UCSF

Rating form completed by:

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STRUCTURAL
STATE OF CALIFORNIA

October 4, 2019

UCSF Building Seismic Ratings

Koret Vision Research

CAAN #2325

10 Koret Way, San Francisco UCSF

Campus: Parnassus

Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	IV	Findings based on drawing review and ASCE 41-17 Tier 1 evaluation[1]
Rating basis	Tier 1	Field visit by CC Thiel and GS Varum on June 6, 2019
Date of rating basis	2019	
Recommended list UCSF priority category for retrofit	N/A	N/A
Ballpark total project cost to retrofit to IV rating	N/A	N/A
Is 2018-2019 rating required by UCOP?	Yes	N/A
Further evaluation recommended?	No	N/A

Building information used in this evaluation

- Forell/Elsesser, 1985. *Original* structural drawings, (22 sheets), and associated architectural drawings by Ripley Associates.
- Impel Corp., 1989. *Performance of UCSF Buildings During the October 17, 1989 Loma Prieta Earthquake*, (50 pages), dated November 17, 1989. NB: Report does not list Koret Building by name or address.
- McGinnis Chen, 1993. UCSF Medical Center, Koret Center, Investigation Report & Building Exterior Evaluation Report, 1993, (38 pages) *Note: This was after the 1989 Loma Prieta earthquake and prior to the 1994 Northridge earthquake*.
- Degenkolb, 2013. *Back-Up* structural drawings sheets, and associated mechanical drawings by Cammisa+Wipf, architectural drawings by Oculus (8 sheets).
- Degenkolb, 2014. *Back-Up Generator* structural drawings, and associated mechanical drawings by Cammisa+Wipf (5 sheets).
- Martin and Martin, 2015. *Fall Protection Upgrades* structural drawings, (6 sheets).
- Rutherford & Chekene, 2006. *Slope Stability Risk Assessment University of California San Francisco San Francisco, California*. (2019 assessment update presented to the SRC, report in progress of being issued).

Additional building information known to exist

None pertinent to seismic evaluation specific to the building.

Scope for completing this form

Structural drawings for original construction and modifications were reviewed and an ASCE 41-17 Tier 1 evaluation was performed.

Brief description of structure

The UCSF Koret Vision Center building is located on the west side of Koret way on the Parnassus Campus of the University of California at San Francisco. It was designed in 1985 by Ripley Architects and Forell/Elsesser Engineers. Reviewed plans indicate that the structural design followed the provisions of the 1982 edition of the Uniform Building Code. The building has an area of approximately 143,108 square feet. There have been limited modifications for back-up power, and addition of balustrades to minimize falling risks. The building is a 3-story with partial basement structure irregularly shaped in plan constructed on a sloped site, see 3rd floor plate and cross-sections. The majority of the superstructure includes a fully distributed moment-resisting steel frame, and the relatively small portion of the structure between grid lines 11 and 19 includes full-length perimeter moment-

[1] The evaluations at UCSF translate the Tier 1 evaluation to a Seismic Performance Level rating using professional judgment discussed among the UCSF 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.

resisting frames. The welded steel moment frame (WSMF) portion of structure is partially constructed over a reinforced concrete basement with perimeter reinforced concrete walls. The upper three-story longitudinal section is a fully-distributed frame, with 3 bays by 11 bays on the long arm, See Figure 2. The frames are detailed in a pre-Northridge manner typical of the time, see Figure 3. The basement is of massive reinforced construction. The building is sufficiently separated from adjacent buildings that they are not expected to be subjected to pounding and possible damage from their interaction.

Vertical Load-Resisting System: The roof and the third and second floors include corrugated light-gage steel deck with concrete topping that is supported on steel wide-flange beams and girders. The roof and floor beams are supported on steel wide-flange girders and columns. The roof and floor girders are supported on steel wide-flange columns. The first floor is a reinforced concrete slab supported on reinforced concrete beams, girders and perimeter walls. The first-floor beams are supported on reinforced concrete girders, columns and perimeter walls.

