UCSF

Rating form completed by: Simpson Gumpertz & Heger
SCD, KDP, KSM

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

Date: 2020-11-02

UCSF Building Seismic Ratings

Mission Center Building

REGISTERED PROFESSIONAL ENGINEER
KEVIN S. MOORE
No. 4528
STRUCTURAL
STATE OF CALIFORNIA
11/02/2020

CAAN# 2415

1855 Folsom Street, San Francisco, CA 94103

UCSF Campus Site: *Not a part of concentrated group of UCSF buildings*

Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	V	Findings based on drawing review and ASCE 41-17 Tier 1 and 2 evaluations[1]
Rating basis	Tier 1 & 2	Design drawings and ASCE 41-17
Date of rating	2020	
Recommended UCSF priority category for retrofit	B	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application for modification
Ballpark total project cost to retrofit to IV rating	Very High (>$400/sf)	Significant retrofit, including site/soil remediation necessary to achieve SPL IV
Is 2018-2019 rating required by UCOP?	Yes	
Further evaluation recommended?	Yes	Additional analysis to determine size and scope of required retrofit

[1] The evaluations at UCSF translate the Tier 1 evaluation to a Seismic Performance Level rating using professional judgment discussed among the Seismic Review Committee. Non-compliant items in the Tier 1 evaluation do not automatically put a building into a particular rating category, but such items are evaluated 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.

Building information used in this evaluation

- P.D. Burtt Engineer, 1926. Building for Illinois California Glass Company, (16 drawings last dated May 10, 1926).
- Rutherford and Chekene, 1970.
    - Preliminary Investigation Report of Existing Building at 1855 Folsom St., San Francisco, California, January 1970, (19 pages)
    - Dynamic soils analysis, June 1, 1970 (21 pages)
    - Schematic Phase Final report (17 pages), February 20, 1970
- Rutherford and Chekene, 1971. Center for Educational Development, Sheets S-6 to S-23 (16 drawings) last dated March 43, 1971; reviewed by State of California Office of the State Architect (OSA) March 4, 1971.
- Impel Corp., 1989. Performance of UCSF Buildings During the October 17, 1989 Loma Prieta Earthquake, (50 pages), dated November 17, 1989.
- UCSF, Report to the President Gardner, UC, October 17, 1989 UCSF Earthquake Report, November 30, 1989, from CSU Chancellor, November 30, 1989. (175 pages)
- Wiss Janney Elstner, 2013. UCSF Mission Center, 1855 Folsom, Façade Evaluation Summary Report Update, (35 pages).
- Estructure, 2019. *UCSF Mission Center, Building Brick Remediation Project, May 20, 2019 (33 sheets).*

Additional building information known to exist

- None pertinent to seismic evaluation specific tothe building.

Scope for Completing this Form

We reviewed the structural drawings for original construction and modifications. We also reviewed the ASCE 41-17 Tier 1 evaluation performed by CC Theil Jr of TELESIS Engineers. We developed a linear elastic structural model in ETABs and performed an ASCE 41-17 Tier 2 evaluation of the structure.

Brief Description of Structure

The Mission Center building is located on the east side of Folsom Street in the City of San Francisco. The building was constructed in the mid-1920s as a glass product manufacturing facility. Plans for the original construction were prepared by P.D. Burtt Engineer. The building was structurally modified in the early 1970s to plans prepared by Rutherford & Chekene Structural Engineers. Reviewed plans do not indicate the building codes used for the original design or subsequent structural modifications. For the latter work this is probably because it was an OSA (predecessor of DSA) reviewed schoolhouse.

The building is a six-story, rectangular in plan structure with overall dimensions of approximately 193 ft by 256 ft. The story heights are as follows: first – 14 ft; second through sixth – 12 ft. The total height of the building is approximately 74 ft. Figure 1 shows the plan of the original building and Figure 2 shows the elevation of the north side, as modified.

Logs of six borings drilled at site by Shannon & Wilson (1970) indicate surficial fill materials, consisting predominantly of loose sand (with some gravel and construction debris), extending to depths between ≈ 13 and 20 ft. The fill is underlain by a stratum of soft, medium-to-high plasticity clay (i.e., Young Bay Mud) extending to depths between ≈ 50 and 75 ft below the existing building, then dense to very dense silty sands and stiff silty clays extending to serpentinite greenstone bedrock that was encountered at a depth of ≈ 125 ft below ground surface. Groundwater was measured in a standpipe at a depth of 9 ft.

An inspection report by Rutherford and Chekene of materials values, foundation support information and several key findings were reviewed including:

- Real loads to the structure can be increased by 50% with capacity for each pile at 22 tons.
- A 33% increase in pile loading under dead plus live plus seismic for overturning forced on piles.
- Future settlement will not be altered by increased loads as limited above.
- Tested piles were assessed as sound.

Foundation System: Foundation support is provided by driven timber piles and concrete pile caps and grade beam, see Figure 3. The first floor is a concrete slab-on-grade reinforced with welded wire fabric. The pile caps are supported by pile groups about 13 piles per group, with heavy timber piles extending 6 in. into the pile cap, see Figure 4. The seismic retrofit included placing 32-inch wide heavily reinforced perimeter grade beams, without 135-degree ductile hooks, at the exterior to couple the foundations over the width of the building, see bottom of Figure 2.

Impel reports that in the 1989 Loma Prieta earthquake the concrete sidewalks located around the building cracked and appeared to have settled between 2 and 6 in. relative to the building foundation. This was because the top layer of sand located directly underneath the sidewalk densified by the earthquake shaking and settled. A review of the soils report for this site revealed that a sand and gravel layer of soil exists on the upper 22 ft below grade. Impell also reviewed top layers of soil underneath the buckled sidewalks and determined that their base was loose sand. The deep driven piles and pile caps mitigate this potential under the building. The Chancellor's report on the 1989 earthquake's impacts on UCSF facilities did not mention this building.

Structural System for Vertical (Gravity) Load: The roof and elevated floors are reinforced concrete slabs supported by reinforced concrete interior columns with capitals and drop panels and reinforced concrete perimeter beams. The perimeter beams are supported by reinforced concrete columns.

Structural System for Lateral Loads: The roof and elevated floor reinforced concrete diaphragms distribute earthquake loads to the perimeter reinforced concrete shear walls. The interior reinforced concrete moment frame provides some backup to the perimeter system, but the perimeter pierced concrete wall system is considerably stiffer than the interior frames.

Condition Observations: The exterior of the building shows signs that some embedded steel that was used in the masonry exterior is degrading and the exterior cracks attributed to water infiltration were observed, see Figure 6. Wiss Janney Elstner found that, in general, the brick masonry components of the building facade were in reasonably good condition, and aside from veneer anchorage issues, the noted deterioration was caused by the normal aging process of the building components. The deteriorated mortar joints and leaking windows have permitted water to infiltrate the masonry and cause corrosion and efflorescence. Implementing the window repairs was recommended and repointing the mortar joints should alleviate many of the noted problems. The window lintel corrosion and corrosion of the veneer ties were not considered hazardous in 2006 but the deterioration has since progressed to the point that prompt attention is required to prevent these accelerating forms of deterioration from progressing further, to the point of becoming falling hazards. We understand that at the present time, a brick remediation project is underway.

Evidence was not observed to suggest that this is also a problem for any of the concrete elements.

Significant concrete floor cracking has been observed and substantial differential settlement between the Folsom Street side and Harrison Street sides has been reported. Load tests were performed on the fourth floor slab in 1971 and indicated that the slab was still elastic and complied with the current ACI code criteria for evaluation of existing structures. The Shannon and Wilson report recommended that the structure be designed (as part of the R+C retrofit) to tolerate settlements of 0.1 to 0.2 feet in the next twenty years. The report states that this settlement would likely occur as differential settlement with the east settling relative to the west.

Past seismic performance: The building was in place at the time of the 1989 Loma Prieta earthquake. Impel Corporation in their 1989 report covered this specific building in its review of UCSF building performance. They reported interior disruption to the fourth and fifth floors (only one occupied at that time by UCSF) as needed repairs for architectural elements, fallen light fixtures and other similar components.

Description of the Tier 2 Analysis Model

We developed a linear elastic computer model using CSI SAP software. Response spectrum analysis was used in accordance with ASCE 41-17 to determine the seismic response of the building. We assessed a Life Safety performance objective under the BSE-R hazard and a Collapse Prevention performance objective under the BSE-C hazard. For the lateral force resisting system, we modeled the perimeter shear walls using shell elements to form piers and spandrels. We accounted for the interior slab-column moment frame using an effective beam model per the methodology in ASCE 41-17.

We assessed the structure using various stiffness modifiers for the perimeter walls and interior slab-column moment frame. The perimeter walls provide the majority of the stiffness under all conditions. We developed simplified models and calculations along with models of the entire building in using ETABS. First, we developed a hand calculation for multistory, multi-bay frames that assumes the inflection points occur at mid-height of each column and midspan of each beam. We applied point loads at each floor level and calculated the displacement for a single exterior bay (walls) and a single interior bay (columns). We used these displacements to calculate frame stiffness per line. To account for the number of frame lines of each type, we multiplied the wall stiffness by (2) and the interior frame stiffness by (9) to get total system stiffness. This calculation resulted in 93% of the base shear in the perimeter wall frames and 7% of the base shear in the interior column frames and is shown in the following table as Case A.

Next, we developed a series of single frame analyses in ETABS. In each case we modeled the exterior frame and the interior frame independently, applied the same loading, used the roof displacement to calculate each frame stiffness, and extrapolated that to the full building as described above. The analyses we conducted and resulting relative stiffnesses are as follows:

- Case B: We modeled a single bay of perimeter walls and a single bay of interior frames with point loads applied at the roof level only. This resulted in a force distribution of 90% in perimeter walls and 10% in interior frames. This methodology is not entirely accurate as it ignores the stiffness due to continuity of the entire frame lengths.
- Case C: We modeled a single bay of perimeter walls and a single bay of interior frames with point loads uniformly up the height of the frames. This resulted in a force distribution of 92% in perimeter walls and 8% in interior frames.
- Case D: We modeled the full building length of perimeter walls and interior frames with point loads applied at the roof level only. This resulted in a force distribution of 96% in perimeter walls and 4% in interior frames.

Relative Force Distribution between Walls and Frames (Hand calcs and partial ETABS models)

Case	% Base Reactions	
	Walls	Columns
A	93	7
B	90	10
C	92	8
D	96	4

The table below lists the relative base shear resisted by the walls and columns based on the full ETABS model with various assumed stiffness modifiers. In comparison to the calculations discussed above, the walls resist over 95% of the base shear regardless of assumed stiffness properties.

