UCSF

Rating form completed by: Simpson Gumpertz & Heger, SGH.COM
HK, LZ, 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

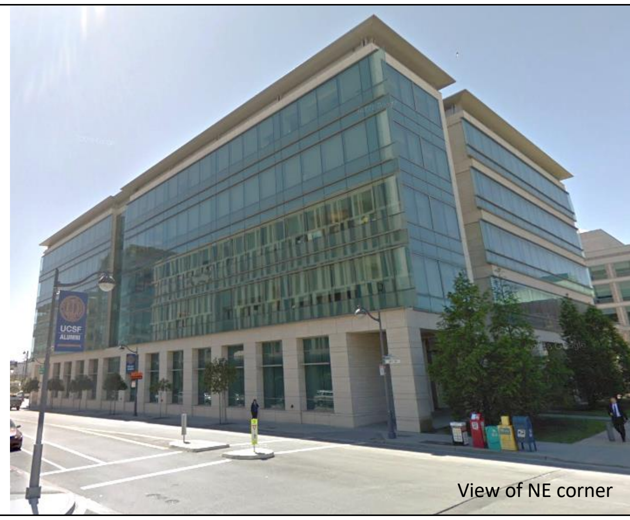
Byers Hall

CAAN# *3034*

1700 4th Street, San Francisco, CA 94158

UCSF Campus Site: *Mission Bay*

11/02/2020



View of NE corner

Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	IV	Findings based on a drawing review, ASCE 41-17 Tier 1 evaluation[1], and CBC 2007 evaluation
Rating basis	Tier 1 & 2	ASCE 41-17 and CBC 2007 Tier 2 limited to brace checks only
Date of rating	2020	
Recommended UCSF priority category for retrofit	N/A	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application for modification
Ballpark total project cost to retrofit to IV rating	N/A	See recommendations on further evaluation and retrofit
Is 2018-2019 rating required by UCOP?	Yes	BRBF building constructed per building code pre-dating IBC 2006 benchmark
Further evaluation recommended?	No	

[1] The evaluations at UCSF translate the Tier 1 & 2 evaluation to a Seismic Performance Level rating using professional judgment discussed among the Seismic Review Committee. Non-compliant items in the Tier 1 & 2 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

- Structure drawings by Rutherford & Chekene, "UCSF QB3," record drawings dated 29 August 2005.

Additional building information known to exist

- Geotechnical report and boring logs by Harding ESE, dated 2002.
- Boring logs by Harding Lawson Associates, dated 1999.

Scope for completing this form

SGH reviewed the original structural record drawings to complete a rudimentary evaluation of the building seismic force-resisting system. In order to determine a seismic performance rating, the primary goal was to confirm whether the buckling-restrained brace frame (BRBF) meets the requirements of the benchmark design code per the UC Facilities Manual UC Seismic Program Guidelines. The Guidelines establish the benchmark as the 2006 International Building Code. This is equivalent to the 2007 California Building Code, which uses the structural seismic design provisions of ASCE 7-05 and AISC 341-05 for the design of BRBFs.

Because the building did not directly meet the applicable provisions of the benchmark code, we also completed an ASCE 41-17 Tier 1 evaluation and a limited Tier 2 evaluation of the braces.

No site visit was performed for the purpose of completing this rating.

Brief description of structure

UCSF QB3, henceforth referred to as Byers Hall is located at the corner of 16th Street and 4th Street in San Francisco, California. The building is a 150,000 sq ft rectangular building. The building has five stories above grade. The roof is 80 ft above Level 1 and the highest penthouse structural elements are approximately 130 ft above Level 1. The outer dimensions measure approximately 216 ft in the longitudinal (north-south) direction and 159 ft in the transverse (east-west) direction.

Byers Hall is directly adjacent to Building 24 A/B, also known as Genentech Hall. The two buildings are connected at Levels 2 through 5. At Level 2, a 6 in. wide seismic joint provides separation for movement. At Levels 3 through Roof, the seismic joint is 24 in. wide.

