of MSB only appeared on one side (wall) of the hallway in which the joint was identified.

C NC N/A U (N/A selected)

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

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

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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:** Does not appear to be a significant reduction in strength of the seismic-force-resisting system at any level with respect to the level above.
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:** Story height from the foundation to the first floor appears to be twice as tall in some areas of the building as others. However, this is confined to only a portion of the building, and with the combination of shear walls and braces, it is not expected that a soft story would form under high seismicity.
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:** Braced frame bays appear to be continuous to the foundation (though variance in brace orientation occurs to work around openings). A shear wall from level 4 to the roof along grid 16 between grids J and K appears to have significant openings from level 3 down. There are other walls with opening at various levels, including some which were identified on the site visit and not present in the original plans.
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. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4) **Comments:** There appear to be changes from story-to-story on the southeastern corner of the building to account for openings, but not more than 30%. Significant interior work was done on the first/second floor within the area where increments 1 and 2 join, incuding a large opening of roughly 30-36' in diameter in the vicinity of column P10. Though not strictly a geometry concern defined in section A2.2.5, this horizontal irregularity is concerning, especially with its proximity to the corner where the two perpendicular halls of the MSB building meet.
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. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5) **Comments:** No changes in effective mass over 50%.
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. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6) **Comments:** Distance between center of mass and center of rigidity does not exceed limit. However, significant torsional response is observed in modal analysis of the three-dimensional model.

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

GEOLOGIC SITE HAZARD

C NC N/A U	Description
☒ 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 (15.2m) under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1) **Comments:** Low likelihood according to the geotechnical study by Egan (2019).
☒ 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. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1) **Comments:** Site is located on the slope of Mt. Sutro in San Francisco, and building is on spread (isolated) footings. A 2006 study of slope stability for the entire campus suggested some level of concern. However the geological study conducted by Egan (2019) indicates slope failure is not a relevant hazard at this site.
☒ 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:** No active faults through the site according to the geotechnical study by Egan (2019).

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:**
☒ 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:** Perimeter Spread Footings appear to have grade beams.

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Collapse Prevention Structural Checklist For Building Type C2

LOW AND MODERATE SEISMICITY

SEISMIC-FORCE-RESISTING SYSTEM

C NC N/A U	Description
C ☒ NC ☐ N/A ☐ U ☐	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:** Without evaluating capacity of connections, secondary components appear to form a complete 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. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1) **Comments:** More than 2 lines are present in both principal directions.
C ☐ NC ☒ N/A ☐ U ☐	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 or 2√f'c. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1) **Comments:** Shear stress in concrete shear walls exceeds the limits of Section 4.4.3.3 for both orthogonal directions. The shear capacity is approximately 100 psi, compared to estimated shear stresses at first floor of 176 psi and 248 psi in E-W and N-S directions, respectively.
C ☒ NC ☐ N/A ☐ U ☐	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:** All reinforcing ratios exceed the minimum limits. The minimum reinforcing ratio is 0.0015 for a 14-inch wall; for other wall thicknesses the ratio is greater than 0.002.

CONNECTIONS

C NC N/A U	Description
C ☐ NC ☐ N/A ☒ U ☐	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:** Diaphragms not flexible.

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C ☒ **NC** ☐ **N/A** ☐ **U** ☐	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:** Shear walls and slabs appear to be connected, though the strength of these connections has not been determined. Wall and slab reinforcing are shown exclusive of one another on the plan set.
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:** Dowels are present and match size and spacing of vertical bars above.

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:** In this case, “secondary components” refers to gravity columns, which for this structure are concrete-encased steel elements. Steel elements do not have a brittle shear mechanism as concrete columns do, and as such this provision does not apply provided that the steel elements have the capacity to support the required gravity load in the absence of the concrete encasement. Considering a typical interior column such as the one at grid J/2, the LRFD factored gravity load at foundation is approximately 1100 kips, which results in a 20ksi axial stress for the given W14x193 shape. If there were no concrete, the steel section would have a slenderness ratio of approximately 40, which is stocky. By inspection, the steel section alone should have the capacity to carry all gravity load and can tolerate the anticipated 1% building drift without brittle failure.
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:** The typical flat slab appears to have continuous bottom steel.
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:** Wall segments exist which initially will act as coupling beams, however none is detailed as required for the ductility demands of coupling beams. As such, the coupling beams may not be able to deliver large coupling forces to the shear walls. Nevertheless, coupled walls have limited vertical reinforcement and do not appear to be detailed to carry large tension forces at both ends.

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DIAPHRAGMS (STIFF OR FLEXIBLE)	
	Description
C (selected) 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:** Diaphragms appear to have a uniform elevation for each level of the structure.
C (selected) 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:** No diaphragm openings immediately adjacent to both sides of the shear walls.

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Collapse Prevention Structural Checklist For Building Type S2

LOW SEISMICITY

SEISMIC-FORCE-RESISTING SYSTEM

	Description
C ☒ NC ☐ N/A ☐ U ☐	REDUNDANCY: The number of lines of braced frames in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.3.1.1. Tier 2: Sec. 5.5.1.1) **Comments:** More than 2 lines in each direction.
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 0.10Fy. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than 0.30Fy. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3) **Comments:** Using the quick check method, overturning stresses in the steel columns are estimated as 38 ksi and 21 ksi in the East-West and North-South directions, respectively. Given the specified yield stress of 33 ksi for column steel, the overturning stresses are expected to significantly exceed the 0.3Fy criterion (10 ksi).
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 0.50Fy. (Commentary: Sec. A.3.3.1.2. Tier 2: Sec. 5.5.4.1) **Comments:** The quick check method was used to check this parameter at first and fifth floors. At fifth floor, the average brace stress was estimated as 24 ksi and 14 ksi in East-West and North-South directions, respectively. At first floor, the corresponding stresses are 26 ksi and 12 ksi. For 33 ksi specified yield stress, 0.50Fy = 16.5 ksi. Therefore, the East-West direction is found non-conforming at both levels and the North-South direction is found conforming at both levels.