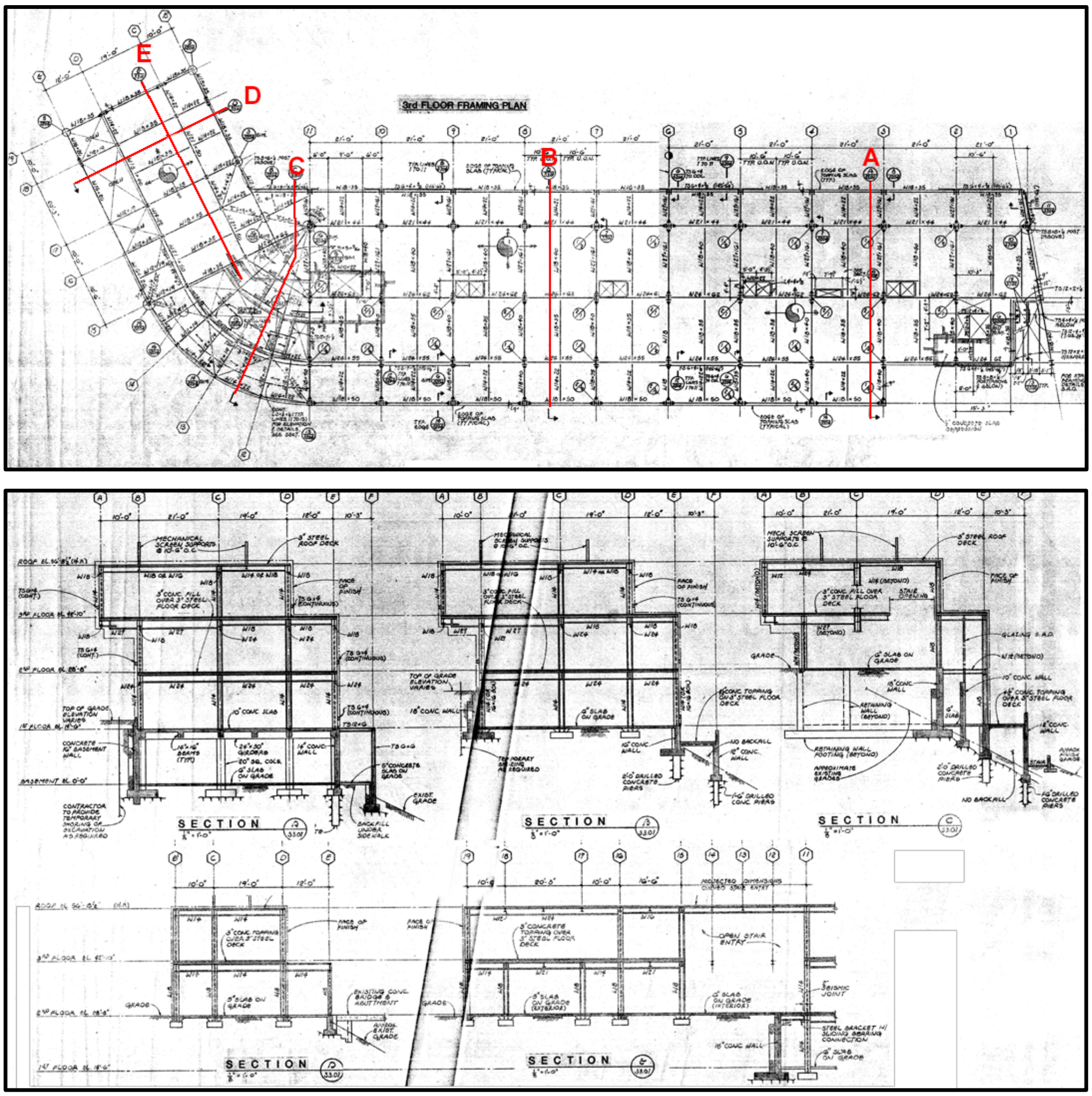
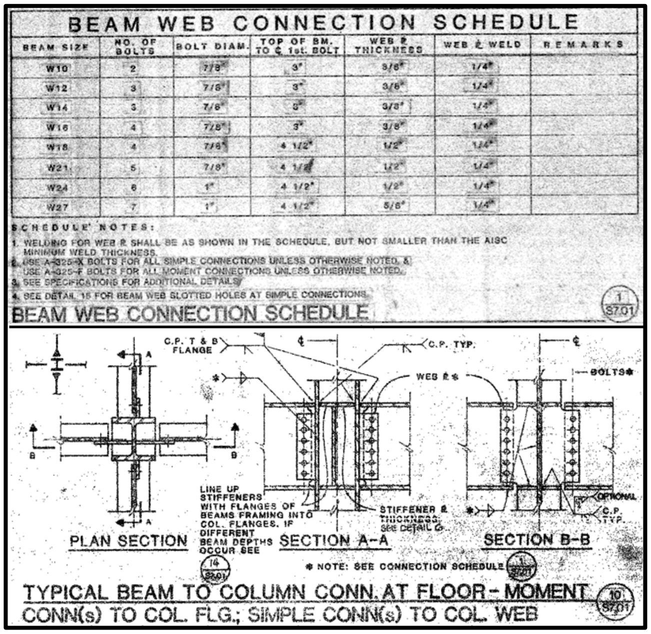


Figure 2. *The upper image shows the third floor framing plan showing the steel framing. The red lines and letters indicate the cross-sections of the lower image.*



BEAM WEB CONNECTION SCHEDULE

BEAM SIZE	NO. OF BOLTS	BOLT DIAM.	TOP OF BM. TO ℄ 1st. BOLT	WEB ℞ THICKNESS	WEB ℞ WELD	REMARKS
W10	2	7/8"	3"	3/8"	1/4"	
W12	3	7/8"	3"	3/8"	1/4"	
W14	3	7/8"	3"	3/8"	1/4"	
W16	4	7/8"	3"	3/8"	1/4"	
W18	4	7/8"	4 1/2"	1/2"	1/4"	
W21	5	7/8"	4 1/2"	1/2"	1/4"	
W24	6	1"	4 1/2"	1/2"	1/4"	
W27	7	1"	4 1/2"	5/8"	1/4"	

SCHEDULE NOTES:

1. WELDING FOR WEB ℞ SHALL BE AS SHOWN IN THE SCHEDULE, BUT NOT SMALLER THAN THE AISC MINIMUM WELD THICKNESS.
2. USE A-325-X BOLTS FOR ALL SIMPLE CONNECTIONS UNLESS OTHERWISE NOTED, & USE A-325-F BOLTS FOR ALL MOMENT CONNECTIONS UNLESS OTHERWISE NOTED.
3. SEE SPECIFICATIONS FOR ADDITIONAL DETAILS.
4. SEE DETAIL 15 FOR BEAM WEB SLOTTED HOLES AT SIMPLE CONNECTIONS.