Rutherford & Chekene is the structural engineer for the building. Byers Hall was designed per the 1998 California Building Code and the underlying structural provisions of the 1997 Uniform Building Code. Construction was completed in 2005.

Identification of Levels: The building is located on an essentially flat site, with levels designated as: Level 1 (EL. 0 ft-0 in.) at street level, Level 2 (EL. 20 ft-0 in.), Level 3 (EL. 35 ft-0 in.), Level 4 (EL. 50 ft-0 in.), Level 5 (65 ft-0 in.), and Roof (EL. 80 ft-0 in.)

Foundation System: The structure is founded on driven, pre-cast, prestressed reinforced concrete piles. Piles are 14 in. square typically and are driven approximately 75 ft-6 in. below grade into bedrock. Reinforced concrete grade beams connect the pile caps at braced frame lines and along the building perimeter. A 12-inch thick structural first floor slab is supported by the pile caps and grade beams. The slab is reinforced with #6 top and bottom bars at 12 in. o.c. each way.

Structural System for Vertical (gravity) load: The typical floor, including the roof levels, uses a 2 in. deep metal deck with 4-1/2 in. normal-weight concrete fill (6-1/2 in. total depth), supported by structural steel W-beams. Bay sizes are typically 21 ft x 28 ft or 28 ft x 28 ft and W18, W21, or W24 beams are generally spaced at 7 ft-0 in. o.c. with heavier girders between columns. All gravity beam connections are typically single-plate shear tabs. Steel wide-flange columns, located on gridlines, vary from W14x74 to W14x342. All steel columns are continuous to foundation elements. The roof structure supports a few pieces of mechanical equipment and includes steel framing supporting stack shrouds and a screen wall.

Structural System for Lateral Loads: The lateral force-resisting system comprises buckling-restrained braced frames. A total of fifteen bays of bracing are provided in the structure: seven bays in three separate lines in the longitudinal direction and eight bays in four separate lines in the transverse direction. Braces vary in core area size from 6 to 28 square inches using SN400B material (Japanese specification).

These braces occur along Gridlines B (3 bays), D (2 bays), G (2 bays), 3 (2 bays), 4 (2 bays), 6 (2 bays), and 7 (2 bays). There is one pair of opposing braces at each level in each frame along Gridlines B, G, 4, and 6. Most bays have a single diagonal brace. However, there is a bay with chevron bracing at Gridline B, and there are two bays with chevron bracing at both Gridlines 3 and 7. There are no multi-tiered frames.

Brace connections are concentric and bolted with 1-1/4 in. A490 slip critical bolts. Gusset plates are 1 in. to 1-1/2 in. thick and welded to columns, beams, and base plates with 3/8 in. double-sided fillet welds. Beams at the braced frames are moment connected to columns with welded flanges and webs.

Each frame column has a 4 in. to 5-1/2 in. thick base plate anchored into the pile cap with 4 ft-0 in. long, 2 in. to 2-1/2 in. diameter, A354 Grade BD high strength anchor bolts. The pile caps are reinforced with #4 ties at 4 in. o.c. around each group of 6 to 15 anchor bolts. Shear loads are transferred from the base plates to the reinforced concrete first floor slab through #9 rebar couplers that are welded to the sides of the base plate.

Brief description of seismic deficiencies and Expected Seismic Performance

Identified seismic deficiencies of the building include the following:

Seismic Deficiency	Expected Seismic Performance
Torsion & Redundancy	The center of mass and center of rigidity of the structure are located at similar locations, resulting in limited inherent torsion for loading in the longitudinal direction and negligible inherent torsion for loading in the transverse direction. However, the transverse direction frames are all located within the middle half of the building rather than at the perimeter. Due to this, the building exhibits significant sensitivity to torsion due to accidental eccentricity. Due to this torsional sensitivity, under IBC 2006/CBC 2007 & ASCE 7-05, the building has a torsional irregularity, bordering on classification as an extreme torsional irregularity. Because of changes in the determination of the redundancy factor that occurred in the building code between UBC 1997 and IBC 2006, having an extreme torsional irregularity would fail the criteria for a unity redundancy factor (ρ=1.0) and would result in the need to use a higher redundancy factor (ρ=1.3). Given the relatively small inherent torsion and the number of braces, we can use engineering judgement to conclude that no extreme torsional irregularity exists and that ρ=1.0 may be used. To account for the torsional sensitivity, the accidental eccentricity has been amplified per ASCE 7-05. Additionally, although not required by IBC 2006/CBC 2007, the brace designs have been checked for 100%+30% directional combinations from seismic loading in both major axis directions. Note: During the original design per CBC 1998, the presence of a torsional irregularity would have been acknowledged in the design. However, at this time, the redundancy factor was based on the maximum element to story shear ratio. Given the number of braces in both directions, this would have resulted in a unity redundancy factor (ρ=1.0) under CBC 1998. Redundancy and torsion checks pass ASCE 41-17 Tier 1 requirements.
Brace Strength	Under IBC 2006/CBC 2007 & ASCE 7-05, braces were evaluated using a lower bound yield strength for the SN400B core plates (Fysc = 34 ksi) and a strength reduction factor (ϕ) of 0.9. 4 braces (out of 110) exhibit DCR values greater than unity. The maximum DCR value is 1.08. Braces fail the ASCE 41-17 Tier 1 quick check. Under a more detailed Tier 2/3 evaluation, the brace stresses improve, but are still estimated to result in a high acceptance criteria ratio of approximately 1.55.

Chevron Bracing	Under IBC 2006/CBC 2007, BRBF beams are required to resist mechanism loading due to the braces reaching their expected capacity. For this evaluation, we used the following values for the adjusted forces in the braces: Fysc = 41 ksi (as specified in the drawings), Ry = 1.1, ω = 1.25, β = 1.35. The actual ω and β values are unknown. Braced frames have chevron bracing at Lines 3, 7, B, and G. When the beams are subjected to mechanism loads corresponding to the simultaneous yielding and buckling of the bracing pair, the beam DCR is as high as 1.9 for combined compression and flexure. This occurs at 26 out of 75 frames beams. The same connection strength check applies to the ASCE 41-17 Tier 1 evaluation.
Column Strength	Under IBC 2006/CBC 2007, BRBF columns are required to resist loading delivered to braces when the columns reach their expected capacity. Assuming a 100%+30% directional combination result, a few first story columns at the ends of chevron frames do not pass this strength check, with DCRs as high as 1.45. This occurs at 8 out of 21 frame columns. (Note: It is unlikely that braces in both directions would reach simultaneously reach expected strength and a require use of a 100%+100% directional combination.) Columns pass the ASCE 41-17 Tier 1 quick check for axial stress under gravity loading.
Brace Connections	Under IBC 2006/CBC 2007, BRBF brace connections are required to resist loading due to the braces reaching their expected capacity. Using Fysc = 41 ksi, Ry = 1.1, ω = 1.25, β = 1.35, the connections are typically able to resist this load with only minor overstresses (DCR = 1.07). The connections would pass this check if the actual ω and β values are lower than the assumed values. The same connection strength check applies to the ASCE 41-17 Tier 1 evaluation.

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/A
Load path	N	Liquefaction	N
Adjacent buildings	N	Slope failure	N/A
Weak story	N/A	Surface fault rupture	N/A
Soft story	N/A	Masonry or concrete wall anchorage at flexible diaphragm	N/A
Geometry (vertical irregularities)	N/A	URM wall height-to-thickness ratio	N/A
Torsion	N	URM parapets or cornices	N/A
Mass – vertical irregularity	N/A	URM chimney	N/A
Cripple walls	N/A	Heavy partitions braced by ceilings	N/A
Wood sills (bolting)	N/A	Appendages	N/A
Diaphragm continuity	N/A		

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

A detailed assessment of nonstructural systems has not been performed.