CONNECTIONS

	Description
C ☐ NC ☐ N/A ☐ U ☒	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2) **Comments:** Slabs are reinforced concrete, and are directly connected to the steel framing, which is completely encased in concrete. Connection between the slab and steel framing is unknown.
C ☒ NC ☐ N/A ☐ U ☐	STEEL COLUMNS: The columns in the seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1) **Comments:** Columns are anchored into footings.

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

SEISMIC FORCE-RESISTING SYSTEM

C NC N/A U	Description
C ● NC ○ N/A ○ U ○	REDUNDANCY: The number of braced bays in each line is greater than or equal to 2. (Commentary: Sec. A.3.3.1.1. Tier 2: Sec. 5.5.1.1) **Comments:** Some lines only have 1 braced bay. However, redundancy exists when taking into account the presence of shear walls and lateral frames in bays located on adjacent gridlines.
C ○ NC ● N/A ○ U ○	CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4) **Comments:** Connections fail before certain braces develop buckling capacity.
C ● NC ○ N/A ○ U ○	COMPACT MEMBERS: All brace elements meet compact section requirements in accordance with AISC 360, Table B4.1. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4) **Comments:** Braces checked were compact.
C ○ NC ● N/A ○ U ○	K-BRACING: The bracing system does not include K-braced bays. (Commentary: Sec. A.3.3.2.1. Tier 2: Sec. 5.5.4.6) **Comments:** K braces present in increment 1 for "Br. 6".

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 ○ NC ● N/A ○ U ○	COLUMN SPLICES: All column splice details located in braced frames develop 50% of the tensile strength of the column. (Commentary: Sec. A.3.3.1.3. Tier 2: Sec. 5.5.4.2) **Comments:** Bolts in the column splices cannot develop 50% of the tensile strength of some of the largest columns found in braced frames.

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C NC N/A U	
NC	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have Kl/r ratios less than 200. (Commentary: Sec. A.3.3.1.4. Tier 2: Sec. 5.5.4.3) **Comments:** Some braces are slender (KL/r >200), as shown in the connection strength calcs and in the slenderness calc.
NC	CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4) **Comments:** Failed Moderate seismicity check earlier in list.
C	COMPACT MEMBERS: All brace elements meet section requirements in accordance with AISC 341, Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4) **Comments:** Members meet requirement of Table D1.1 with Ry=1.1.
NC	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. (Commentary: Sec. A.3.3.2.3. Tier 2: Sec. 5.5.4.8) **Comments:** Assuming tension brace can reach yield point, the induced moment due to the vertical point load at the midspan of the beam would exceed the beam's capacity
NC	CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces frame into the beam-column joints concentrically. (Commentary: Sec. A.3.3.2.4. Tier 2: Sec. 5.5.4.8) **Comments:** Does not appear that braces and beams are concentric at joints per steel framing details for increment 1.

DIAPHRAGMS (STIFF OR FLEXIBLE)

C NC N/A U	Description
C	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 25% of the frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3) **Comments:** Large opening in the 2nd floor in increment 2 is near a braced frame (Grid P between grids 12 and 13), but 1 bay away, so not immediately adjacent.

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

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APPENDIX C

UCOP Seismic Safety Policy Falling Hazards Assessment Summary

UC Campus:	**San Francisco Parnassus**		Date:	**10/25/2019**		
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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:** Large auditorium has gyp board ceiling which could fall a significant distance, but is not considered a heavy ceiling. Lightweight metal panels in lobby areas by front entrance are also not heavy enough to trigger this warning.
P ☒ **N/A** ☐	**Heavy masonry or stone veneer above exit ways or public access areas** **Comments:** No heavy masonry or stone veneer directly above exit ways. Tile veneer on building façade falling from great heights could cause harm to people directly outside of the building.
P ☐ **N/A** ☒	**Unbraced masonry parapets, cornices, or other ornamentation above exit ways or public access areas** **Comments:** There are no masonry parapets, cornices, or other ornamentation.
P ☒ **N/A** ☐	**Unrestrained hazardous material storage** **Comments:** Restraints on hazardous materials throughout the building were very hit-or-miss. Some were restrained, some were not. Some were restrained but insufficiently.
P ☐ **N/A** ☒	**Masonry chimneys** **Comments:** No masonry chimneys are in the building.
P ☐ **N/A** ☒	**Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.** **Comments:** Unrestrained natural gas-fueled equipment was not identified during site visit.
P ☐ **N/A** ☐	**Other:** **Comments:**
P ☐ **N/A** ☐	**Other:** **Comments:**
P ☐ **N/A** ☐	**Other:** **Comments:**

Falling Hazards Risk: *Moderate*

Note: P= Present, N/A = Not Applicable; Falling Hazards Risk: Low, Moderate, or High

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APPENDIX D

Quick Check Calculations

ARUP	Job No. 567774-00	Sheet No.	Rev.
	Member/Location		
Job Title: USCF Parnassus MSB Tier 1 Evaluation	Drg. Ref.		
Calculation: Building Weight Calcs	Made by: ML	Date: 4/17/19	Chd.: BT

BUILDING WEIGHT

*****THESE CALCULATIONS INCLUDE THE ADDITION OF THE FULL-HEIGHT ADD-ON STRUCTURE TO THE TRUE SOUTHERN EDGE OF THE ORIGINAL MSB BUILDING (~40'x30')**

The seismic weight of this building is a sum of the above grade floor weights (including superimposed dead load), steel framing, concrete wall weights, column weights, and tile cladding weights. 5/15 update: mass of the added structure was refined. This change is not reflected in the Teir 1 report.