Relative Force Distribution between Walls and Frames (Full ETABS model)

-	Wall Modifiers		Column Modifiers		Effective Beams		% Base Reactions		-
No.	Flexure	Shear	Flexure	Shear	Flexure	Shear	Walls	Columns	Notes
1	0.35*Ig	1.0*Ag	0.3*Ig	1.0*Ag	0.3-0.5*Ig	1.0*Ag	99%	1%	ASCE 41-17 Default Modifiers
2	0.35*Ig	0.5*Ag	0.3*Ig	1.0*Ag	0.3-0.5*Ig	1.0*Ag	99%	1%	Wall Shear Modifier per TBI
3	0.175*Ig	0.25*Ag	0.3*Ig	1.0*Ag	0.3-0.5*Ig	1.0*Ag	98%	2%	0.5*TBI Wall Modifiers
4	0.35*Ig	0.5*Ag	1.0*Ig	1.0*Ag	1.0*Ig	1.0*Ag	97%	3%	TBI and Uncracked Interior
5	0.175*Ig	0.25*Ag	1.0*Ig	1.0*Ag	1.0*Ig	1.0*Ag	95%	5%	0.5xTBI and Uncracked Interior

We compared the design load between the original building (1926), the retrofit (1971), and the current requirements and tabulate this comparison below.

Year	Basis	Base Shear Coeff	Building Weight	Base Shear
			kips	*kips*
1926	Approximate	0.04	53000	2120
1971	1967 UBC Assuming Shear Wall System	0.08	70000	5670
	1967 UBC assuming Box System	0.11	70000	7560
2020	ASCE 41-17 BSE-C Response Spectrum (unreduced)	1.42 (1.0)*	70000	99220
	ASCE 41-17 BSE-C Response Spectrum (assume average Pier m = 2.3)	0.62	70000	43139

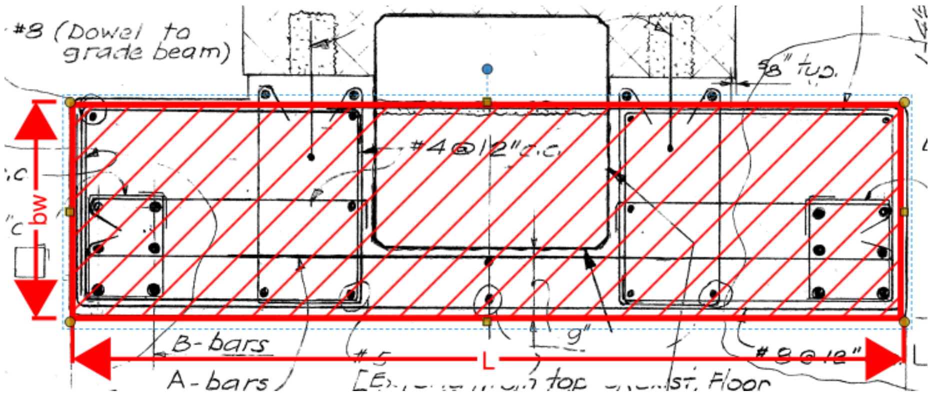
*ASCE 41-17 values are LRFD based. Number in parentheses indicates ASD value.

Description of the Perimeter Wall Capacity Calculations

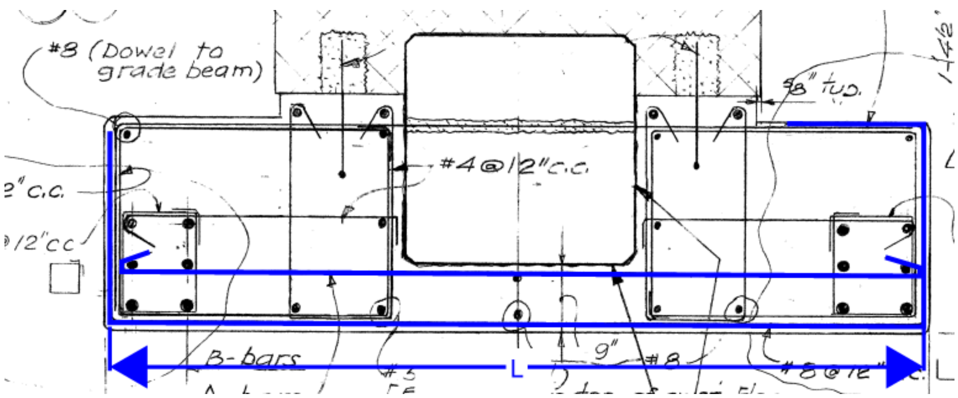
The retrofit performed in the 1970s added concrete piers and spandrels at the perimeter of the building. These were integrated with the existing columns and beams by doweling into them. We calculated the capacities of the composite elements as described below.

The vertical wall elements (piers) are comprised of the original rectangular perimeter columns that have been surrounded by retrofit wall elements. The shear capacity of the section is composed of three elements in the combined section:

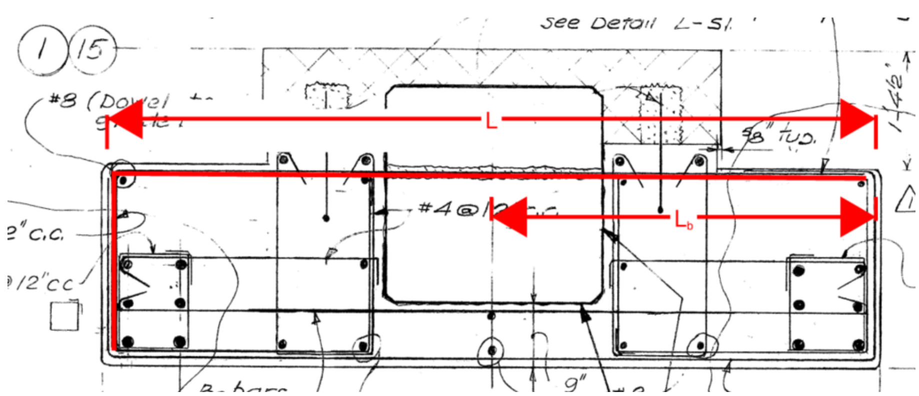
- Concrete Shear Strength



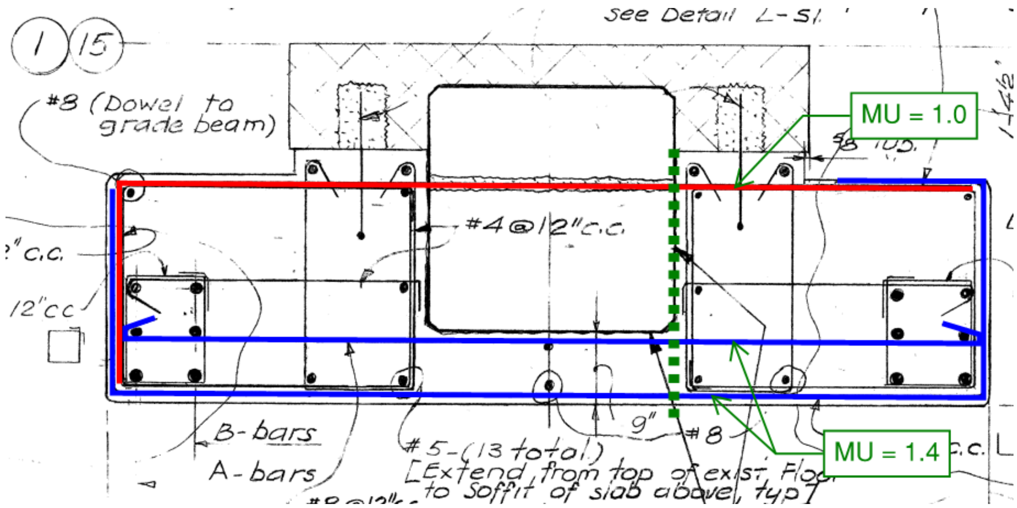
- Continuous bars with appropriate hooks/bends to develop the bar



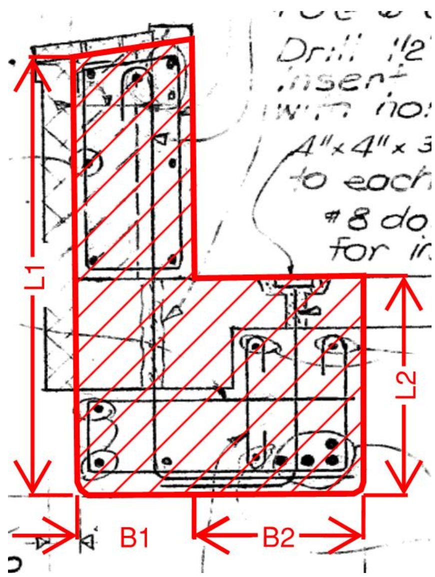
- Straight bars that are partially developed in the section. We calculated the effective yield stress of these bars using EQ 10-1a from ASCE 41-17.



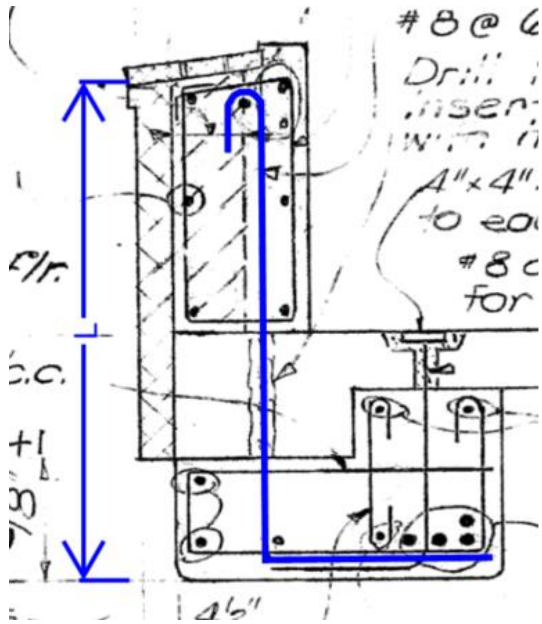
The capacity provided by these elements were combined to determine the expected capacity of the cross section. We did not consider the capacity provided by the hoops on either side of the original column as they are not able to provide a tie the full width of the cross section. Additionally, we evaluated the ability of the retrofit bars to tie the original columns to the added shear wall sections together. This relies on the retrofit bars in shear friction to resist the shear flow across the interface between the original column and the retrofit section. We considered the shear friction interface shown below with the associated shear friction coefficients (mu). Using shear flow, we were able to calculate a maximum shear in the cross section based on shear flow across this interface. This results in a 10%-75% reduction in shear capacity compared to the expected strength outlined above.