BEAM WEB CONNECTION SCHEDULE

Figure 3. *Typical details for the WSMF connections. They are typical of pre-Northridge practice. They were not observed but typical experience at this time indicates that the run-off tabs and backing bars used in welding the joint are still in place.*

The first-floor girders are supported on the interior reinforced concrete columns and perimeter reinforced concrete walls.

Foundation System: Foundation support is provided by shallow reinforced concrete spread footings and grade beams. Additional support is provided by drilled reinforced concrete piers. The basement floor and the first floor of the portion of the building between grid lines 5-19 is a concrete slab-on-grade reinforced with #4 bars at 12 inches on center each way.

Lateral Load-Resisting System: The composite concrete roof and the third and second floor diaphragms distribute the earthquake loads among the elements of the building's moment-resisting steel frame (WSMF). The steel frame of the portion of the building between grid lines 1 and 11 is fully distributed, with all columns being part of the moment-resisting space frame. The portion of the building between grid lines 11 and 19 includes moment-resisting frames along the perimeter of this building section. The moment-resisting frame between grid lines 1 and 5 transfers the earthquake loads to the reinforced concrete first floor diaphragm, which transfers them to the perimeter reinforced concrete shear walls of the basement.

Past seismic performance: The building was in place at the time of the 1989 Loma Prieta earthquake. The Impel Corporation 1989 report did not cover this specific building in its review of UCSF building performance, but did have access to a report on waterproofing issues from 1993.

The building's principal steel lateral load-resisting system is a pre-Northridge WSMF, see Figure 3. The PGA at the site in 1989 Loma Prieta earthquake was measured as 0.09*g* horizontal peak acceleration by instruments in the adjacent UCSF Nursing Building. The SAC WSMF inspection for WSMF buildings experiencing earthquakes includes three criteria: a magnitude threshold, a specified measured or estimated PGA at the building site, and physical observations of building's impacts as the means of determining whether a building frame's steel joints should be inspected for possible damage, [FEMA, 1996]. The magnitude threshold by the site PGA was more than 50% lower than the trigger point. We examined the condition of the concrete, and gypsum board and found no indications in protected areas (from use and maintenance) of any signs of cracking, whether present or repaired, that would indicate interstory displacements sufficient to suggest that inspection of welded joints is warranted. Telesis has discovered a number of previously unidentified damaged WSMF buildings in the Bay Area by such observations, but they were all at this distance from the epicenter located on soft soil sites. The site here is stiff. The McGinnis Chen 1993 report concentrated on water proofing issues in the building's exterior, which were resolved, and apparently not related to the 1989 seismic response of the building. The building is not subject to inspection for WSMF connection integrity by the SAC criteria.

Non-structural systems: 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	**Hazard”**	**UCOP non-structural checklist item**	**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	None	Masonry chimneys	None
Unbraced masonry parapets, cornices or other ornamentation above exit ways and public access areas	None	Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.	Mostly braced others identified to manager.

Liquefaction hazard: The site is evaluated as not subject to liquefaction hazard.

Landslide hazard: The Parnassus campus is located on relatively steep slope site, and could be considered to be landslide prone. A recent presentation by Gyimah Kasali of Rutherford & Chekene reports that if utility lines are adequately maintained then landsliding should not be an threat, either from hydraulic or seismic events to the Koret Building. The Carnage Commission report on the 1906 earthquake was explicit in reporting no observed landslides that were triggered by the 1906 San Francisco earthquake in this extended region. This is definitive in Telesis’ judgement that seismically-induced landsliding does not pose an appreciable hazard to this building.

Brief description of seismic deficiencies and expected seismic performance

The reviewed drawings indicate great attention paid by the structural engineer to the seismic performance of this relatively complicated structure constructed on a steep site. The building has both favorable and unfavorable features that influence its seismic behavior:

Favorable:

- The pre-Northridge WSMF is evaluated, following the SAC criteria, not to be suspected of suffering damage in the 1989 Loma Prieta earthquake.
- The building design shows attention to detailing in excess of common practice at the time of design.
- There are no identified site response failure mechanisms evaluated as expectable.
- The building was observed to be in excellent condition structurally and has not been modified structurally to decrease its seismic response reliability.
- No non-structural life-safety concerns were observed, including at exit routes. [2]

Unfavorable:

- The pre-Northridge WSMF frame joints are recognized as prone to failure, and not allowed for new construction starting with CBC 1997.
- The building is constructed on a sloped site in two directions.
- ASCE 41 Tier 1 non-conformance issues are evaluated as leading to the potential for structural instability.