Input

Reinf Conc. Density = 105 [pcf]
Steel Density = 490 [pcf]

Flooring	Elevation [ft]	Avg Story Height (below) [ft]	Floor Area - Incr 1 [sf]	Floor SW [psf]	Floor Area - Incr 2 [sf]	Floor SW [psf]	Floor SDL [psf]	Column Weight (below) [psf]	Floor Weight [kips]
High Roof	203.1	9.8	1284	69	1700	69	37.5	5	332
Mach Room Floor	193.3	10.8	1625	69	2033	69	20	5	343
Inc 2 Roof	182.5	13.0			12411	91	42.5	5	1714
PH Floor (Inc 1)	182.5	13.0	4850	69			30	5	503
14th Floor (Inc 2)	169.5	13.0			12411	82	20	5	1326
Inc 1 Roof	169.5	13.0	13126	78			42.5	5	1641
13th Floor	156.5	13.0	13126	78	12411	82	20	5	2672
12th Floor	143.5	13.0	13126	78	12411	82	20	5	2672
11th Floor	130.5	13.0	13126	78	12411	82	20	5	2672
10th Floor	117.5	13.0	13126	78	12411	82	20	5	2672
9th Floor	104.5	13.0	13126	78	12411	82	20	5	2672
8th Floor	91.5	13.0	13126	78	12411	82	20	5	2672
7th Floor	78.5	13.0	13126	78	12411	82	20	5	2672
6th Floor	65.5	13.0	13126	78	12411	82	20	5	2672
5th Floor	52.5	13.0	13126	78	12411	82	20	5	2672
4th Floor	39.5	13.0	13126	78	17828	82	20	5	3251
3rd Floor	26.5	13.0	13126	78	17828	82	20	5	3251
2nd Floor	13.5	13.5	13126	78	17064	82	20	5	3169
Incr. 2 1st Floor	0	7.0			17828	82	20	5	1905
Incr. 1 1st Floor	0	13.0	13126	78			20	5	1345
Incr. 2 Basement	-7	0			17828				
Incr. 1 Basement	-13	0	13441						

Steel Framing	Inc 1 Length of Framing [ft]	Inc 1 Avg Section Weight [lb/ft]	Inc 2 Length of Framing [ft]	Inc 2 Avg Section Weight [lb/ft]	Total Steel Framing Weight [kips]
High Roof	262	35	0	0	9
Mach Room Floor	320	75	210.5	55	36
Inc 2 Roof			1711	40	68
PH Floor (Inc 1)	721	40			29
14th Floor (Inc 2)			1711	45	77
Inc 1 Roof	2009	40			80
13th Floor	2009	45	1711	45	167
12th Floor	2009	45	1711	45	167
11th Floor	2009	45	1711	45	167
10th Floor	2009	45	1711	45	167
9th Floor	2009	45	1711	45	167
8th Floor	2009	50	1711	45	177
7th Floor	2009	50	1711	45	177
6th Floor	2009	55	1711	45	187
5th Floor	2009	55	1711	45	187
4th Floor	2009	55	2634	50	242
3rd Floor	2009	55	2432	55	244
2nd Floor	2009	55	2634	55	255
Incr. 2 1st Floor			2634	55	145
Incr. 1 1st Floor	2009	55			110

Narrative for mass of added structure:

-The previous size of the added structure was overestimated to be 30ftx40ft, rather than 30ftx30ft. This discrepancy represented an 32kips of weight. However the additional wall and column weight was neglected, which represent 76kips of weight. The net difference is 44kips. 44kips/2672kips = 1.7%, which represents a small enough amount to neglect for the purposes of this study.

<u>Do not updated mass and mass moment of inertia in model from original mass take-off.</u>

Interior Walls	Story Height (below) [ft]	Incr 1 RC wall length [ft]	Incr 1 RC wall thick [ft]	Incr 2 RC wall length [ft]	Incr 2 RC wall thick [ft]	1/2 Wall Volume [ft^3]	Conc 1/2 Wall Weight [kips]	Total Wall Weight [kips]
High Roof	9.8	0	0.00	0	0.00	0	0	0
Mach Room Floor	10.8	0	0.00	27	0.75	109	11	11
Inc 2 Roof	13.0			138	0.75	672	71	77
PH Floor (Inc 1)	13.0	139	0.67			601	63	68
14th Floor (Inc 2)	13.0			42	0.67	181	19	90
Inc 1 Roof	13.0	157	1.00			1018	107	170
13th Floor	13.0	157	1.00	42	0.67	1199	126	252
12th Floor	13.0	157	1.00	42	0.67	1199	126	252
11th Floor	13.0	157	1.00	42	0.67	1199	126	252
10th Floor	13.0	157	1.00	42	0.67	1199	126	252
9th Floor	13.0	157	1.00	42	0.67	1199	126	252

ARUP	Job No.	Sheet No.	Rev.
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Job Title: USCF Parnassus MSB Tier 1 Evaluation	Drg. Ref.		
Calculation: Building Weight Calcs	Made by: ML	Date: 4/17/19	Chd.: BT

8th Floor	13.0	157	1.00	42	0.67	1199	126	252
7th Floor	13.0	157	1.00	42	0.67	1199	126	252
6th Floor	13.0	157	1.00	42	0.67	1199	126	252
5th Floor	13.0	157	1.00	42	0.92	1267	133	259

ARUP	Job No. 567774-00	Sheet No.	Rev.
Job Title: USCF Parnassus MSB Tier 1 Evaluation	Member/Location		
Calculation: Building Weight Calcs	Drg. Ref.		
	Made by ML	Date 4/17/19	Chd. BT

4th Floor	13.0	157	1.00	97	1.00	1651	173	306
3rd Floor	13.0	168	1.00	97	1.00	1722	181	354
2nd Floor	13.5	168	1.00	97	1.00	1788	188	369
Incr. 2 1st Floor	0.0			97	1.00	0	0	188
Incr. 1 1st Floor	0.0	168	1.08			0	0	188