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	Not visited	Unrestrained hazardous materials storage	Not visited
Heavy masonry or stone veneer above exit ways and public access areas	Not visited	Masonry chimneys	Not visited
Unbraced masonry parapets, cornices or other ornamentation above exit ways and public access areas	Not visited	Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.	Not visited

Basis of seismic performance level rating

The building does not meet the benchmark code for BRBF structures, lowering the potential SPL-III rating to SPL-IV. The structure is generally capable of withstanding IBC 2006/CBC 2007 and ASCE 7-05 code-level ground motion forces. However, the key issue is that the columns and beams are not capable of developing the expected capacity of the braces.

In the ASCE 41-17 Tier 1 evaluation, the building demonstrates several deficiencies involving stresses in columns, braces, and chevron frames under the BSE-C pseudo seismic force. More detailed Tier 2 or 3 analysis improve the results. However, the brace demands remain high and fail to meet the acceptance criteria. Beam failures associated with behavior within a chevron bracing arrangement are expected to result in localized failures without compromising the overall lateral or gravity stability of the structure.

Recommendations for further evaluation or retrofit:

No additional evaluation is required.

Peer review comments on rating

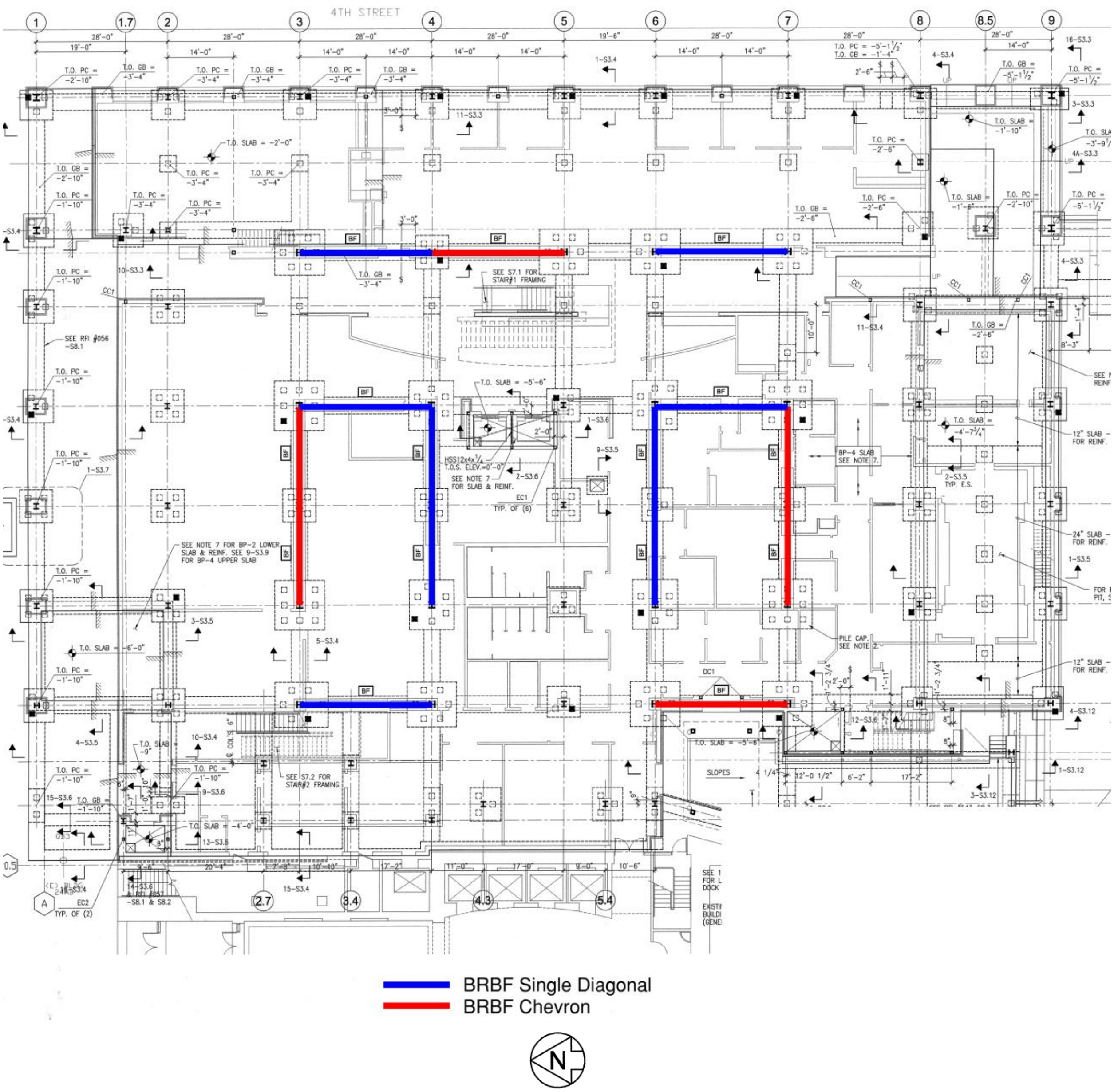
The structural members of the UCSF Seismic Review Committee (SRC) reviewed the evaluation on 12 February 2020 and agree with the rating of IV.