[2] For these Tier 1 evaluations, we did not visit all spaces of the building; we rely on campus staff to report to us their understanding of if and where non-structural hazards may occur.

Structural deficiency	Affects rating? (Y/N)	Structural deficiency	Affects rating? (Y/N)
Discontinuous Shear Walls	N	Quick Shear Stress Check	N
Wall Aspect Ratios	N	Hillside Site	N

Structural deficiency	Affects rating? (Y/N)	Structural deficiency	Affects rating? (Y/N)
Lateral system stress check (wall shear, column shear or flexure, or brace axial as applicable)	N	Openings at shear walls (concrete or masonry)	N
Load path	N	Liquefaction	N
Adjacent buildings	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		

Koret (transverse direction)

Story	Story Shear (k)	Number of bays	Story drift	Column axial stress check	Flexural stress check	Pass? (Y/N)
3rd	70	2	0.017 < 0.030	0.37 ksi < $0.10F_y$	19.6 ksi < F_y	Y
2nd	195	3	0.005 < 0.030	1.24 ksi < $0.10F_y$	33.5 ksi < F_y	Y
1st	82	3	0.005 < 0.030	2.14 ksi < $0.10F_y$	29.5 ksi < F_y	Y

These lead to the conclusion that the Koret Vision Center building is expected to exhibit good performance when subject to strong earthquake site ground motions, consistent with Level IV in the UC lexicon.

Stability: It is expected that the Koret Vision Center building will remain stable under earthquake loads specified in the 2016 edition of the California Existing Building Code for the subject building type and site.

Expected Damage: At relatively low to moderate level site ground motions damage to the Koret Vision Center building is expected to be limited to non-structural elements, such as ceilings, partitions, ornamentation, building's equipment, etc. At site ground motions approaching or exceeding code-level values, damage to the building may be more substantial, including some structural damage. At these levels, it is possible that there will be damage to the WSMF joints, including severing connections and elements. It is expected that the damage at code-level or higher site ground motions may not exceed economically repairable limits.

Recommendations for further evaluation or retrofit

No further evaluation or retrofit is recommended.

Peer review comments on rating

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

Additional building data	Entry	Notes
Latitude	-122.46015 °	Reported by John Egan
Longitude	-122.46015 °	Reported by John Egan
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	3	
Number of stories (basements) below lowest perimeter grade	1	
Building occupiable area (OGSF)	43,108	
Risk Category per 2016 CBC 1604.5	II	
Building structural height, h_n	55 ft	
Coefficient for period, C_t	0.035	Per ASCE 41-17 equation 7-18
Coefficient for period, β	0.80	Per ASCE 41-17 equation 7-18
Estimated fundamental period	0.70 sec	Per ASCE 41-17 equation 7-18
Site data		
975 yr. hazard parameters S_s, S_1	1.554, 0.614	Report by John Egan
Site class	C	
Site class basis		Report by John Egan
Site parameters F_a, F_v	1.2, 1.4	Report by John Egan
Ground motion parameters S_{cs}, S_{c1}	1.865, 0.895	Report by John Egan
S_a at building period	1.280	
Site V_{s30}	570m/s	Report by John Egan
V_{s30} basis		Report by John Egan
Liquefaction potential	None	Report by John Egan
Liquefaction assessment basis	Assessment	Report by John Egan
Landslide potential	No	
Landslide assessment basis	Assessment	Report by John Egan
Active fault-rupture hazard identified at site?	No	Report by John Egan
Site-specific ground motion study?	No	
Applicable code		
Applicable code or approx. date of original construction	1982 UBC	Code identified on Forell/Elsesser Original design documents
Applicable code for partial retrofit	NA	
Applicable code for full retrofit	NA	
FEMA P-154 data		
Model building type North-South	S1	Steel moment- resisting frame
Model building type East-West	S1	Steel moment- resisting frame