Perimeter Walls	Story Height (below) [ft]	Incr 1 RC wall length [ft]	Incr 1 RC wall thick [ft]	Incr 2 RC wall length [ft]	Incr 2 RC wall thick [ft]	1/2 Wall Volume [ft^3]	Conc 1/2 Wall Weight [kips]	Total Wall Weight [kips]
High Roof	9.8	135	0.79	91	0.79	881	93	93
Mach Room Floor	10.8	135	0.79	91	0.79	963	101	194
Inc 2 Roof	13.0			100	0.88	571	60	161
PH Floor (Inc 1)	13.0	0	0.79			0	0	0
14th Floor (Inc 2)	13.0			100	1.04	679	71	131
Inc 1 Roof	13.0	146	0.88			830	87	87
13th Floor	13.0	146	0.88	100	1.04	1510	159	317
12th Floor	13.0	146	0.88	100	1.04	1510	159	317
11th Floor	13.0	146	0.88	100	1.04	1510	159	317
10th Floor	13.0	146	0.88	100	1.04	1510	159	317
9th Floor	13.0	146	0.88	100	1.04	1510	159	317
8th Floor	13.0	146	0.88	100	1.04	1510	159	317
7th Floor	13.0	146	1.04	100	1.04	1668	175	334
6th Floor	13.0	146	1.04	100	1.04	1668	175	350
5th Floor	13.0	146	1.04	100	1.04	1668	175	350
4th Floor	13.0	146	1.04	259	1.04	2742	288	463
3rd Floor	13.0	171	1.13	447	1.04	4277	449	737
2nd Floor	13.5	171	1.13	447	1.04	4441	466	915
Incr. 2 1st Floor	0.0					0	0	466
Incr. 1 1st Floor	0.0					0	0	466

Perimeter Spandrel Walls	Avg. Spandrel Height [ft]	Incr 1 RC wall length [ft]	Incr 1 RC wall thick [psf]	Incr 2 RC wall length [ft]	Incr 2 RC wall thick [psf]	1/2 Wall Volume [kips]	Conc 1/2 Wall Weight [kips]	Spandrel Weight [kips]
High Roof	6.7	0	0.00	0	0.00	0	0	0
Mach Room Floor	6.7	0	0.00	0	0.00	0	0	0
Inc 2 Roof	6.7			298	0.88	871	91	91
PH Floor (Inc 1)	6.7	334	0.79			882	93	93
14th Floor (Inc 2)	6.7			298	1.04	1036	109	200
Inc 1 Roof	6.7	334	0.88			975	102	195
13th Floor	6.7	334	0.88	298	1.04	2011	211	422
12th Floor	6.7	334	0.88	298	1.04	2011	211	422
11th Floor	6.7	334	0.88	298	1.04	2011	211	422
10th Floor	6.7	334	0.88	298	1.04	2011	211	422
9th Floor	6.7	334	0.88	298	1.04	2011	211	422
8th Floor	6.7	334	0.88	298	1.04	2011	211	422
7th Floor	6.7	334	1.04	298	1.04	2197	231	442
6th Floor	6.7	334	1.04	298	1.04	2197	231	461
5th Floor	6.7	334	1.04	298	1.04	2197	231	461
4th Floor	6.7	334	1.04	245	1.04	2011	211	442
3rd Floor	6.7	334	1.13	0	0.00	1253	132	343
2nd Floor	6.7	328	1.13	0	0.00	1231	129	261
Incr. 2 1st Floor	0.0			0	0.00	0	0	129
Incr. 1 1st Floor	0.0	0	0.00			0	0	129

TOTAL WEIGHT	Total Weight [kips]	Total Floor Area [SF]
High Roof	434	2983
Mach Room Floor	584	3657
Inc 2 Roof	2112	12411
PH Floor (Inc 1)	693	4850
14th Floor (Inc 2)	1825	12411
Inc 1 Roof	2173	13126
13th Floor	3830	25537
12th Floor	3830	25537
11th Floor	3830	25537
10th Floor	3830	25537
9th Floor	3830	25537
8th Floor	3840	25537
7th Floor	3877	25537
6th Floor	3923	25537
5th Floor	3930	25537
4th Floor	4704	30954
3rd Floor	4929	30954
2nd Floor	4969	30190
Incr. 2 1st Floor	2833	17828
Incr. 1 1st Floor	2239	13126
TOTAL	**62216**	

	Story Weight [kips]	Story Weight [kips/g]	Story Area [SF]	Smeared D + SDL Load [PSF]
Roof+	3822	118.7	23902	160
14th Floor	3998	124.2	25537	157
13th Floor	3830	119.0	25537	150
12th Floor	3830	119.0	25537	150
11th Floor	3830	119.0	25537	150
10th Floor	3830	119.0	25537	150
9th Floor	3830	119.0	25537	150
8th Floor	3840	119.3	25537	150
7th Floor	3877	120.4	25537	152
6th Floor	3923	121.8	25537	154
5th Floor	3930	122.0	25537	154
4th Floor	4704	146.1	30954	152
3rd Floor	4929	153.1	30954	159
2nd Floor	4969	154.3	30190	165
1st Floor	5073	157.5	30954	164
TOTAL	**62216**	**1932.2**		

Floor Load Type Description: Typical floor at Increment 1
Applies at locations: Increment 1, levels 1-13

Description	Unit Loads (psf)	
	SW	**SDL**
Structural slab: Mark S-4, 4.5" LWC	39.4	
Secondary beams: Mark B-8, 10"x13" LWC @ 9.3' spacing	10.2	
Concrete cover over primary beams: 2" LWC all around W24x84 @ 18' spacing	12.7	
Floor finish: linoleum		2
Ceiling: suspended acoustic tiles		5
MEP suspended components		10
Miscellaneous allowance	15.7	3
Totals	**78**	**20**

Floor Load Type Description: Typical floor at Increment 2
Applies at locations: Increment 2, levels 1-14

Description	Unit Loads (psf)	
	SW	**SDL**
Structural slab: Mark S-1, 5" LWC	43.8	
Concrete cover over secondary beams: 2" LWC all around W14x30 @ 9.7' spacing	12.1	
Concrete cover over primary beams: 2" LWC all around W24x94 @ 18.5' spacing	12.4	
Floor finish: linoleum		2
Ceiling: suspended acoustic tiles		5
MEP suspended components		10
Miscellaneous allowance	13.8	3
Totals	**82**	**20**

Floor Load Type Description: Increment 1 Roof
Applies at locations: Increment 1 Roof

Description	Unit Loads (psf)	
	SW	**SDL**
Structural slab: Mark RS-1, 4" LWC	35.0	
Secondary beams: Mark RB-1, 12"x13" LWC @ 9.3' spacing	12.2	
Concrete cover over primary beams: 2" LWC all around W18x55 @ 18' spacing	8.7	
Rock balast		7
2" pavers		10
Miscellaneous rooftop equipment		5
Roof insulation and waterproofing		3
Ceiling: suspended acoustic tiles		5
MEP suspended components		10
Miscellaneous allowance	21.8	2.5
Totals	**78**	**42.5**