Additional building data	Entry	Notes
Latitude	37.767°	
Longitude	-122.391°	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	5	
Number of stories (basements) below lowest perimeter grade	0	Building site is essentially flat
Building occupiable area (OGSF)	150,000	Estimated from drawings
Risk Category per 2016 CBC 1604.5	II	
Building structural height, h_n	80 ft	As defined per ASCE 7-16 Section 11.2
Coefficient for period, C_t	0.02	ASCE 41-17 equation 4-4 and 7-18
Coefficient for period, ⍰	0.75	ASCE 41-17 equation 4-4 and 7-18
Estimated fundamental period	0.535 sec	ASCE 41-17 equation 4-4 and 7-18
Site data		
975 yr hazard parameters S_s, S_1	1.380, 0.532 [1.5, 0.634]	https://hazards.atcouncil.org/ ASCE 41-17 BSE-C [IBC 2006/ASCE 7-05]
Site class	E	UCSF Pre-2006 BRBF Buildings Geotechnical Characteristic and Geohazards (2019)
Site class basis	Estimated	UCSF Pre-2006 BRBF Buildings Geotechnical Characteristic and Geohazards (2019)
Site parameters F_a, F_v	1.3, 4.2 [0.9, 2.4]	https://hazards.atcouncil.org/ ASCE 41-17 BSE-C [IBC 2006/ASCE 7-05]
Ground motion parameters S_{cs}, S_{c1}	1.794, 2.236	UCSF Group 2 Buildings, Geotechnical Characteristic and Geohazards (2019) ASCE 41-17 BSE-C
S_a at building period	1.792	Calculated
Site V_{s30}	200 m/s	UCSF Pre-2006 BRBF Buildings Geotechnical Characteristic and Geohazards (2019)
V_{s30} basis	Estimated	UCSF Pre-2006 BRBF Buildings Geotechnical Characteristic and Geohazards (2019)
Liquefaction potential	No	UCSF Pre-2006 BRBF Buildings Geotechnical Characteristic and Geohazards (2019)
Liquefaction assessment basis	Piles extend to non-liquefiable soils/bedrock	Borings by Harding ESE (2002) and Harding Lawson Associates (1999)