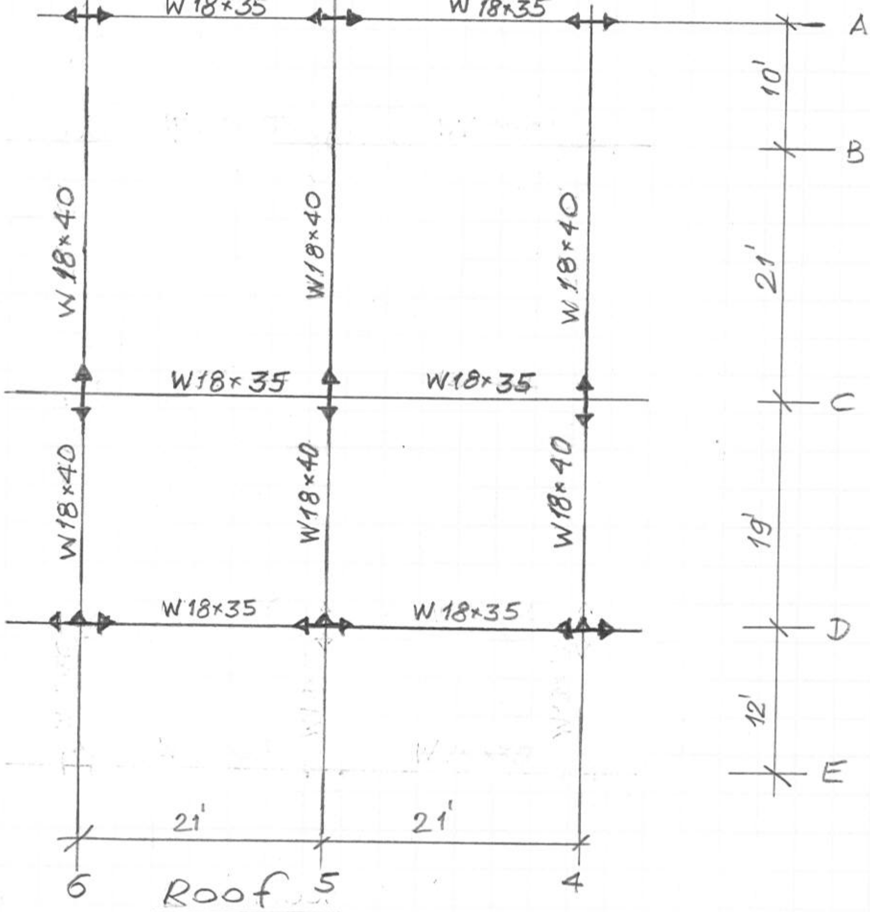
FEMA P-154 score	N/A	Not included here because an ASCE 41-17 Tier 1 evaluation was conducted.
Previous ratings		
Most recent rating	IV	2013 UCSF SRC Rating
Date of most recent rating	10/7/2013	
2nd most recent rating	NA	
Date of 2nd most recent rating		
3rd most recent rating	NA	
Date of 3rd most recent rating		
Appendices		
ASCE 41 Tier 1 checklist included here?	Yes	Refer to attached checklist file

Limitations:

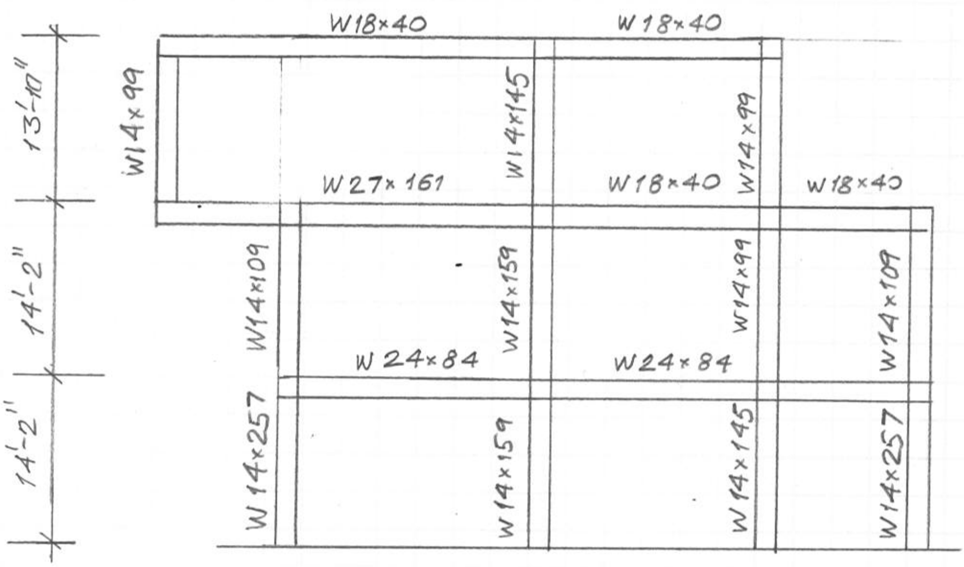
The purpose of this report is to evaluate the expected damageability performance of the building and potential hazards of the site. Telesis performed an estimate of damageability to Koret Vision Research from earthquakes in conformance with the scope and requirements for Building Damageability at ASTM Level 1 by Senior Field Assessors CC Thiel and GS Varum of Standard Practice Probable Maximum Loss (PML) Evaluations for Earthquake Due-Diligence Assessments [ASTM E2557-16a] for the property located at 10 Koret Way, San Francisco, California. The assessment was performed and reported in a format required by the UC consistent with an ASCE 41-17 Level 1 assessment that does not include many ASTM reporting requirements.