- The horizontal wall elements (spandrels) are comprised of the original edge slab beam, with additional retrofit sections above and below. The shear capacity of the section is composed of two elements in the combined section:
    - Concrete Shear Strength

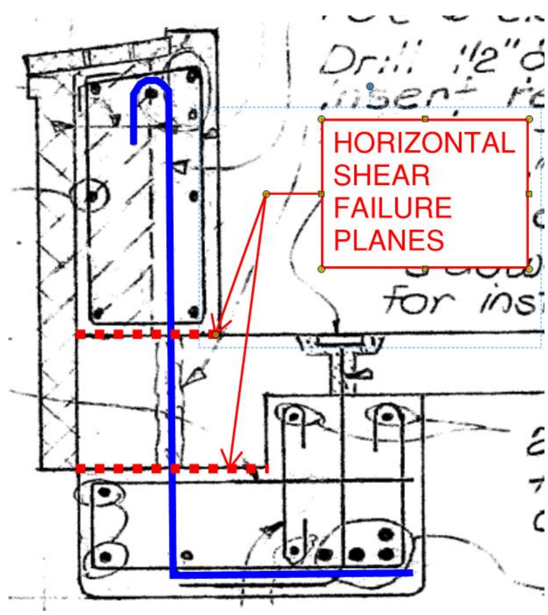


- Continuous bar doweled through the original edge beam and hooked 90 or 180 degrees at each end.



The capacity provided by these elements were combined to determine the expected capacity of the cross section. We did not consider the stirrups provided in the retrofit sections above and below the original edge beam because they are not able to provide a tie at the full height of the cross section.

Additionally, we evaluated the ability of the retrofit bars to tie the combined retrofit section together. This relies on the doweled bar in shear friction to resist the shear flow across the interfaces between the original edge beam and the retrofit sections. We considered the shear friction interfaces shown with a shear friction coefficient (mu) of 1.0. Using shear flow, we were able to calculate a maximum shear in the cross section based on shear flow across this interface. This results in a 55%-77% reduction in shear capacity compared to the expected strength outlined above.



Description of seismic deficiencies

Identified seismic deficiencies of the building include the following:

- The wall spandrels and piers at Floor 6 exhibit weak column-strong beam behavior for the east and west walls (loading in the north-south direction). All other locations exhibit strong column-weak beam behavior.
- The flat slab bottom steel consists of orthogonal bands within the column strips and diagonal bands across the full slab. Both bands are square bars that are lapped 50 bar diameters at the column. The laps are insufficient to develop the bars, resulting in limited positive flexural capacity and the potential for brittle failure.
- The vertical wall elements (piers) do not provide sufficient capacity to meet the BSE-C demands. For elements controlled by shear capacity outlined above, overstressed piers had DCRS up to 4.2. Without the reduction due to shear friction capacity (expected strength only), the piers controlled by shear are still overstressed up to a DCR of 3.8. Overstressed piers controlled by PM interaction have DCRs up to 5.3.
- The horizontal wall elements (spandrels) do not provide sufficient capacity to meet the BSE-C demands. For elements controlled by the shear capacity, overstressed spandrels had DCRs up to 8.4. Without the reduction due to shear friction capacity (expected strength only), the spandrels controlled by shear are still overstressed up to a DCR of 2.0. Overstressed spandrels controlled by PM interaction have DCRs up to 9.7.
- We assessed the capability of the interior frames alone to resist the seismic demands. As shown in the study above, the frames are significantly less stiff than the perimeter walls, so it was assumed that the perimeter walls would have to be heavily damaged before the frames provide significant lateral resistance. This was modeled by setting the lateral stiffness of the walls in moment and flexure to zero. The resulting analysis showed the columns are overstressed up to a DCR of 6.3 due to PM interaction. However, the columns are well reinforced with spiral hoops (gage wire specified on drawings) and have shear capacities greater than the resultant shear due to flexural hinges forming at the ends, indicating that they will likely continue to support gravity load even if flexural hinges form in them.
- The exterior of the building shows evidence of corrosion of embedded steel that supports and restrains the brick masonry walls. A project is currently underway to mitigate the falling hazard associated with the exterior brick. There is no evidence that the exterior concrete elements beneath the brick is deteriorated.

The large number of items noted above will collectively affect the seismic performance of the building such that failures will occur and negatively affect the building performance. The perimeter piers and spandrels added during the retrofit are deficient in shear and flexure. Additionally, the dowels used to tie these newer elements into the existing elements do not have adequate shear friction capacity to enable the elements to act compositely, further reducing their capacities. However, the relatively large punching shear strength of the existing flat slab and the columns should prevent collapse in a large event. The slab punching strength is greater than the strength required to develop flexural hinges at the columns. Similarly, the column shear strengths are greater than that required to develop flexural hinges in the columns indicating that the columns will likely continue to support gravity load even after failure in flexure.

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
Geometry (vertical irregularities)	N	URM wall height-to-thickness ratio	N
Torsion	N	URM parapets or cornices	N
Mass – vertical irregularity	N	URM chimney	N
Cripple walls	N	Heavy partitions braced by ceilings	N
Wood sills (bolting)	N	Appendages	N
Diaphragm continuity	N		

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

The UCOP non-structural checklist item check list for *Life Safety Hazard* concludes that there are no nonstructural issues of concern in evaluating this building's expected seismic performance.

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
Heavy masonry or stone veneer above exit ways and public access areas	Yes*	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

* Remediation under way to mitigate hazard associated with brick cladding; Permit drawings by Estructure, April, 2019

Basis of seismic performance level rating

The building is rated as V considering the list of deficiencies and the high level of redundancy found throughout the building.

Recommendations for further evaluation or retrofit

No further evaluation required. Installation of new perimeter multi-bay shear walls recommended to improve rating.

Peer review comments on rating

The structural members of the UCSF Seismic Review Committee (SRC) reviewed the evaluation on 10 March 2020 and are in unanimous agreement with the rating.

Additional building data	Entry	Notes
Latitude	37.76745°	John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
Longitude	-122.41511°	John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	6	
Number of stories (basements) below lowest perimeter grade	0	
Building occupiable area (OGSF)	240,000	
Risk Category per 2016 CBC 1604.5	II	It is reported that the building is not used for any educational purposes.
Building structural height, h_n	74 ft	Structural height defined per ASCE 7-16 Section 11.2
Coefficient for period, C_t	0.02	ASCE 41-17 equation 7-18
Coefficient for period, β	0.75	ASCE 41-17 equation 7-18
Estimated fundamental period	0.505 sec	ASCE 41-17 equation 7-18
Site data		
975 yr hazard parameters S_s, S_1	1.414, 0.550	https://hazards.atcouncil.org/
Site class	E	This is for the surface soils in the immediate area. This does not reflect the impact on seismic motions from interaction of highly-redundant deep pile foundations into more competent materials at depth. A higher Level ASCE 41 will assess this assignment's appropriateness.
Site class basis		John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
Site parameters F_a, F_v	1.3, 4.2	John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report

Additional building data	Entry	Notes
Ground motion parameters S_{cs}, S_{c1}	1.839, 2.308	John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
S_a at building period	1.834	John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
Site V_{s30}	210 m/s	John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
V_{s30} basis		John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
Liquefaction potential	Yes, as mapped	John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
Liquefaction assessment basis	Assessment	Evaluated as not a significant risk to the building that is supported by over 2,172 deep piles in 165 clusters, with about one pile per 23 square feet. The building is evaluated as having no overturning risk. Assuming that the piles extend below the liquefiable zones, the densification caused by the piles is about 6% by volume, moderating the liquefaction susceptibility of the supporting soils.
Landslide potential	No	John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
Landslide assessment basis	-	
Active fault-rupture hazard identified at site?	No	John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
Site-specific ground motion study?	No	
Applicable code		
Applicable code or approx. date of original construction	1926 design and 1971 retrofit, none referenced	1971 design reviewed by DSA as an educational facility. Assumed equivalent to 1970 UBC.
Applicable code for partial retrofit	Unknown	DSA review as a schoolhouse in 1970, missing first design sheet with details not located.
Applicable code for full retrofit	N/A	
FEMA P-154 data		
Model building type North-South	C2	Concrete shear wall with stiff diaphragms
Model building type East-West	C2	Concrete shear wall with stiff diaphragms

Additional building data	Entry	Notes
FEMA P-154 score	N/A	Not included here because an ASCE 41-17 Tier 1 evaluation was conducted
Previous ratings		
Most recent rating	IV	2013 UCSF SRC Rating
Date of most recent rating	10/7/2013	
Appendices		
ASCE 41 Tier 1 checklist included here?	Yes	Refer to attached checklist file

Appendix A

Figures

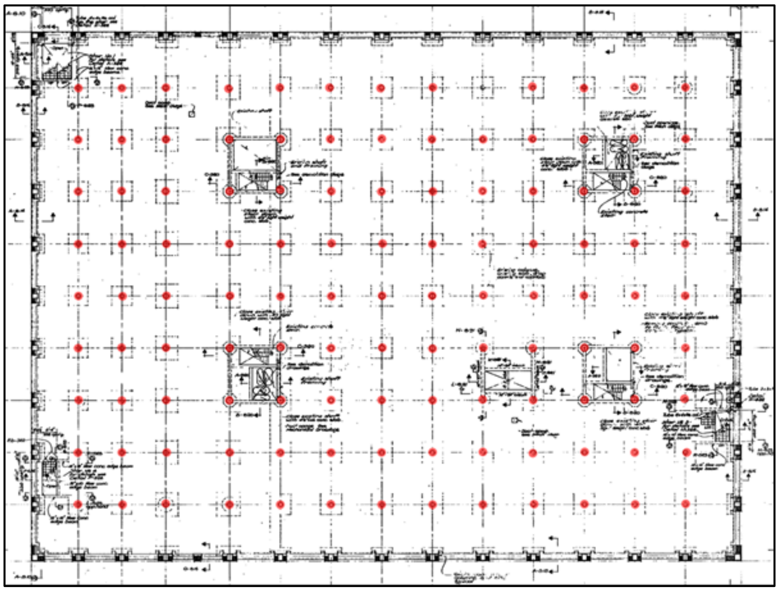


Figure 1. The original plan of one of the floors showing the interior concrete columns (red), and the exterior concrete columns (black dots) within the exterior concrete frame, and exterior brick walls. See Figure 7 for an elevation of the typical exterior wall with annotations of the added concrete framing on the interior surface of the brick infill. As constructed, the fourth bay from the left had a drive isle the full width of the building. At the perimeter there are rectangular columns integrated into the concrete spandrels in the original construction, and the black dots represent the 1971 retrofit piers, see Figure 4.