Floor Load Type Description: Increment 2 Roof
Applies at locations: Increment 2 Roof

Description	Unit Loads (psf)	
	SW	**SDL**
Structural slab: Mark S-10, 6.5" LWC	56.9	
Concrete cover over secondary beams: 2" LWC all around W10x22 @ 9.7' spacing	8.5	
Concrete cover over primary beams: 2" LWC all around W21x62 @ 18.5' spacing	10.3	
Rock balast		7
2" pavers		10
Miscellaneous rooftop equipment		5
Roof insulation and waterproofing		3
Ceiling: suspended acoustic tiles		5
MEP suspended components		10
Miscellaneous allowance	14.8	2.5
Totals	**91**	**42.5**

Floor Load Type Description: Penthouse floor, Increment 1
Applies at locations: Increment 1, elevation 182.5'

Description	Unit Loads (psf)	
	SW	**SDL**
Structural slab: Mark RS-1, 4" LWC	35.0	
Secondary beams: Mark RB-1, 12"x13" LWC @ 9' spacing	12.6	
Concrete cover over primary beams: 2" LWC all around W14x34 @ 18' spacing	6.6	
Miscellaneous equipment		10
Ceiling: suspended acoustic tiles		5
MEP suspended components		10
Miscellaneous allowance	14.8	5
Totals	**69**	**30**

Floor Load Type Description: Machine room floor
Applies at locations: Increment 1 and 2, elevation 193.3'

Description	Unit Loads (psf)	
	SW	**SDL**
Structural slab: Increment 1 Mark S-2, 4.5" LWC	39.4	
Concrete cover over secondary beams: 2" LWC all around W16x45 @ 9' spacing	12.6	
Concrete cover over primary beams: 2" LWC all around W18x50 @ 11' spacing	14.3	
Miscellaneous equipment		6
Ceiling: suspended acoustic tiles		5
MEP suspended components		6
Miscellaneous allowance	3	3
Totals	**69**	**20**

Floor Load Type Description: High roof
Applies at locations: Increment 1 and 2, elevation 203.1'

Description	Unit Loads (psf)	
	SW	**SDL**
Structural slab: Mark RS-1, 4" LWC	35.0	
Concrete cover over secondary beams: 2" LWC all around W14x34 @ 9' spacing	13.2	
Concrete cover over primary beams: 2" LWC all around W16x36 @ 21.7' spacing	6.3	
Rock balast		7
2" pavers		10
Roof insulation and waterproofing		3
MEP suspended components		15
Miscellaneous allowance	14.8	2.5
Totals	**69**	**37.5**

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Calculation: Seismic Hazard Calcs	Made by ML	Date 4/17/19	Chd. BT

SEISMIC HAZARD

Seismicity

		X - Direction	Y - Direction		
S_S	=	1.563	1.563	[g]	from SEAOC Seismic Design Map Tool
S_1	=	0.632	0.632	[g]	
Soil Class	=	D	D		
F_a	=	1	1		[ASCE 7-16 Table 11.4-1]
F_v	=	1.7	1.7		[ASCE 7-16 Table 11.4-2]
S_{MS}	=	1.563	1.563	[g]	[ASCE 7-16 Eqn 11.4-1]
S_{M1}	=	1.074	1.074	[g]	[ASCE 7-16 Eqn 11.4-2]
S_{DS}	=	1.042	1.042	[g]	[ASCE 7-16 Eqn 11.4-3]
S_{D1}	=	0.716	0.716	[g]	[ASCE 7-16 Eqn 11.4-4]
(BSE-2N) S_{XS}	=	1.042	1.042	[g]	[ASCE 41-17 Sec. 2.4.1.1]
(BSE-2N) S_{X1}	=	0.716	0.716	[g]	[ASCE 41-17 Sec. 2.4.1.1]
S_S	=	1.819	1.819	[g]	From 5%/50-year maximum direction spectral response
S_1	=	1.132	1.132	[g]	acceleration [ASCE 41-17 Sec. 2.4.1.3]
(BSE-2E) S_{XS}	=	1.042	1.042	[g]	
(BSE-2E) S_{X1}	=	0.716	0.716	[g]	
Level of Seismicity	=	High	High		

Fundamental Period

		X	Y	
C_t	=	0.02	0.02	
h_n	=	182	182	[ft]
β	=	0.75	0.75	
$T_{approx} = C_t h_n^{\beta}$	=	0.991	0.991	[sec]
Given T	=	none	none	[sec]
T	**=**	**0.991**	**0.991**	**[sec]**

Seismic Hazard Level specified in Section 4.1.2. Alternatively, a site-specific response spectrum shall be permitted to be developed according to Section 2.4.2 for the Seismic Hazard Level specified in Section 4.1.2.

Spectral Acceleration

		X	Y	
S_{X1}	=	0.716	0.716	[g]
S_{XS}	=	1.042	1.042	[g]
S_{X1} / T	=	0.723	0.723	[g]
$\mathbf{S_a}$	**=**	**0.723**	**0.723**	**[g]**

Modification Factor, C

		X	Y
Building Type	=	C2, S2	C2, S2
Number of Stories	=	14	14
C	**=**	**1.0**	**1.0**

Pseudo Seismic Force, V

		X	Y	
W	=	62216	62216	kips
V	**=**	**44967**	**44967**	kips

4.4.2.1 Pseudo Seismic Force. The pseudo seismic force, in a given horizontal direction of a building, shall be calculated in accordance with Eq. (4-1).

$$V = CS_aW \qquad (4\text{-}1)$$

Story Shear Forces

k = 1

Level	w [kips]	h [ft]	wh^k [kip-ft]	F_x [kips]	V_j [kips]
Roof+	3822	13	49685	2754	2754
14th Floor	3998	13	51970	2880	5634
13th Floor	3830	13	49796	2760	8394
12th Floor	3830	13	49796	2760	11154
11th Floor	3830	13	49796	2760	13914
10th Floor	3830	13	49796	2760	16674
9th Floor	3830	13	49796	2760	19434
8th Floor	3840	13	49926	2767	22201
7th Floor	3877	13	50396	2793	24995
6th Floor	3923	13	50995	2826	27821
5th Floor	3930	13	51088	2832	30653