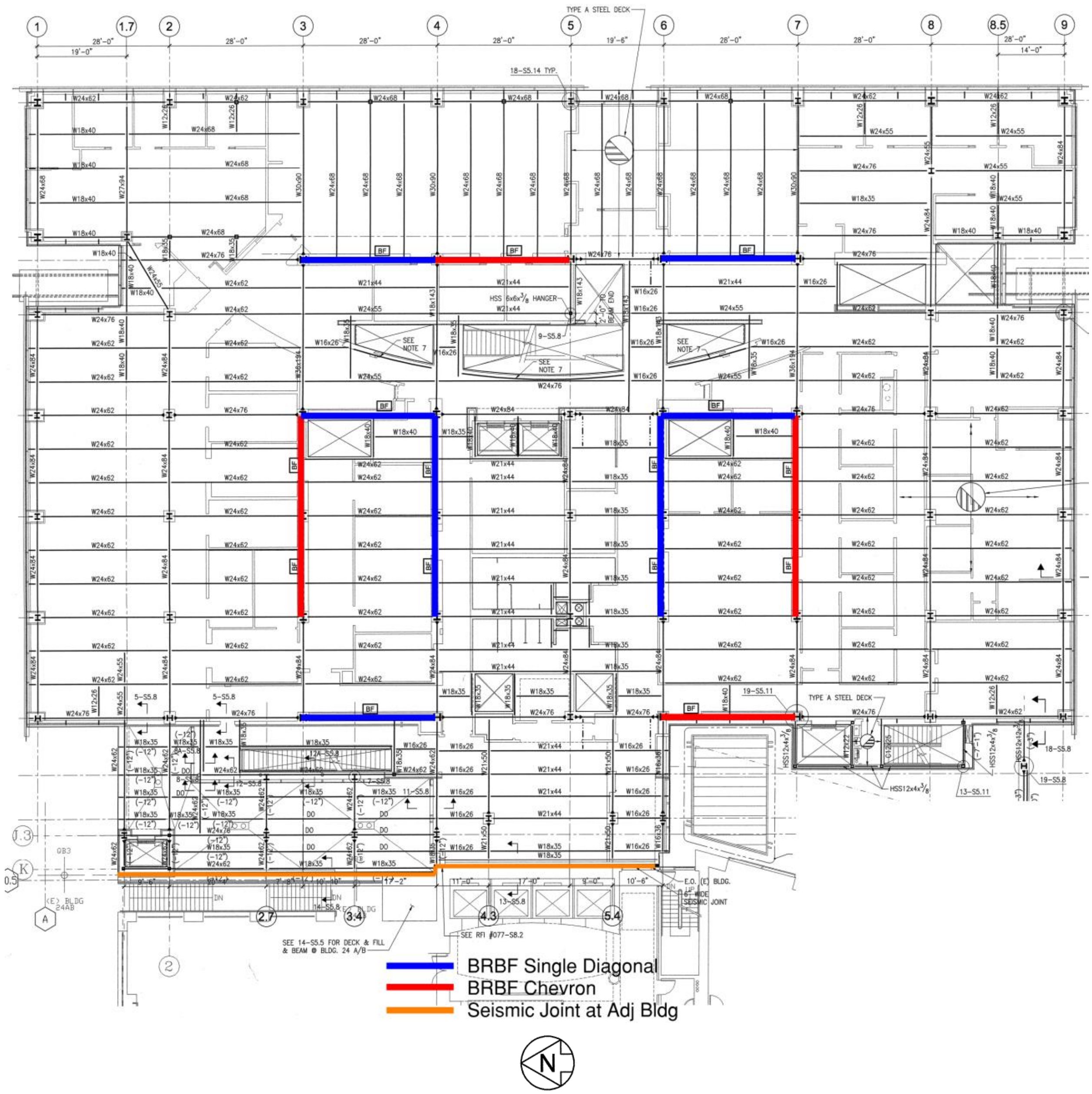
Landslide potential	No	UCSF Pre-2006 BRBF Buildings Geotechnical Characteristic and Geohazards (2019)
Landslide assessment basis	Essentially flat site	
Active fault-rupture hazard identified at site?	No	UCSF Pre-2006 BRBF Buildings Geotechnical Characteristic and Geohazards (2019)
Site-specific ground motion study?	Yes	Harding ESE (2002)
Applicable code		
Applicable code or approx. date of original construction	Code: CBC 1998 Built: 2005	
Applicable code for partial retrofit	None	No partial retrofit known
Applicable code for full retrofit	None	No full retrofit known
Model building data		
Model building type North-South	S2 Steel Braced Frame	Buckling-Restrained Braced Frame
Model building type East-West	S2 Steel Braced Frame	Buckling-Restrained Braced Frame
FEMA P-154 score	N/A	Not included here because we performed ASCE 41 Tier 1 evaluation.
Previous ratings		
Most recent rating	-	
Date of most recent rating	-	
2nd most recent rating	-	
Date of 2nd most recent rating	-	
Appendices		
ASCE 41 Tier 1 checklist included here?	Yes	Refer to attached checklist file