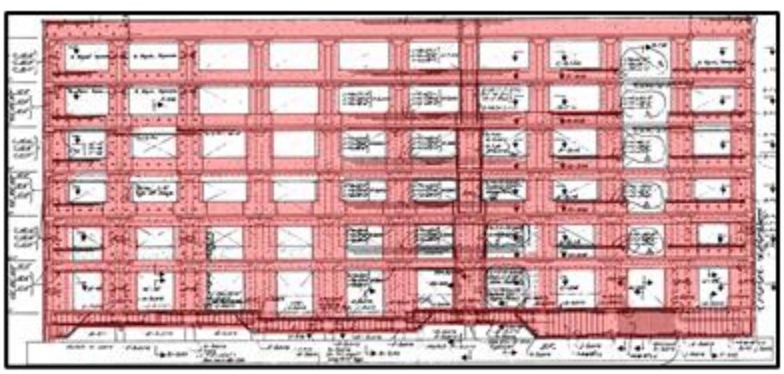


Figure 2. The reinforced concrete pier and spandrel system (red highlight) installed in the 1971 retrofit. See Figure 5 for the detailing of the added pier elements around the existing columns and attachment to the exterior brick masonry by the small + marks. At the bottom, the piers terminate into a heavily reinforced concrete grade beam.

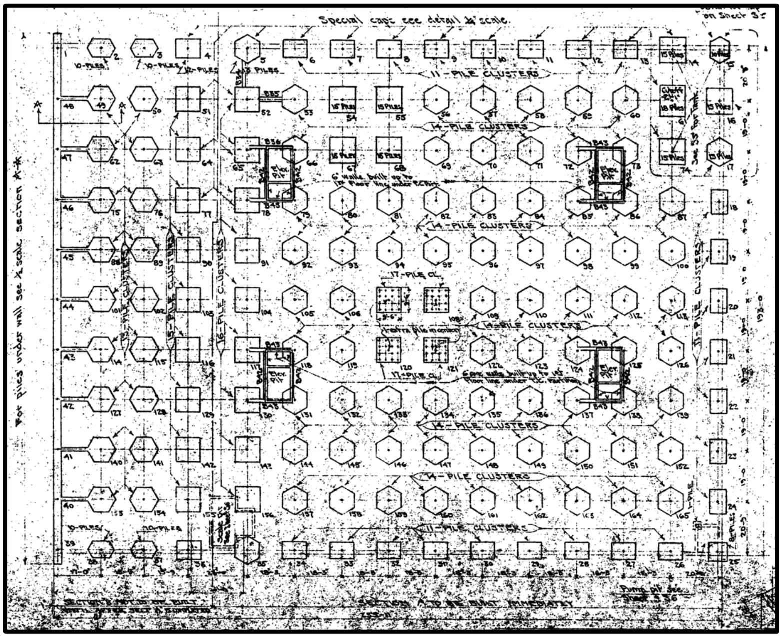


Figure 3. The pile plan from the original design. There are a number of different pile cap arrangements. The square caps of the fourth and fifth vertical grid lines indicate the location of a driveway the full width of the building, since enclosed. A typical pile cap is shown in Figure 4. Note that the pile caps support all interior columns and walls.

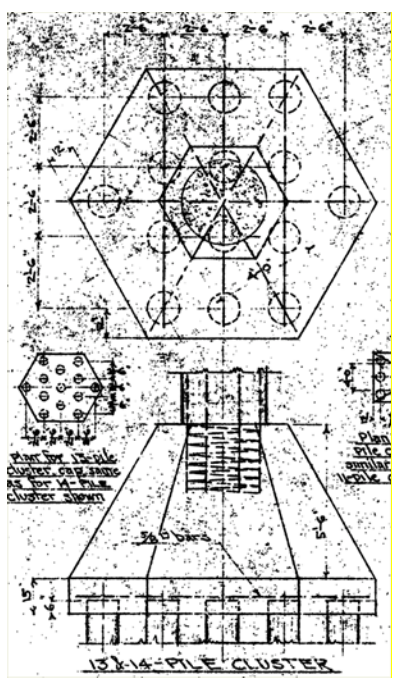


Figure 4. A typical pile cap for a 13- or 14-pile cluster. The driven timber piles extend 6 inches into the concrete pile cap. The concrete column reinforcing cage extends into the pile cap. At the bottom of the cap, above the piles is a dense two-directional reinforcing mat.

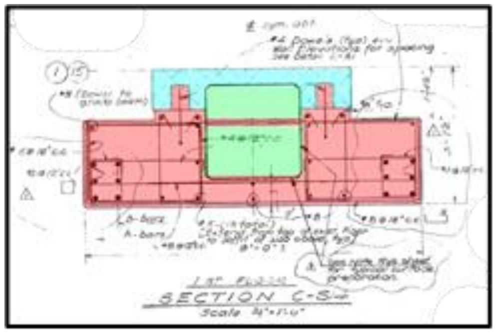


Figure 5. The 1971 retrofit resulted in the conversion of the reinforced concrete perimeter columns (green highlight) into heavily reinforced piers (red highlight) of the perimeter shear walls. These piers along with the original reinforced concrete perimeter beams form a very robust perimeter shear walls. The exterior brick masonry (blue highlight) is reliably anchored to the pier.

Figure 6. Sections of steel lintel rusting (upper marked by green), and cracking (lower image to the left marked with green) in the brick masonry exterior from Wiss Janney Elstner report on the façade in 2013 showing conditions probably caused by water infiltration at windows.

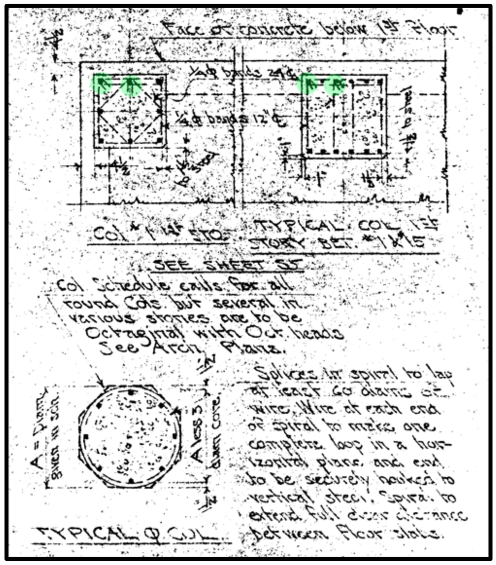


Figure 7. The 1926 columns reinforcing included spiral reinforcing for round and octagonal column, and well reinforced square and rectangular columns that include 135° hooks (green). The ties are #2 at 12 in. o.c., and the spiral reinforcing is #0-#3 wire at pitches of ±2.25 in. .

Appendix B

Checklists

UC Campus:	Parnassus		Date:	06-11-2020		
Building CAAN:	2415	Auxiliary CAAN:		By Firm:	Simpson Gumpertz & Heger	
Building Name:	Mission Center Building		Initials:	AS	Checked:	KDP
Building Address:	1855 Folsom Street, 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: Concrete diaphragms transfer loads to the walls, and the walls transfers load to the foundations.**
C NC N/A U (N/A 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: No adjacent building in close vicinity.**
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: No mezzanines.**

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

	Description
C NC N/A U (C selected)	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: Shear strength increases as we go down the stories**
C NC N/A U (C selected)	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: Shear stiffness increases as we go down the stories**

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

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Building Address:	1855 Folsom Street, San Francisco, CA 94143		Page:	2	of	3

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

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: Compliant for most of the structure. Three west elevation wall piers stop at level 02.**
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: The wall lengths and floor plans are consistent over the height. Below 2nd level where the net wall length is smaller, however the net change is less than 10% of story above.**
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: Mass of level 6 is 57% more than roof, others are around 10% different. Since light roofs need not be considered, the building is compliant.**
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: The building is rectangular with approximately symmetric about both axes.**

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

GEOLOGIC SITE HAZARD

	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: Liquefaction potential is very high but the building bears on over 2,000 piles (assumed to be driven to adequate depth).**
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: slope failure not likely to affect the building because of no slope.**

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Building Name:	Mission Center Building			Initials:	AS	Checked:	KDP
Building Address:	1855 Folsom Street, San Francisco, CA 94143			Page:	3	of	3

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

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

GEOLOGIC SITE HAZARD

C	NC	N/A	U	
◉	○	○	○	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1) **Comments: Faults are adequately distant and do not pose a risk at this site.**

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

FOUNDATION CONFIGURATION

C	NC	N/A	U	Description
◉	○	○	○	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3) **Comments: base/height at East corner is 189.8/74 = 2.6 > 1.1**
○	◉	○	○	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: Pile caps are not tied together with grade beams, relatively think slab-on-grade several feet above top of pile will likely be heavily damaged when liquefaction of the underlying soil occurs and may not tie caps together.**

UC Campus:	**UCSF - Parnassus**			Date:	**06-11-2020**		
Building CAAN:	**2415**	Auxiliary CAAN:		By Firm:	**Simpson Gumpertz & Heger**		
Building Name:	**Mission Center**			Initials:	***GSV***	Checked:	***CCT***
Building Address:	**1855 Folsom Street San Francisco, CA**			Page:	**1**	of	**3**

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

Low and Moderate Seismicity

Seismic-Force-Resisting System

C NC N/A U	Description
(•) 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: Interior spirally reinforced concrete column system is a secondary component and together with other elements (perimeter reinforced concrete walls) form a complete vertical load-carrying 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: The number of lines of shear walls in each principal direction is equal to 2.**
( ) 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} (0.69 MPa) or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1) **Comments: The shear stress in the concrete shear walls is more than $2\sqrt{f'c}$ psi. Maximum DCR at Ground Level is 3.5**
(•) 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: The ratio of reinforcing steel to gross concrete area is greater than 0.0020 in both directions.**

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 are concrete.**
(•) 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: The roof and elevated floor diaphragms are continuous reinforced concrete slabs that are reliably connected to the perimeter shear walls.**
(•) 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: (4)#10 dowels are used where (2)#11 wall reinforcement exists.**

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

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Building CAAN:	2415	Auxiliary CAAN:		By Firm:	Simpson Gumpertz & Heger		
Building Name:	Mission Center			Initials:	*GSV*	Checked:	*CCT*
Building Address:	1855 Folsom Street San Francisco, CA			Page:	2	of	3

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

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

Seismic-Force-Resisting System

C NC N/A U	Description
C (selected)	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2) **Comments: The secondary columns are well reinforced with spiral hoops and have shear capacities greater than the resultant shear due to flexural hinges forming at the ends.**
NC (selected)	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3) **Comments: Bottom steel is not continuous through the joints.**
C (selected)	COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1) **Comments: All walls are supported at each end.**

Diaphragms (Stiff or Flexible)

C NC N/A U	Description
C (selected)	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1) **Comments: The diaphragms are not composed of split-level floors and do not have expansion joints.**
C (selected)	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3) **Comments: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.**

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

Flexible Diaphragms

C NC N/A U	Description
N/A	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) **Comments:**
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: Diaphragms are concrete.**