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Calculation: Seismic Hazard Calcs	Made by: ML	Date: 4/17/19	Chd.: BT

4th Floor	4704	13	61153	3390	34042
3rd Floor	4929	13	64074	3551	37594
2nd Floor	4969	13.5	67084	3718	41312
1st Floor	5073	13	65944	3655	44967
			811293	44967	

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Calculation: Basic Configuration Calcs	Made by: ML	Date: 4/17/19	Chd.: BT

Mass

C	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)

	Total Story Weight	$(W_{i+1} - W_i) / W_i$	Check
	[kip]		
Roof+	3822	n/a	n/a
14th Floor	3998	4%	**Conforming**
13th Floor	3830	4%	**Conforming**
12th Floor	3830	0%	**Conforming**
11th Floor	3830	0%	**Conforming**
10th Floor	3830	0%	**Conforming**
9th Floor	3830	0%	**Conforming**
8th Floor	3840	0%	**Conforming**
7th Floor	3877	1%	**Conforming**
6th Floor	3923	1%	**Conforming**
5th Floor	3930	0%	**Conforming**
4th Floor	4704	16%	**Conforming**
3rd Floor	4929	5%	**Conforming**
2nd Floor	4969	1%	**Conforming**

Torsion

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)

Center of Mass

Building	X,com	Y,com
	[ft]	[ft]
Roof	121.2	-114.1
13th Floor	121.2	-114.1
12th Floor	121.2	-114.1
11th Floor	121.2	-114.1
10th Floor	121.2	-114.1
9th Floor	121.2	-114.1
8th Floor	121.2	-114.1
7th Floor	121.2	-114.1
6th Floor	121.2	-114.1
5th Floor	121.2	-114.1
4th Floor	116.7	-123.7
3rd Floor	116.7	-123.7
2nd Floor	116.7	-123.7
1st Floor	116.7	-123.7

Used Bluebeam to calculate approximate center of area

Center of Masses.pdf

*Assume braced frames and concrete shear walls all have relative stiffnesses to one another as shown in the table below.

Center of Rigidity

Level 4

Element	Length	Orientation	Dist From Orig.
	[ft]		[ft]
B1	16	X	8
B2	21	Y	19
B3	19	Y	50
B4	16	X	8
B5	16	X	165.5
B6	14	Y	66.25
B7	21	X	162
B8	10	Y	94.5
B9	10	Y	94.5

Stiffness Approximations:			% Total
k(typ brace)	3603	kip/in	33%
k(typ wall)	7299	kip/in	67%
Relative Stiffness Ratio of wall to brace:			2.03

				Length*Relative Stiffness Ratio
B10	18.5	X	134	
B11	10	Y	116	
B12	21	X	162	
B13	10	Y	116	
B14	19	Y	259	
B15	20	X	124.75	
B16	16	X	152.25	
B17	18	Y	259	
B18	17	X	152.25	
B19	18	Y	150	
B20	18	Y	242	
B21	18	Y	187	
B22	18	Y	224	
W1	16	X	8	32.4
W2	21	Y	19	42.5
W3	19	Y	50	38.5
W4	16	X	8	32.4
W5	16	X	165.5	32.4
W6	14	Y	66.25	28.4
W6.5	21	X	162	42.5
W7	21	X	162	42.5
W8	10	Y	94.5	20.3
W8.5	21	X	162	42.5
W9	10	Y	94.5	20.3
W10	18.5	X	134	37.5
W11	10	Y	116	20.3
W12	30	X	167	60.8
W13	10	Y	116	20.3
W14	19	Y	259	38.5
W15	20	X	124.75	40.5
W16	30	X	166	60.8
W16.5	14	X	187	28.4
W17	10	Y	254	20.3
W18	43.5	X	178	88.1
W18.5	29	Y	283	58.8
W21	18	Y	187	36.5
W22	18	Y	224	36.5

	X,cor	Y,cor
	[ft]	[ft]
Combined (1st-4th)	**139**	**155**
Combined (5th-roof)	**139**	**139**

Torsion Check

Direction	Bldg Width	20% width	Δ	Result
	[ft]	[ft]	[ft]	
X (1st-4th)	174	35	22.2	**Conforming**
X (5th-Roof)	174	35	17.7	**Conforming**
Y (1st-4th)	300	60	31.1	**Conforming**
Y (5th-Roof)	300	60	24.8	**Conforming**

Overturning

C	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 0.6Sa. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

Least Dim	Height	Ratio	0.6Sa	Check
[ft]	[ft]			
174	195	0.89	0.43	**Conforming**

ARUP	Job No. 567774-00	Sheet No.	Rev.
	Member/Location		
Job Title: USCF Parnassus MSB Tier 1 Evaluation	Drg. Ref.		
Calculation: Type C2(a) Calcs	Made by: ML	te: 4/17/19	d.: BT

Reinforcing Steel

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)

Wall thickness [in]	Horiz. Reinforcing Ratio	Vert. Reinforcing Ratio
6	0.0028	0.0028
8	0.0028	0.0028
10	0.0025	0.0025
12	0.0028	0.0028
14	0.0046	0.0046

TYPICAL WALL REINFORCING

Wall Thickness	No. of Curtains	Reinf. ea. Curtain Horiz.	Reinf. ea. Curtain Vert.	Trim Bars ea. side Openings No. & Size	Trim Bars ea. side Openings Extension E	Trim Bars ea. side Openings Diagonal
6 & 7	1	#4@12	#4@12	1-#6	3'-0"	1-#4
8 & 9	2	#4@18	#4@18	2-#5	3'-0"	1-#4
10 & 11	2	#4@16	#4@16	2-#6	3'-0"	1-#5
12 & 13	2	#4@12	#4@12	2-#7	4'-0"	1-#5
14 & over	2	#5@15	#5@15	2-#8	4'-0"	1-#5

Single curtains in center of wall, double curtains 1½" clear each face. Stagger bars of double-curtain walls under 12" thick. Additional reinforcing each side of wall openings as scheduled above and detailed below.