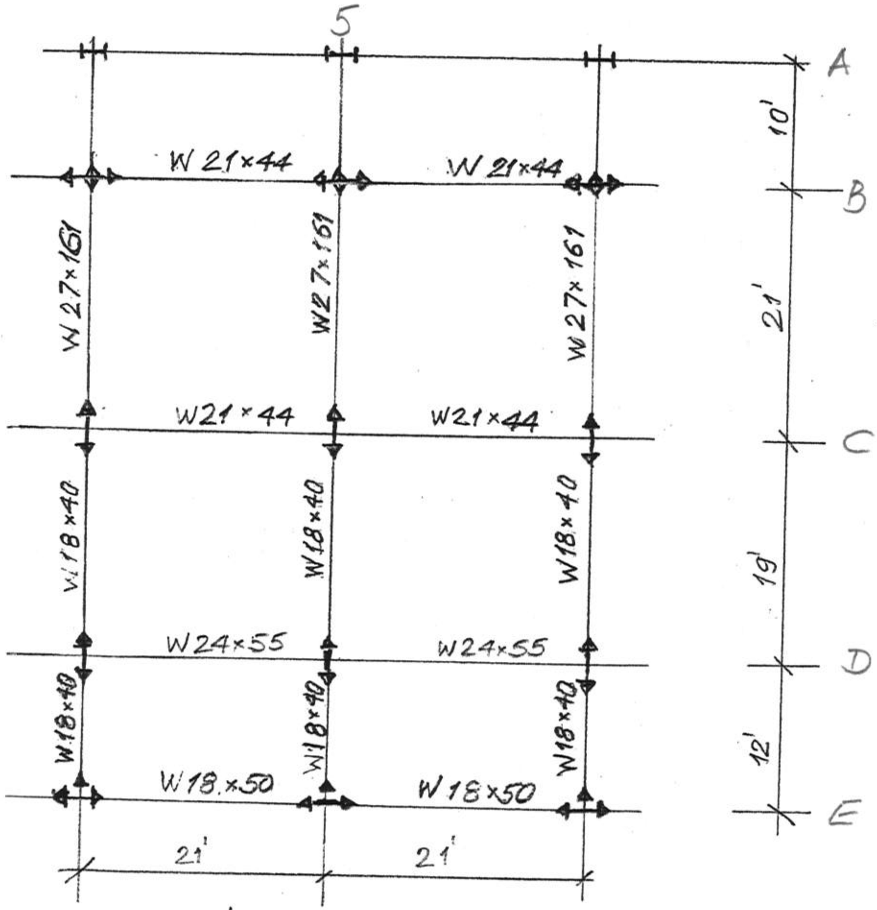
This report is for the exclusive use of the University of California, its assigns and successors, and no other party shall have any right to rely on any service provided by Telesis without prior written consent.

Services were performed by Telesis in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions. No other warranty, expressed or implied, is made. This report is based on a limited review of the building's available design documents. Biuldoing permit drawings, shop drawings, construction testing reports, computations and assumptions that would have been useful in the analysis were not available. Further, the actual seismic resistance characteristics of the building could not be fully assessed since architectural finishes did not allow detailed inspection of the quality of construction. Information not available under these conditions to Telesis and hidden construction quality conditions could alter the expected seismic vulnerability of the building from those assumed in this report. The assessment of earthquake performance and the assignment of a Level estimation process reflects uncertainty in both the seismic environment and the buildings' performance. There is no assurance that damage observed to the buildings in a future earthquake will be less than the estimates given.


W18×35
W18×35
A
10'
B
W18×40
W18×40
W18×40
21'
W18×35
W18×35
C
W18×40
W18×40
W18×40
19'
W18×35
W18×35
D
12'
E
21'
21'
6
5
4

Roof


W18×40
W18×40
13'-10"
W14×99
W14×145
W14×99
W27×161
W18×40
W18×40
14'-2"
W14×109
W14×159
W14×99
W14×109
W24×84
W24×84
14'-2"
W14×257
W14×159
W14×145
W14×257



$V = C S_a W$ $\quad S_S = 1.580 \quad S_1 = 0.639 \quad F_a = 1.200 \quad F_v = 1.400$

$S_{2NS} = 1.896 \quad S_{2N1} = 0.895$

$T = 0.035 \times (42.2)^{0.80} = 0.699$ sec.