Connections

C NC N/A U	Description
NC	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: The timber piles are embedded 6” into the pile caps and not mechanically connected to resist uplift. Calculations in the appendix show uplift forces in the corner piers.**

Appendix C

Tier 1 Calculations

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

CLIENT UCSF
SUBJECT Mission Center - Flat Loads

SHEET NO.
PROJECT NO. 197042.00
DATE 09/10/2019
BY JSF/SCD
CHECKED BY KDP

1 - Typical Floor

Level	Material	Slab (psf)	Beam (psf)	Cols (psf)	Seismic (psf)	Gr. Cols (psf)	Modeled Y/N	SDL (psf	Remarks
TYP FL.	10" Concrete Slab	125.0	125.0	125.0	125.0	125.0	Y	0.0	
"	Ceilings, sprinklers & mech	10.0	10.0	10.0	10.0	10.0		10.0	
"	Flooring	2.0	2.0	2.0	2.0	2.0		2.0	
"	Partitions (Full height)	0.0	0.0	0.0	10.0	0.0		10.0	
"	Columns (assume 26" diam average)	19.1	19.1	19.1	19.1	19.1	Y	0.0	
"	Miscellaneous	3.9	3.9	3.9	3.9	3.9		3.9	
	Sum of Dead Loads	**160.0**	**160.0**	**160.0**	**170.0**	**160.0**		**25.9**	
	Sum of Live Loads	**50.0**	**50.0**	**50.0**	**-**	**50.0**		**50.0**	
	Sum of Dead Plus Live Loads	**210.0**	**210.0**	**210.0**	**170.0**	**210.0**			

2 - Roof

Level	Material	Slab (psf)	Beam (psf)	Cols (psf)	Seismic (psf)	Gr. Cols (psf)	Modeled Y/N	SDL (psf	Remarks
TYP RF.	7" Concrete Slab	87.5	87.5	87.5	87.5	87.5	Y	0.0	
"	Ceilings, sprinklers & mech	10.0	10.0	10.0	10.0	10.0		10.0	
"	Roofing	5.0	5.0	5.0	5.0	5.0		5.0	
"	Partitions (Half height)	0.0	0.0	0.0	5.0	0.0		5.0	
"	Columns (assume 18" diam average)	4.6	4.6	4.6	4.6	4.6	Y	0.0	
"	Miscellaneous	2.9	2.9	2.9	2.9	2.9		2.9	
	Sum of Dead Loads	**110.0**	**110.0**	**110.0**	**115.0**	**110.0**		**22.9**	
	Sum of Live Loads	**20.0**	**20.0**	**20.0**	**-**	**20.0**		**20.0**	
	Sum of Dead Plus Live Loads	**130.0**	**130.0**	**130.0**	**115.0**	**130.0**			

3 - Wall Piers

Level	Material	Slab (psf)	Beam (psf)	Cols (psf)	Seismic (psf)	Gr. Cols (psf)	Modeled Y/N	SDL (psf	Remarks
TYP RF.	Wall Pier (Assume 27" Average)	337.5	337.5	337.5	337.5	337.5	Y	0.0	
	Sum of Dead Loads	**337.5**	**337.5**	**337.5**	**337.5**	**337.5**		**0.0**	
	Sum of Live Loads		**0.0**	**0.0**	**-**	**0.0**		**0.0**	
	Sum of Dead Plus Live Loads	**337.5**	**337.5**	**337.5**	**337.5**	**337.5**			

4 - Spandrel Beams Below

Level	Material	Slab (psf)	Beam (psf)	Cols (psf)	Seismic (psf)	Gr. Cols (psf)	Modeled Y/N	SDL (psf	Remarks
TYP RF.	Spandrel Beam 4'	600.0	600.0	600.0	600.0	600.0	Y	0.0	
	Sum of Dead Loads	**600.0**	**600.0**	**600.0**	**600.0**	**600.0**		**0.0**	
	Sum of Live Loads	**0.0**	**0.0**	**0.0**	**-**	**0.0**		**0.0**	
	Sum of Dead Plus Live Loads	**600.0**	**600.0**	**600.0**	**600.0**	**600.0**			

5 - Spandrel Beams Above

Level	Material	Slab (psf)	Beam (psf)	Cols (psf)	Seismic (psf)	Gr. Cols (psf)	Modeled Y/N	SDL (psf	Remarks
TYP RF.	Spandrel Beam (Assume 10" Average)	125.0	125.0	125.0	125.0	125.0	Y	0.0	
	Sum of Dead Loads	**125.0**	**125.0**	**125.0**	**125.0**	**125.0**		**0.0**	
	Sum of Live Loads	**0.0**	**0.0**	**0.0**	**-**	**0.0**		**0.0**	
	Sum of Dead Plus Live Loads	**125.0**	**125.0**	**125.0**	**125.0**	**125.0**			

6 - Cladding

Level	Material	Slab (psf)	Beam (psf)	Cols (psf)	Seismic (psf)	Gr. Cols (psf)	Modeled Y/N	SDL (psf	Remarks
TYP RF.	Brick - 3 Wythes	115.0	115.0	115.0	115.0	115.0		115.0	
"	Windows	8.0	8.0	8.0	8.0	8.0		8.0	6'10x13'8"
	Sum of Dead Loads	**71.2**	**71.2**	**71.2**	**71.2**	**71.2**		**71.2**	*Weighted Average of wall Load*
	Sum of Live Loads	**0.0**	**0.0**	**0.0**	**-**	**0.0**		**0.0**	
	Sum of Dead Plus Live Loads	**71.2**	**71.2**	**71.2**	**71.2**	**71.2**			

SIMPSON GUMPERTZ & HEGER
Engineering of Structures
and Building Enclosures

SHEET NO. ______
PROJECT NO. 197042.00
DATE 09/09/2019
BY SCD
CHECKED BY KDP

CLIENT UCSF
SUBJECT Mission Center – Weight Takeoff

	Gravity	Seismic	Live
	(psf)	(psf)	(psf)
1 - Typical Floor	160.0394794	170.03948	50
2 - Roof	109.98668	114.98668	20
3 - Wall Piers	337.5	337.5	0
4 - Spandrel Beams Below	600	600	0
5 - Spandrel Beams Above	125	125	0
6 - Cladding	71.17275828	71.172758	0

Element	Loading Type	Level Trib	L1 or A	L2	n	Area	Gravity Weight	Seismic Weight	% open	Gravity Weight	Seismic Weight	Diaphragm Wt		
			ft	ft		ft^2		psf		kip	kip	NS	EW	1926
Roof	**2**	**Roof**	198	253	1	50094	110	115.0	0%	5510	5760	Y	Y	Y
6th Floor	1	6th	198	253	1	50094	160	170.0	0%	8017	8518	Y	Y	Y
5th Floor	1	5th	198	253	1	50094	160	170.0	0%	8017	8518	Y	Y	Y
4th Floor	1	4th	198	253	1	50094	160	170.0	0%	8017	8518	Y	Y	Y
3rd Floor	1	3rd	198	253	1	50094	160	170.0	0%	8017	8518	Y	Y	Y
2nd Floor	1	2nd	198	253	1	50094	160	170.0	0%	8017	8518	Y	Y	Y
NS Wall Piers Roof	3	**Roof**	5.0	6.0	20	600	338	337.5	0%	203	203		Y	
NS Wall Piers 6th	3	6th	5.0	12.0	20	1200	338	337.5	0%	405	405		Y	
NS Wall Piers 5th	3	5th	7.0	12.0	20	1680	338	337.5	0%	567	567		Y	
NS Wall Piers 4th	3	4th	7.0	12.0	20	1680	338	337.5	0%	567	567		Y	
NS Wall Piers 3rd	3	3rd	9.0	12.0	20	2160	338	337.5	0%	729	729		Y	
NS Wall Piers 2nd	3	2nd	9.0	12.0	20	2160	338	337.5	0%	729	729		Y	
EW Wall Piers Roof	3	**Roof**	5.0	6.0	28	840	338	337.5	0%	284	284	Y		
EW Wall Piers 6th	3	6th	5.0	12.0	28	1680	338	337.5	0%	567	567	Y		
EW Wall Piers 5th	3	5th	7.0	12.0	28	2352	338	337.5	0%	794	794	Y		
EW Wall Piers 4th	3	4th	7.0	12.0	28	2352	338	337.5	0%	794	794	Y		
EW Wall Piers 3rd	3	3rd	9.0	12.0	28	3024	338	337.5	0%	1021	1021	Y		
EW Wall Piers 2nd	3	2nd	9.0	12.0	28	3024	338	337.5	0%	1021	1021	Y		
NS Top Spandrel Roof	5	**Roof**	198.0	3.5	2	1386	125	125.0	0%	173	173		Y	
NS Top Spandrel 6th	5	6th	198.0	3.5	2	1386	125	125.0	0%	173	173		Y	
NS Top Spandrel 5th	5	5th	198.0	3.5	2	1386	125	125.0	0%	173	173		Y	
NS Top Spandrel 4th	5	4th	198.0	3.5	2	1386	125	125.0	0%	173	173		Y	
NS Top Spandrel 3rd	5	3rd	198.0	3.5	2	1386	125	125.0	0%	173	173		Y	

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

CLIENT UCSF

SUBJECT Mission Center – Weight Takeoff

SHEET NO.