Shear Stress Check

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 or 2√f'c. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)

Wall		Length	Thickness
W1	X	16	12
W2	Y	21	12
W3	Y	19	14
W4	X	16	14
W5	X	16	12
W6	Y	14	8
W6.5	X	21	10
W7	X	21	12
W8	Y	10	12
W8.5	X	21	8
W9	Y	10	12
W10	X	18.5	14
W11	Y	10	12
W12	X	30	14
W13	Y	10	12
W14	Y	19	12
W15	X	20	8
W16	X	30	11
W16.5	X	14	11
W17	Y	10	11
W18	X	43.5	14
W18.5	Y	29	14
W21	Y	18	11

4.4.3.3 Shear Stress in Shear Walls. The average shear stress in shear walls, v_j^{avg}, shall be calculated in accordance with Eq. (4-8).

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \tag{4-8}$$

where

V_j = Story shear at level j computed in accordance with Section 4.4.2.2;

A_w = Summation of the horizontal cross-sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration where computing A_w. For masonry walls, the net area shall be used. For wood-framed walls, the length shall be used rather than the area; and

M_s = System modification factor; M_s shall be taken from Table 4-8.

Table 4-8. M_s Factors for Shear Walls

Wall Type	Level of Performance 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.

W22	Y	18	11

Level 1, X Direction

$V_{controlling}$	=	30107	[kip]
Aw	=	264.2	[ft^2]
M_s	=	4.5	
v_{L1x}^{avg}	=	**175.9**	[psi]
f'c	=	2500	[psi]
2√f'c	=	**100.0**	[psi]
Check	=	**Nonconforming**	

Level 1, Y Direction

$V_{controlling}$	=	30107	[kip]
Aw	=	187.5	[ft^2]
M_s	=	4.5	
v_{L1x}^{avg}	=	**247.8**	[psi]
f'c	=	2500	[psi]
2√f'c	=	**100.0**	[psi]
Check	=	**Nonconforming**	

Wall Anchorage out-of-plane

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)

Development Length (per ACI 318-14 Ch 25)

Bar no.	=	4	
f'c	=	2500	psi
fy	=	33	ksi
db	=	0.5	in
ld	**=**	**19.8**	in
		Conforming for L1=8'	

--> Compare with (1/4)*L1 as shown in typ detail (L1=8')

$ldh_{(a)}$	=	6.6	in
$ldh_{(b)}$	=	4	in
$ldh_{(c)}$	=	6	in
ldh	**=**	**6.6**	in

--> Compare with (wall thickness) - (1.5") for hooked end of bars, as shown in typ detail

Nonconforming for 8" wall with 1.5" cover
Conforming for 10"+ Walls

4.4.3.7 Flexible Diaphragm Connection Forces. The horizontal seismic forces associated with the connection of a flexible diaphragm to either concrete or masonry walls, T_c, shall be calculated in accordance with Eq. (4-12).

$$T_c = \psi S_{XS} w_p A_p \tag{4-12}$$

where

w_p = Unit weight of the wall;
A_p = Area of wall tributary to the connection;
ψ = 1.0 for Collapse Prevention Performance Level, 1.3 for Life Safety Performance Level, and 1.8 for Immediate Occupancy Performance Level; and
S_{XS} = Value specified in Section 4.4.2.3.

Testing Shear Wall Labeled W10

S_{XS}	=	1.042	g	
wp	=	1950	psf	(Assumes 13' height)
ψ	=	1		
Ap	=	21.58	ft^2	
Tc	=	43.9	kips	

Bar No.	=	4	
Bar Spacing	=	8	in
As_{bar}	=	0.2	in^2
As	=	5.55	in^2
fy	=	33	ksi
Tn	=	183.2	kips
Check	=	**Conforming**	

Column Axial Stress Check

NC	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than 0.10 Fy. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than 0.30 Fy. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3)

Check Axial Stress Caused by Overturning

Load Direction		X	Y	
n_f	=	9	13	
V_{tot}	=	44967	44967	[kip]
h_n	=	182	182	[ft]
L	=	158	203.0	[ft]
M_s	=	2.5	2.5	
Column	=	14WF136	14WF136	
A_{col}	=	40.00	40.00	[in^2]
p_{ot}	=	38.4	20.7	[ksi]
F_y	=	33	33	[ksi]
$0.3F_y$	=	9.9	9.9	[ksi]
Check	=	**Nonconforming**	**Nonconforming**	

4.4.3.6 Column Axial Stress Caused by Overturning. The axial stress of columns in moment frames at the base subjected to overturning forces, p_{ot}, shall be calculated in accordance with Eq. (4-11).

$$p_{ot} = \frac{1}{M_s}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right)\left(\frac{1}{A_{col}}\right) \quad (4\text{-}11)$$

where

n_f = Total number of frames in the direction of loading;
V = Pseudo seismic force;
h_n = Height (ft) above the base to the roof level;
L = Total length of the frame (ft);
M_s = System modification factor taken as equal to 2.5 for buildings being evaluated to the Collapse Prevention Performance Level, equal to 1.5 for buildings being evaluated to the Life Safety Performance Level, and equal to 1.0 for buildings being evaluated to the Immediate Occupancy Performance Level; and
A_{col} = Area of the end column of the frame.