$C = 1.0$

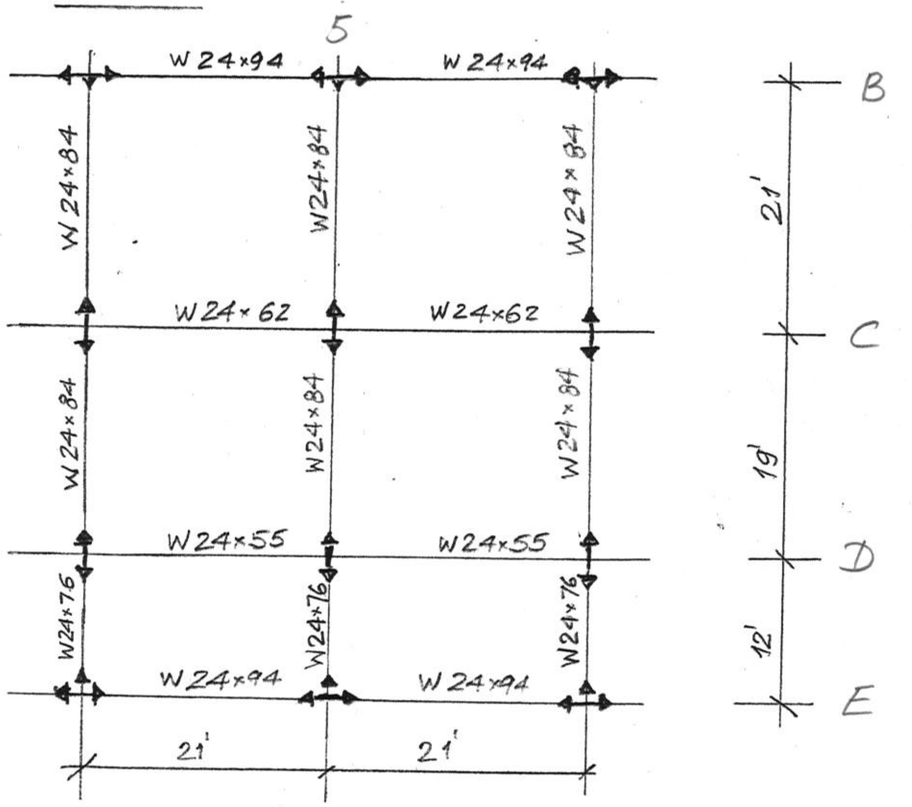
$S_{x1} = 1.400 \times 0.639 = 0.895$

$S_a = \dfrac{0.895}{0.699} = 1.280 < S_{xs} = F_a\, S_s = 1.200 \times 1.580 = 1.896$

<u>Use</u> $S_a = 1.280$

$V = 1.0 \times 1.280 \times 270.9 = 347^k$

Koret



2nd Floor

Bay width 21 ft.

Roof load (incl. partitions & walls) 30 psf

Floor load (incl. partitions & walls) 100 psf

$W_R = 30 \times 21 \times 50 = 31.5^k$

$W_3 = 100 \times 21 \times 62 = 130.2^k$

$W_2 = 100 \times 21 \times 52 = 109.2^k$

Level	W	h	Wh	$\frac{Wh}{\Sigma Wh}$	F_x'
R	31.5	42.2	1329	0.202	70.0
3	130.2	28.4	3698	0.562	195.0
2	109.2	14.2	1551	0.236	82.0
Total	270.9	14.2	6578	1.000	347.0

Story drift

$$D_r = \left(\frac{k_b + k_c}{k_b\, k_c}\right)\left(\frac{h}{12E}\right)V_c$$

3rd story

$$k_b = \frac{612}{50\times12} = 1.02 \qquad k_c = \frac{1710}{13\times12} = 10.96$$

$V = 70 \times 0.5 = 35^k$

$E = 29\times10^3\ ^k/_{in^2}$. $\qquad h = 156$ in.

$$D_r = \frac{1.02+10.96}{1.02\times10.96} \times \frac{156}{12\times29\times10^3}\times35 = 0.017$$

2nd story

$$k_b = \frac{6280}{21\times12} = 24.92 \qquad k_c = \frac{1900}{12\times12} = 13.19$$

$V = (70+195)\times0.4 = 106.^k$

$h = 144$ in.

$$D_r = \frac{24.92+13.19}{24.92\times13.19} \times \frac{144}{12\times29\times10^3}\times106 = 0.0051$$

1st story

$$k_b = \frac{2370}{20\times12} = 9.88 \qquad k_c = \frac{1900}{12\times12} = 13.19$$

$V = 347\times0.2 = 69^k$

$h = 12.2\times12 = 146$ in.

$$D_r = \frac{9.88+13.19}{9.88\times13.19} \times \frac{146}{12\times29\times10^3}\times69 = 0.0050$$

$\therefore$ story drift < 0.03

Column axial stress check

3rd story $\quad W\,14 \times 145$

$$P = 30 \times 21 \times 50 \times 0.5 \times 10^{-3} = 16^{k}$$

$$f_c = \frac{16}{42.7} = 0.37\ ksi$$

2nd story $\quad W\,14 \times 159$

$$P = 16 + 100 \times 21 \times 40 \times 0.5 \times 10^{-3} = 58^{k}$$

$$f_c = \frac{58}{46.7} = 1.24\ ksi$$

1st story $\quad W\,14 \times 159$

$$P = 58 + 100 \times 21 \times 40 \times 0.5 \times 10^{-3} = 100^{k}$$

$$f_c = \frac{100}{46.7} = 2.14\ ksi$$

$$\therefore f_c < 0.10\,F_y = 3.6\ ksi$$
(A36)

Flexural stress check

3rd story

Middle column takes 45% ; W14×145
neglect beams

$$M = 70 \times 0.45 \times 12 = 378 \text{ k-ft.}$$

$$f_b = \frac{378 \times 12}{232} = 19.6 \text{ ksi} < F_y = 36 \text{ ksi}$$

2nd story

col. W14×159 takes 20%
beam W27×161 takes 40%

$$M = (70 + 195) \times 12 = 3180 \text{ k-ft.}$$

Column $f_b = \dfrac{3180 \times 0.20 \times 12}{254} = 30.0 \text{ ksi} < F_y = 36 \text{ ksi}$

Beam $f_b = \dfrac{3180 \times 0.40 \times 12}{455} = 33.5 \text{ ksi} < F_y = 36 \text{ ksi}$

1st story

Col. W14×159 takes 15%
Beam W24×84 takes 5%

$$M = 347 \times 12 = 4164 \text{ k-ft.}$$

Column $f_b = \dfrac{4164 \times 0.15 \times 12}{254} = 29.5 \text{ ksi} < F_y = 36 \text{ ksi}$

Beam $f_b = \dfrac{4164 \times 0.05 \times 12}{196} = 12.7 \text{ ksi} < F_y = 36 \text{ ksi}$