PROJECT NO. 197042.00

DATE 09/09/2019

BY SCD

CHECKED BY KDP

NS Top Spandrel 2nd	5	2nd	198.0	3.5	2	1386	125	125.0	0%	173	173		Y	
EW Top Spandrel Roof	5	**Roof**	253.0	3.5	2	1771	125	125.0	0%	221	221	Y		
EW Top Spandrel 6th	5	6th	253.0	3.5	2	1771	125	125.0	0%	221	221	Y		
EW Top Spandrel 5th	5	5th	253.0	3.5	2	1771	125	125.0	0%	221	221	Y		
EW Top Spandrel 4th	5	4th	253.0	3.5	2	1771	125	125.0	0%	221	221	Y		
EW Top Spandrel 3rd	5	3rd	253.0	3.5	2	1771	125	125.0	0%	221	221	Y		
EW Top Spandrel 2nd	5	2nd	253.0	3.5	2	1771	125	125.0	0%	221	221	Y		
NS Bot Spandrel Roof	4	**Roof**	198.0	1.5	1	297	600	600.0	0%	178	178		Y	
NS Bot Spandrel 6th	4	6th	198.0	1.5	2	594	600	600.0	0%	356	356		Y	
NS Bot Spandrel 5th	4	5th	198.0	1.5	2	594	600	600.0	0%	356	356		Y	
NS Bot Spandrel 4th	4	4th	198.0	3.0	2	1188	600	600.0	0%	713	713		Y	
NS Bot Spandrel 3rd	4	3rd	198.0	3.0	2	1188	600	600.0	0%	713	713		Y	
NS Bot Spandrel 2nd	4	2nd	198.0	3.0	2	1188	600	600.0	0%	713	713		Y	
EW Bot Spandrel Roof	4	**Roof**	253.0	1.5	1	380	600	600.0	0%	228	228	Y		
EW Bot Spandrel 6th	4	6th	253.0	1.5	2	759	600	600.0	0%	455	455	Y		
EW Bot Spandrel 5th	4	5th	253.0	1.5	2	759	600	600.0	0%	455	455	Y		
EW Bot Spandrel 4th	4	4th	253.0	3.0	2	1518	600	600.0	0%	911	911	Y		
EW Bot Spandrel 3rd	4	3rd	253.0	3.0	2	1518	600	600.0	0%	911	911	Y		
EW Bot Spandrel 2nd	4	2nd	253.0	3.0	2	1518	600	600.0	0%	911	911	Y		
NS Cladding	6	**Roof**	198.0	3.5	2	1386	71	71.2	0%	99	99		Y	Y
NS Cladding	6	6th	198.0	12.0	2	4752	71	71.2	0%	338	338		Y	Y
NS Cladding	6	5th	198.0	12.0	2	4752	71	71.2	0%	338	338		Y	Y
NS Cladding	6	4th	198.0	12.0	2	4752	71	71.2	0%	338	338		Y	Y
NS Cladding	6	3rd	198.0	12.0	2	4752	71	71.2	0%	338	338		Y	Y
NS Cladding	6	2nd	198.0	12.0	2	4752	71	71.2	0%	338	338		Y	Y
NS Cladding	6	Grnd	198.0	14.0	2	5544	71	71.2	0%	395	395		Y	Y
EW Cladding	6	**Roof**	253.0	3.5	2	1771	71	71.2	0%	126	126	Y		Y
EW Cladding	6	6th	253.0	12.0	2	6072	71	71.2	0%	432	432	Y		Y
EW Cladding	6	5th	253.0	12.0	2	6072	71	71.2	0%	432	432	Y		Y
EW Cladding	6	4th	253.0	12.0	2	6072	71	71.2	0%	432	432	Y		Y
EW Cladding	6	3rd	253.0	12.0	2	6072	71	71.2	0%	432	432	Y		Y
EW Cladding	6	2nd	253.0	12.0	2	6072	71	71.2	0%	432	432	Y		Y
EW Cladding	6	Grnd	253.0	14.0	2	7084	71	71.2	0%	504	504	Y		Y
									Sum	67517	70272	60819	57803	53325

SHEET NO.

PROJECT NO. 197042.00

DATE 09/09/2019

CLIENT UCSF

BY SCD

SUBJECT Mission Center – Weight Takeoff

CHECKED BY KDP

Level	Gravity Weight	Seismic Weight	NS Diaphragm	EW Diaphragm
	kip	kip	kip	kip
Roof	7021	7271	6619	6413
6th	10966	11467	10194	9791
5th	11355	11856	10421	9953
4th	12166	12667	10876	10309
3rd	12555	13056	11103	10471
2nd	12555	13056	11103	10471
Grnd	899	899	504	395
Total	67517	70272		

Total Seismic Weight = 70272 - Grnd Wt = 69373 kips

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

CLIENT	UCSF
SUBJECT	Mission Center - General building information
SHEET NO.	
PROJECT NO.	197042.00
DATE	06/10/2020
BY	AS
CHECKED	KDP

General Building Information			
	Value	Units	Reference Document
Total building height	74.0	ft	
Effective Seismic Weight	69373	kips	(seismic weight excluding ground level)
Compliance (per CBC)			2016 CBC 3412A.2.3
Structural Performance Level	S-5	BSE - C	2019 CBC Table 317.5
Non-structural	N-D		
Lateral System per ASCE 41	C2		
Risk Category	II		2016 CBC 1604.5 (Building not used for educational purposes)
$S_{CS,\ BSE\text{-}C}$	1.839	g	John Egan UCSF Group 2 Building Geotechnical Characteristics and Geohazards Report
$S_{C1,\ BSE\text{-}C}$	2.308	g	
Ts	1.255	s	
T0	0.251	s	
Site Class	E		
Ct	0.02		
beta	0.75		
height	74	ft	
Time Period T	0.505	s	
Sa	1.839	g	T0<T<Ts
C	1		ASCE 41-17, Table 4-7
Base Shear	127578	kips	Base Shear

Floor	Wi kip	$(hi)^k$ ft	$Wi\ (hi)^k$	Cvi	Fi kip	Vi kip	
Roof	7271	12.0	87256.3	0.10	12,965	12,965	1.58 (mass ratio)
6th	11467	12.0	137601.1	0.16	20,446	33,411	
5th	11856	12.0	142266.7	0.17	21,139	54,551	
4th	12667	12.0	152008.3	0.18	22,587	77,137	
3rd	13056	12.0	156673.9	0.18	23,280	100,418	
2nd	13056	14.0	182786.2	0.21	27,160	127,578	
Grnd							
Sum	69373.3		858592.4	1.00	127,578		

*K = 1 for 6 stories or lower per 4.4.2.2

Shear Stress in Shear Walls

per ASCE 41-17 4.4.3.3

Ms 4.5

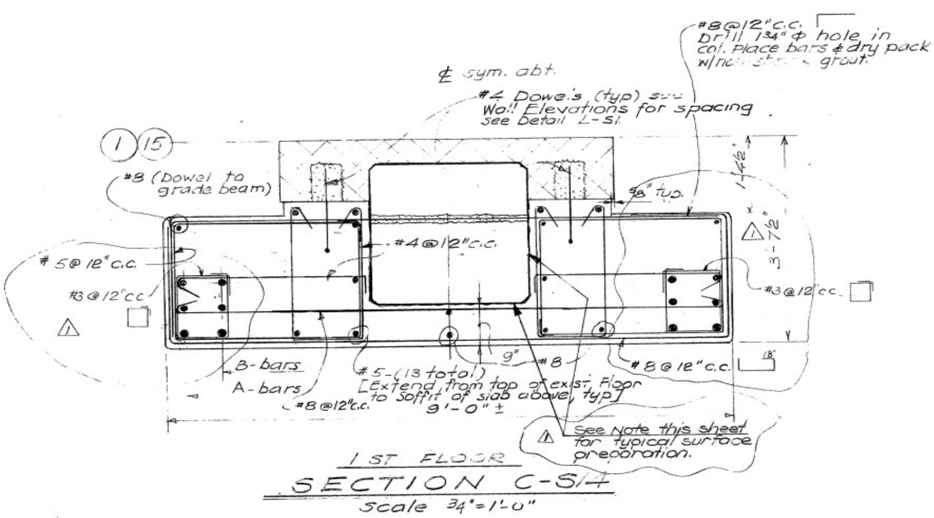
Allowable Shear stress (ksi) = max(0.1 ksi, 2sqrt(f'c)) = 0.126 ksi

Floor above represented wall	Story Shear at level j (V_j)	N-S Loading Shear Area	N-S Loading v_j^{avg}	E-W Loading Shear Area	E-W Loading v_j^{avg}			
	kips	ft^2	ksi	ft^2	ksi			
Roof	12,965	108.2	0.18	70.8	0.28	NC		
6th	33,411	157.0	0.33	131.5	0.39	NC		
5th	54,551	406.8	0.21	309.3	0.27	NC		
4th	77,137	421.6	0.28	392.3	0.30	NC		
3rd	100,418	458.9	0.34	439.6	0.35	NC	max DCR	
2nd	127,578	547.1	0.36	444.1	0.44	NC		3.51

NC - Noncompliant

Minimum reinforcement check

Typical section 1st floor



Longitudinal Steel -

Total Area (Ac) =	20.25	ft^2
(2)#8	1.58	
(6) A (#11)	9.36	
(6) B (#11)	9.36	
(9)#5	2.79	
(4) #8	3.16	
Total longitudinal Steel Area (As) =	26.25	sq.in

rho (1st floor) = 0.009 > 0.0012

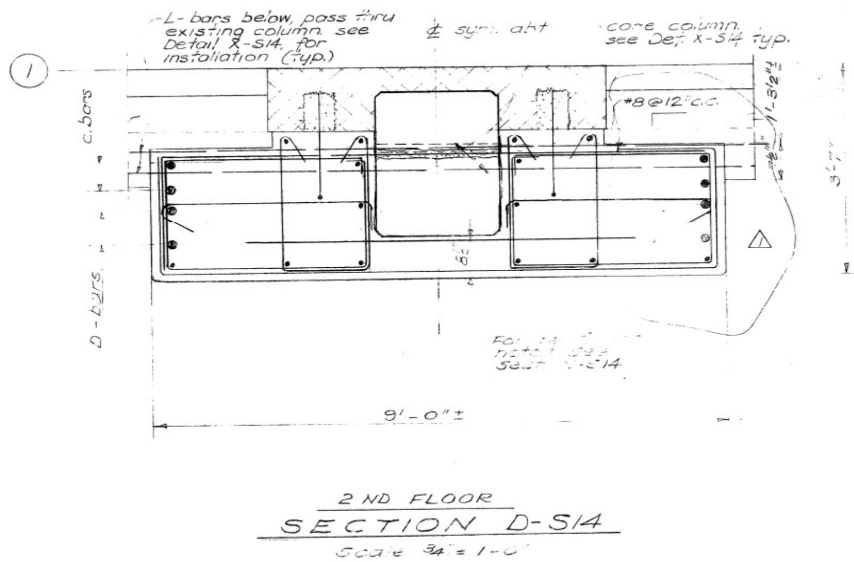
Horizontal Steel -

Av = (2) #8 @ 12" OC (full depth bars)
= 1.58 sq.in

rho,v (1st floor) = As / (thk wall * spacing)
= 1.58 / ((2.25*12) * 12)

rho,v (1st floor) = 0.0049 > 0.002

Typical section 2nd floor



Longitudinal Steel -		
Total Area (Ac) =	20.25	ft^2
(2)#8	1.58	
(4) C (#11)	6.24	
(4) D (#11)	6.24	
(14)#5	4.34	
Total Steel Area (As) =	18.4	sq.in
rho (1st floor) =	0.006	> 0.0012

Horizontal Steel -

Av = (1) #8 @ 12" OC & (1)#6 @ 12

= 0.79 + 0.44 sq.in (full depth bars)