Brace Axial Stress Check

NC	BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.5.3.4, is less than 0.50 Fy. (Commentary: Sec. A.3.3.1.2. Tier 2: Sec. 5.5.4.1)

Assume braces are designed for tension and compression
Collapse Prevention Performance Level

Level		Level 1 (X)	Level 1 (Y)	Level 5 (X)	Level 5 (Y)	
L_{br}	=	21.26	19.69	21.26	19.69	[ft]
N_{br}	=	9	13	9	9	
s	=	17.6	15.6	17.556	15.615	[ft]
Brace	=	2-C8x18.75	2-C12x30	2-C8x13.75	2-C10x25	
A_{br}	=	11.02	17.62	8.08	14.68	[in^2]
V_{story}	=	14860	14860	10130	10130	[kip]
depth	=	8	12	8	10	[in]
t_{nom}	=	0.974	1.020	0.606	1.052	[in]
d/t	=	8	12	13	10	
F_y	=	33	33	33	33	[ksi]
F_{ye}	=	41	41	41	41	[ksi]
$90/(F_{ye})^{1/2}$	=	14	14	14	14	
$190/(F_{ye})^{1/2}$	=	30	30	30	30	
M_s	=	7.00	7.00	7.00	7.00	
f_j^{avg}	=	25.9	11.7	24.1	13.8	[ksi]
$0.5F_y$	=	16.5	16.5	16.5	16.5	[ksi]
Check	=	**Nonconforming**	**Conforming**	**Nonconforming**	**Conforming**	

4.4.3.4 Diagonal Bracing. The average axial stress in diagonal bracing elements, f_j^{avg}, shall be calculated in accordance with Eq. (4-9).

$$f_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{sN_{br}}\right)\left(\frac{L_{br}}{A_{br}}\right) \tag{4-9}$$

L_{br} = Average length of the braces (ft);
N_{br} = Number of braces in tension and compression if the braces are designed for compression, number of diagonal braces in tension if the braces are designed for tension only;
s = Average span length of braced spans (ft);
A_{br} = Average area of a diagonal brace (in.^{2});
V_j = Maximum story shear at each level (kip); and
M_s = System modification factor; M_s shall be taken from Table 4-9.

Table 4-9. M_s Factors for Diagonal Braces

Brace Type	d/t^b	CP[a]	LS[a]	IO[a]
		Level of Performance		
Tube[b]	$<90/(F_{ye})^{1/2}$	7.0	4.5	2.0
	$>190/(F_{ye})^{1/2}$	3.5	2.5	1.25
Pipe[c]	$<1{,}500/F_{ye}$	7.0	4.5	2.0
	$>6{,}000/F_{ye}$	3.5	2.5	1.25
Tension-only		3.5	2.5	1.25
Cold-formed steel strap-braced wall		3.5	2.5	1.25
All others		7.0	4.5	2.0

Note: $F_{ye} = 1.25F_y$; expected yield stress.
[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.
[b] Depth-to-thickness ratio.
[c] Interpolation to be used for tubes and pipes.

Connection Strength (Moderate Seismicity)

NC	CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4)

Check Connection Strength (Moderate Seismicity)

Brace	=	2-C8x18.75	2-C12x30	2-C8x13.75	2-C10x25	
A_g	=	11.02	17.62	8.08	14.68	
Brace Length	=	21.26	19.69	21.26	19.69	[ft]
ry	=	1.25	1.49	1.25	1.49	
KL/r	=	204.14	158.61	204.14	158.61	[kip]
Brace Cap.	=	**66**	**176**	**49**	**146**	[kip]
Pl thickness	=	5/8	5/8	5/8	5/8	[in]
Bolt diameter	=	0.912	0.912	0.912	0.912	[in]
n	=	8	8	8	8	
Fy	=	33	33	33	33	[ksi]
Fu	=	60	60	60	60	[ksi]
block length	=	11	11	11	11	[in]
block width	=	4	4	4	4	[in]
$\varphi Rn_{block\ shear}$	=	**484**	**484**	**484**	**484**	[kip]
Bolt Area	=	0.653	0.653	0.653	0.653	[in^2]
Fnv	=	27	27	27	27	[ksi]
$\varphi Rn_{bolt\ shear}$	=	**106**	**106**	**106**	**106**	[kip]
lc	=	2	2	2	2	[in]
$\varphi Rn_{bearing}$	=	**113**	**113**	**113**	**113**	[kip]
Check	=	**Conforming**	**Nonconforming**	**Conforming**	**Nonconforming**	

4.71*sqrt(E/Fy) = 139.62

Compact Members

C	COMPACT MEMBERS: All brace elements meet compact section requirements set forth by AISC 360, Table B4.1. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4)

	b/t_{des}	Fy	λ_r	Check
		[ksi]	0.56*SQRT(E/Fy)	
2-C8x18.75	4.6	33	16.6	**Conforming**
2-C12x30	4.4	33	16.6	**Conforming**
2-C8x13.75	4.2	33	16.6	**Conforming**
2-C10x25	4.7	33	16.6	**Conforming**

Column Splices

NC	COLUMN SPLICES: All column splice details located in braced frames develop 50% of the tensile strength of the column. (Commentary: Sec. A.3.3.1.3. Tier 2: Sec. 5.5.4.2)

Section	Area [in^2]	Tensile Strength [kips]	Bolts	Conn Shear Str. [kips]	
W14x193 (Col. 9-D)	56.7	1871.1	38	670.2	**Nonconforming**

Slenderness of Diagonals

NC	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have Kl/r ratios less than 200. (Commentary: Sec. A.3.3.1.4. Tier 2: Sec. 5.5.4.3)

Section	K	L	ry	KL/r	Check
		[ft]	[in]		
2-C8x13.75 (Br8)	1.0	25.3	1.3	242.6	**Nonconforming**

Compact Members

C	COMPACT MEMBERS: All brace elements meet section requirements set forth by AISC 341, Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4)

	b/t_{des}	Fy	λ_r	Check
		[ksi]	0.4*SQRT(E/Ry*Fy)	
2-C8x18.75	4.6	33	11.3	**Conforming**
2-C12x30	4.4	33	11.3	**Conforming**
2-C8x13.75	4.2	33	11.3	**Conforming**
2-C10x25	4.7	33	11.3	**Conforming**

Chevron Bracing

NC	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. (Commentary: Sec. A.3.3.2.3. Tier 2: Sec. 5.5.4.8)

Horiz. Member	h14	
Section	2-15C33.9	
Length	20	[ft]
Unbraced Length	10	[ft]
Estimated LRFD Moment Cap.	*210*	[kip-ft]
Story	9	
Brace Yielding in Tension	267	[kips]
Vertical Component of Tension Brace	211	[kips]
Induced Moment	*1057*	[kip-ft]
	Nonconforming	

Note that midspan of beam is supported perperdicularly (in plan) by another framing member (unbraced length = 10')