$\therefore f_b < F_y = 36 \text{ ksi}$ (A 36)

UC Campus:	**University of California San Francisco**			Date:	**June 26, 2019**	
Building CAAN:	**2325**	Auxiliary CAAN:	**NA**	**2418**	Auxiliary CAAN:	
Building Name:	**Koret Vision Research**			Initials:	***GSV***	**50 Kirkham Residenc e**	*Initials:*
Building Address:	**10 Koret Way, San Framcosco**			Page:	**1**	of	**4**

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type S1-S1A

LOW SEISMICITY

SEISMIC-FORCE-RESISTING SYSTEM

	Description
C NC N/A U ◉ ○ ○ ○	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1) **Comments: The number of lines of moment frames in each principal direction is greater than 2.**
C NC N/A U ◉ ○ ○ ○	DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.4.3.1, is less than 0.030. (Commentary: Sec. A.3.1.3.1. Tier 2: Sec. 5.5.2.1.2) **Comments: The drift ratio of the steel moment frames is less than 0.030.**
C NC N/A U ◉ ○ ○ ○	COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3) **Comments: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$.**
C NC N/A U ◉ ○ ○ ○	FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.4.3.9, is less than F_y. Columns need not be checked if the strong column–weak beam checklist item is compliant. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.2) **Comments: The average flexural stress in the moment frame columns and beams is less than F_y.**

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: Diaphragms are connected for transfer of seismic forces to the steel frames.**

UC Campus:	University of California San Francisco			Date:	June 26, 2019		
Building CAAN:	2325	Auxiliary CAAN:	NA	2418	Auxiliary CAAN:		
Building Name:	Koret Vision Research			Initials:	GSV	50 Kirkham Residence	*Initials:*
Building Address:	10 Koret Way, San Framcosco			Page:	2	of	4

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type S1-S1A

LOW SEISMICITY

SEISMIC-FORCE-RESISTING SYSTEM

C NC N/A U	
C: ● NC: ○ N/A: ○ U: ○	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1) **Comments: The columns in the seismic-force-resisting frames are anchored to the building foundation.**

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

SEISMIC-FORCE-RESISTING SYSTEM

	Description
C: ● NC: ○ N/A: ○ U: ○	REDUNDANCY: The number of bays of moment frames in each line is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1) **Comments: The number of bays of moment frames in each principal direction is greater than 2.**
C: ○ NC: ○ N/A: ● U: ○	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1) **Comments:**
C: ● NC: ○ N/A: ○ U: ○	MOMENT-RESISTING CONNECTIONS: All moment connections can develop the strength of the adjoining members based on the specified minimum yield stress of steel. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1). **Comments: All moment connections can develop the strength of the adjoining members based on the specified minimum yield stress of steel.**

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UC Campus:	University of California San Francisco			Date:	June 26, 2019		
Building CAAN:	2325	Auxiliary CAAN:	NA	2418	Auxiliary CAAN:		
Building Name:	Koret Vision Research			Initials:	GSV	50 Kirkham Residence	*Initials:*
Building Address:	10 Koret Way, San Framcosco			Page:	3	of	4

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Collapse Prevention Structural Checklist For Building Type S1-S1A

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
○ ● ○ ○	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel in accordance with AISC 341, Section A3.2. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1) **Comments: The moment-resisting connections were detailed according to pre-Northridge procedure, and thus do not comply with the provisions of AISC 341 Section A3.2.**
○ ● ○ ○	PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column. (Commentary: Sec. A.3.1.3.5. Tier 2: Sec. 5.5.2.2.2) **Comments: The moment-resisting connections were detailed according to pre-Northridge procedure, and thus do not possess the capacity to develop 0.8 times the sum of the flexural strengths of the girders framing in the face of the column.**
● ○ ○ ○	COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web. (Commentary: Sec. A.3.1.3.6. Tier 2: Sec. 5.5.2.2.3) **Comments: All column splice details located in moment-resisting frames include connection of both flanges and web.**
● ○ ○ ○	STRONG COLUMN—WEAK BEAM: The percentage of strong column–weak beam joints in each story of each line of moment frames is greater than 50%. (Commentary: Sec. A.3.1.3.7. Tier 2: Sec. 5.5.2.1.5) **Comments: The percentage of strong column-weak beam joints in each story of each line of moment frames is greater than 50%.**
● ○ ○ ○	COMPACT MEMBERS: All frame elements meet section requirements in accordance with AISC 341, Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.4) **Comments:**

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Collapse Prevention Structural Checklist For Building Type S1-S1A

DIAPHRAGMS (STIFF OR FLEXIBLE)

	Description
C NC N/A U ◉ ○ ○ ○	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3) **Comments: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length.**

FLEXIBLE DIAPHRAGMS

	Description
C NC N/A U ○ ○ ◉ ○	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) **Comments:**
C NC N/A U ○ ○ ◉ ○	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:**
C NC N/A U ○ ○ ◉ ○	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) **Comments:**
C NC N/A U ○ ○ ◉ ○	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) **Comments:**
C NC N/A U ○ ○ ◉ ○	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:**

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