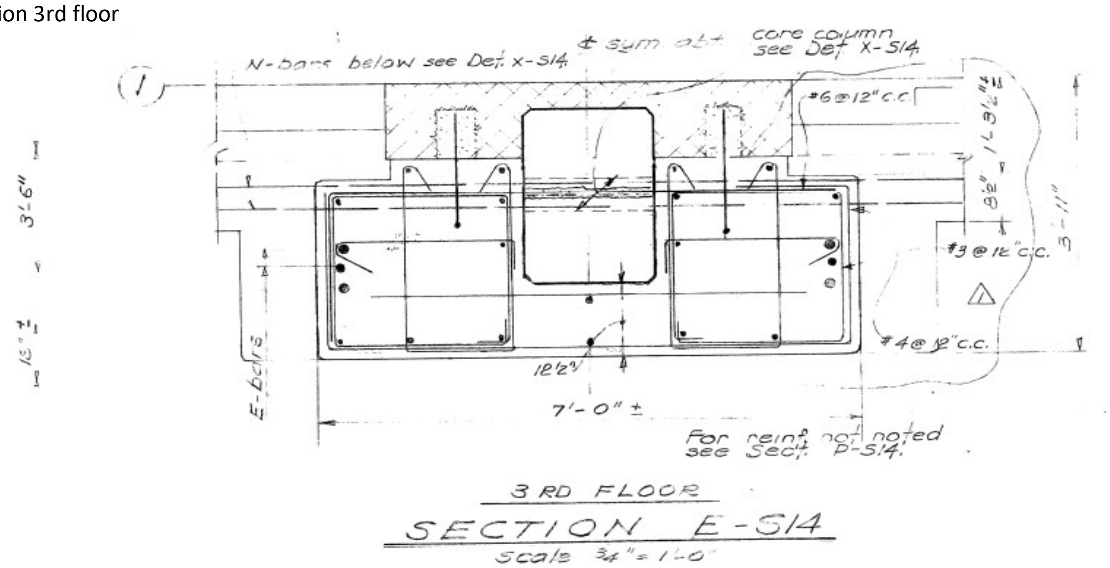
= 1.23 sq.in

rho,v (1st floor) = As / (thk wall * spacing)

= 1.23 / ((2.25*12) * 12)

rho,v (2st floor) = 0.0038 > 0.002

Typical section 3rd floor



Longitudinal Steel -		
Total Area (Ac) =	18.375	ft^2
(4)#8	1.58	
(4) E (#11)	6.24	
(14)#5	4.34	
Total Steel Area (As) =	12.16	sq.in
rho (1st floor) =	0.005	> 0.0012

Horizontal Steel -

Av = (1) #6 @ 12" OC & (1)#5 @ 12

= 0.44 + 0.31 sq.in (full depth bars)

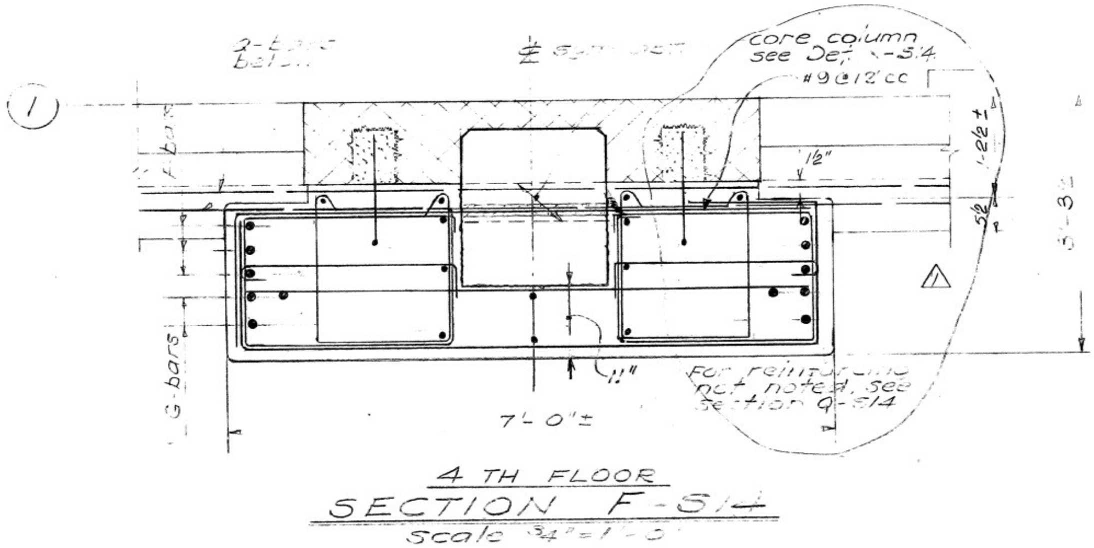
= 0.75 sq.in

rho,v (1st floor) = As / (thk wall * spacing)

= 0.75 / ((31.5) * 12)

rho,v (2st floor) = 0.0020 = 0.002

Typical section 4rd floor



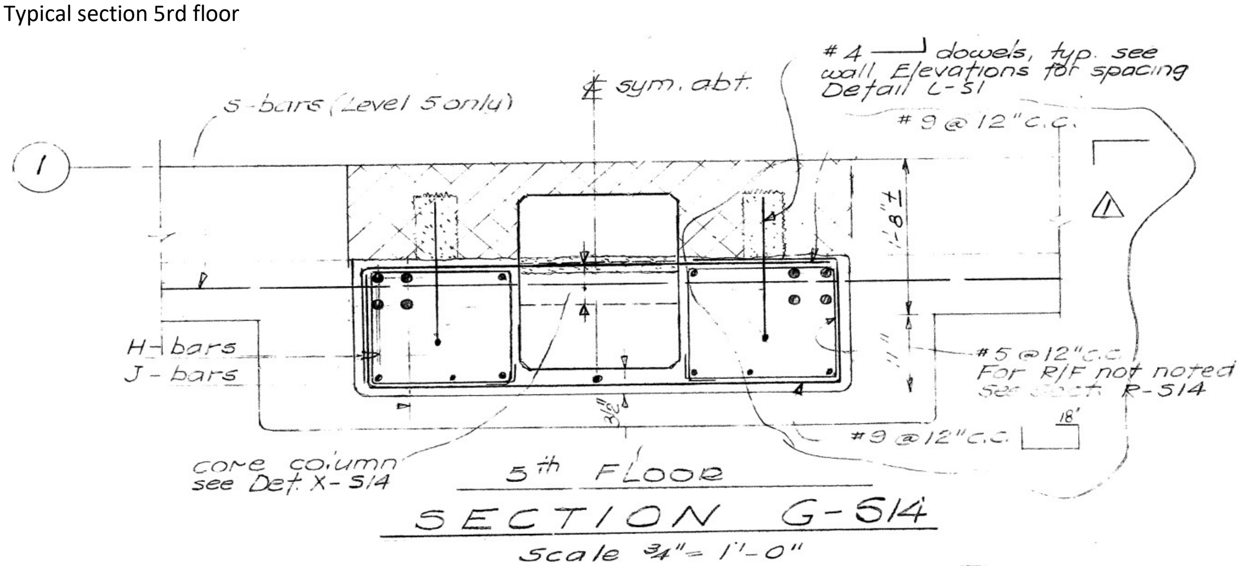
Longitudinal Steel -

Total Area (Ac) =	14.58	ft^2
(2)#9	2	
(6) F (#11)	9.36	
(4) G (#11)	6.24	
(10)#5	4.34	
Total Steel Area (As) =	21.94	sq.in
rho (1st floor) =	0.010	> 0.0012

Horizontal Steel -

Av = (1) #9 @ 12" OC & (1)#6 @ 12
= 1.00 + 0.44 sq.in (full depth bars)
= 1.44 sq.in
rho,v (1st floor) = As / (thk wall * spacing)
= 1.44 / (25 * 12)
rho,v (2st floor) = 0.0048 = 0.002

Typical section 5rd floor



Longitudinal Steel -

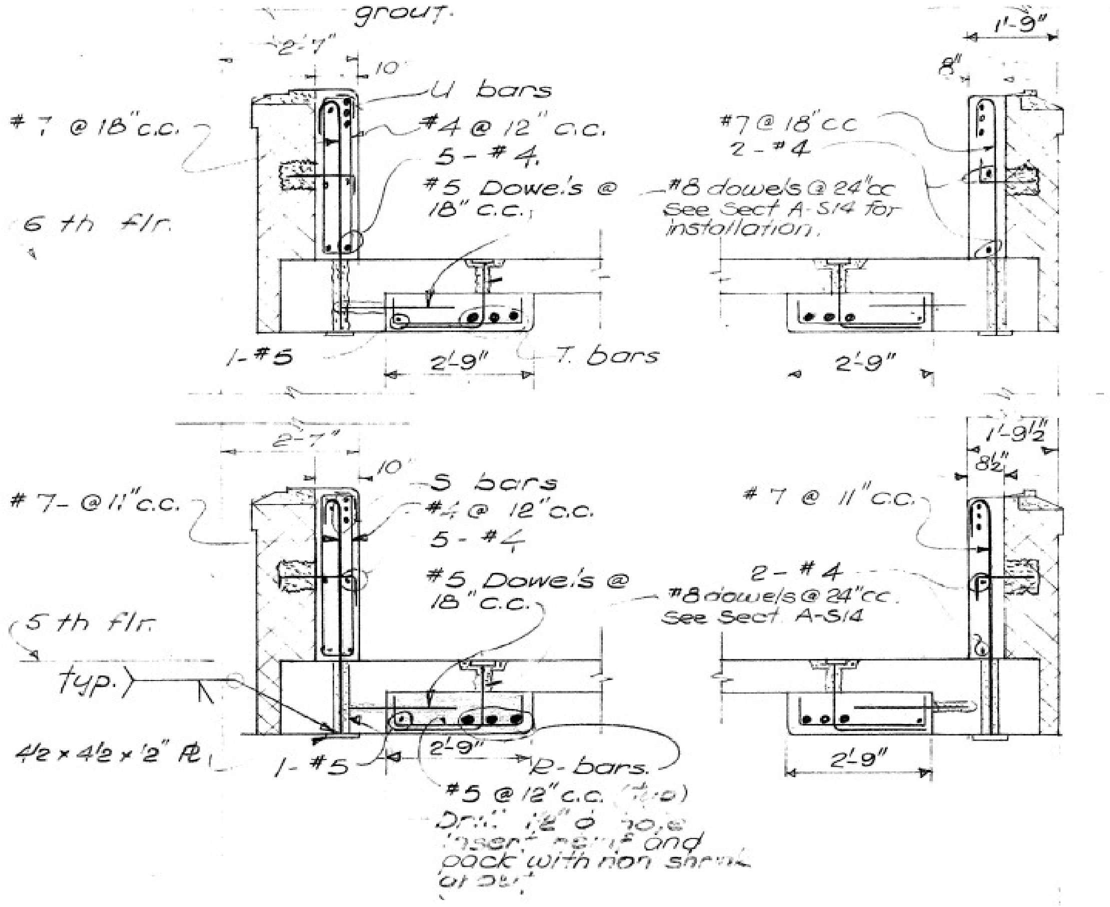
Total Area (Ac) =	14.58	ft^2
(4) H (#8)	3.16	
(4) J (#8)	3.16	
(9)#5	4.34	
Total Steel Area (As) =	10.66	sq.in
rho (1st floor) =	0.005	> 0.0012

Horizontal Steel -

Av = (1) #9 @ 12" OC
= 1.00 sq.in (full depth bars)

rho,v (1st floor) = As / (thk wall * spacing)
= 1.0 / (25 * 12)
rho,v (2st floor) = 0.0033 = 0.002

Slab Connection to Wall TYP:

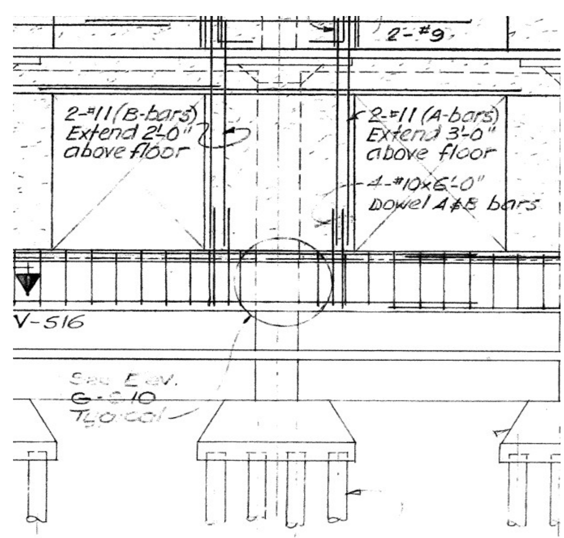


Wall to foundation connection:

A typical detail on right shows that the (2) #11 bars in the wall are followed by (4) #10 dowels

Area of (2)#11 bars =	3.12 in^2
Area of (4)#10 bars =	5.08 in^2

Hence OK



The overturning on the EW wall will be critical due to smaller span

Total Shear at each level on EW wall:

Level	Height	Shear Force	Moment
	ft	kip	kip-ft
Roof	74	12,965	479717.9
6th	62	20,446	633827.1
5th	50	21,139	528482.4
4th	38	22,587	429149
3rd	26	23,280	302640.6
2nd	14	27,160	190120.4
Grnd	0	0	0
		sum M =	2563937 kip-ft
		length of wall =	189.8333 ft

Deal load in the wall:

Trib width = 7.75 ft (GL 1)

UDL (DL+SDL):

	UDL(psf)	Total wt (k)
Roof	115.0	169.2
6th	170.0	250.2
5th	170.0	250.2
4th	170.0	250.2
3rd	170.0	250.2
2nd	170.0	250.2
Grnd	170.0	250.2
		1670

wall/cladding = 6235 kip

Dead load per unit length = 41.6 kip/ft

Total Dead load on the corner pile cap = 41.6 * 224/2/12

= 389 kips

Uplift due to overturning = 4277 kips

Net Uplift = 3889 kips

SIMPSON GUMPERTZ & HEGER

Engineering of Structures
and Building Enclosures

CLIENT UCSF

SUBJECT Mission Center Building - Pile Cap

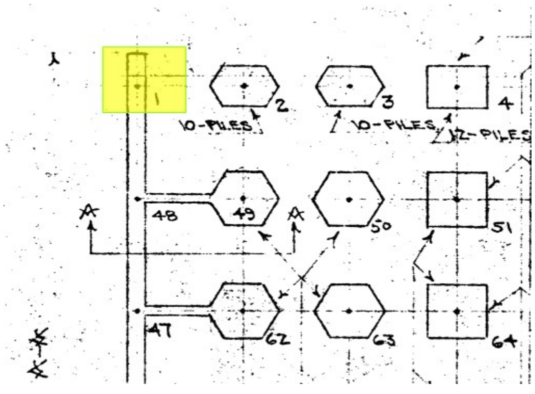
SHEET NO.

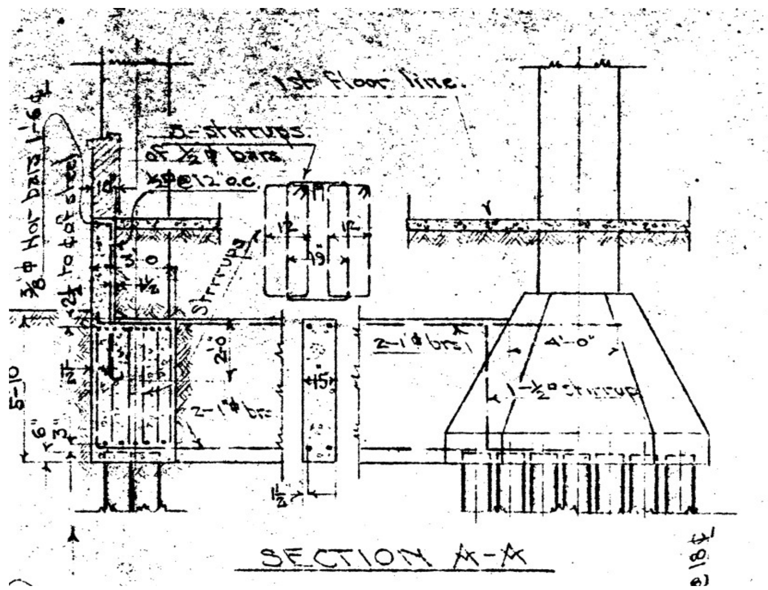
PROJECT NO. 197042.00

DATE 06/11/2020

BY AS

CHECKED KDP





Overturning Check (A6.2.1)

Least Dimension =	189.8333	ft
Height =	74	ft
ratio =	2.6	
0.6 Sa =	1.10	C

SIMPSON GUMPERTZ & HEGER
Engineering of Structures
and Building Enclosures

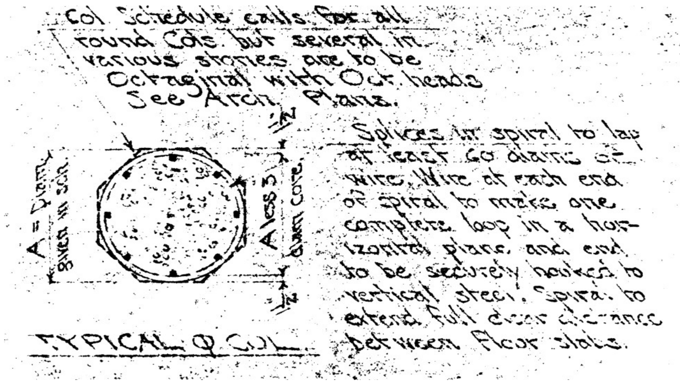
SHEET NO.	
PROJECT NO.	197042.00
DATE	06/11/2020
BY	AS
CHECKED	KDP

CLIENT UCSF

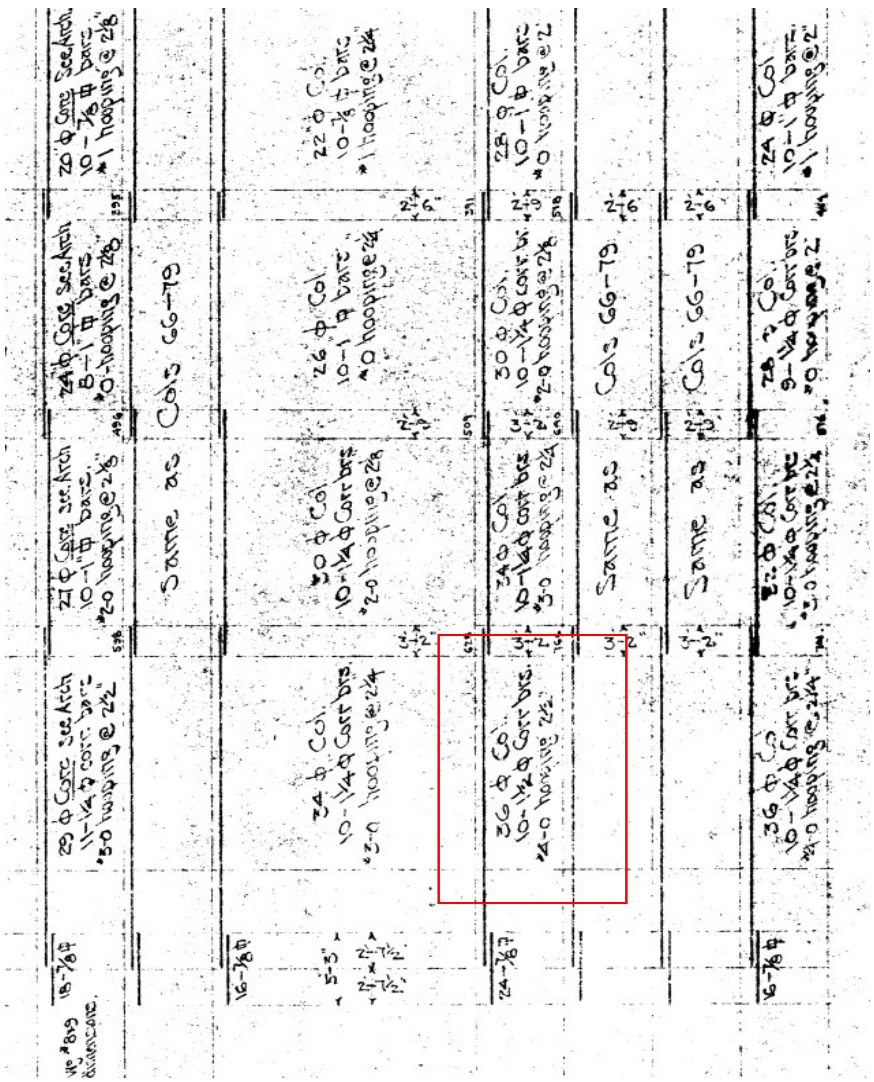
SUBJECT Mission Center Building - Secondary Elements

Deflection Compatibility of Secondary Component

Typical column section:



Column section below shows hoopties



Column Capacity Check (In red box):

fye =	75			
f'ce =	6000		psi	
dc =	36		in	
Ac =	1018		in^2	
P =	1700	0	kip	
Mcap max =	2240	1425	k-ft	(from SPColumn)
L =	156.5		in	(floor height - drop cap)
2Mp/L =	344	219	kip	

Shear capacity:	4-0 @ 2.5 in spacing Spiral			
Av =	0.166		in^2	
s =	2.5		in	
Ace =	804.2		in^2	core only
vc =	2*sqrt(f'c)	2*sqrt(f'c)* (1 + N/2000Ag)		
=	284	155	Kip	
Vc =	229	125		
Vs =	159.36		kip	
Vn =	Vc + Vs			
=	388	284	kip	
DCR =	0.89	0.77		

PM Curve -