Appendix A

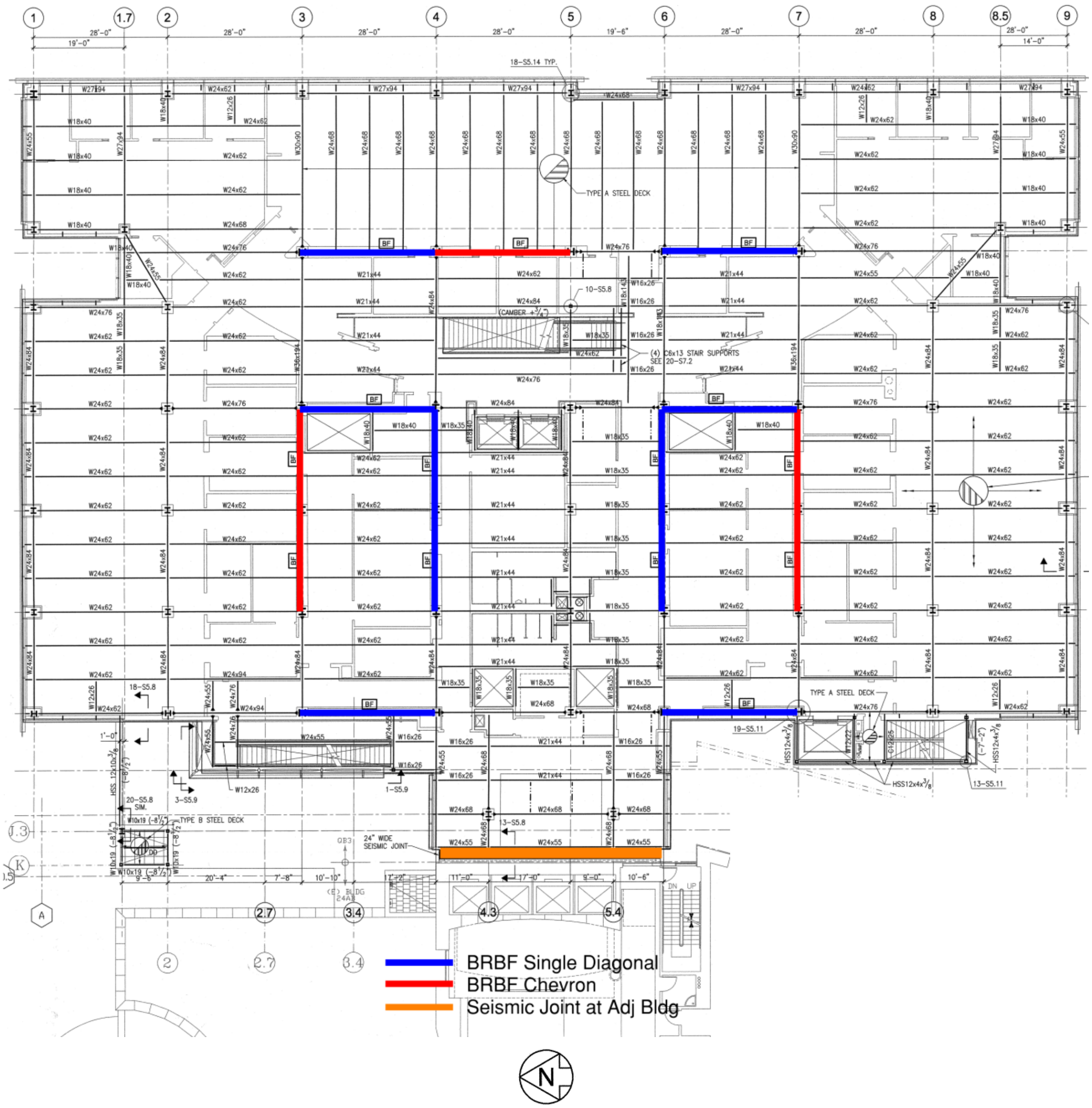
Drawing Images



Level 1 Framing and Foundations Plan



Level 2 Framing Plan



Level 3 Framing Plan

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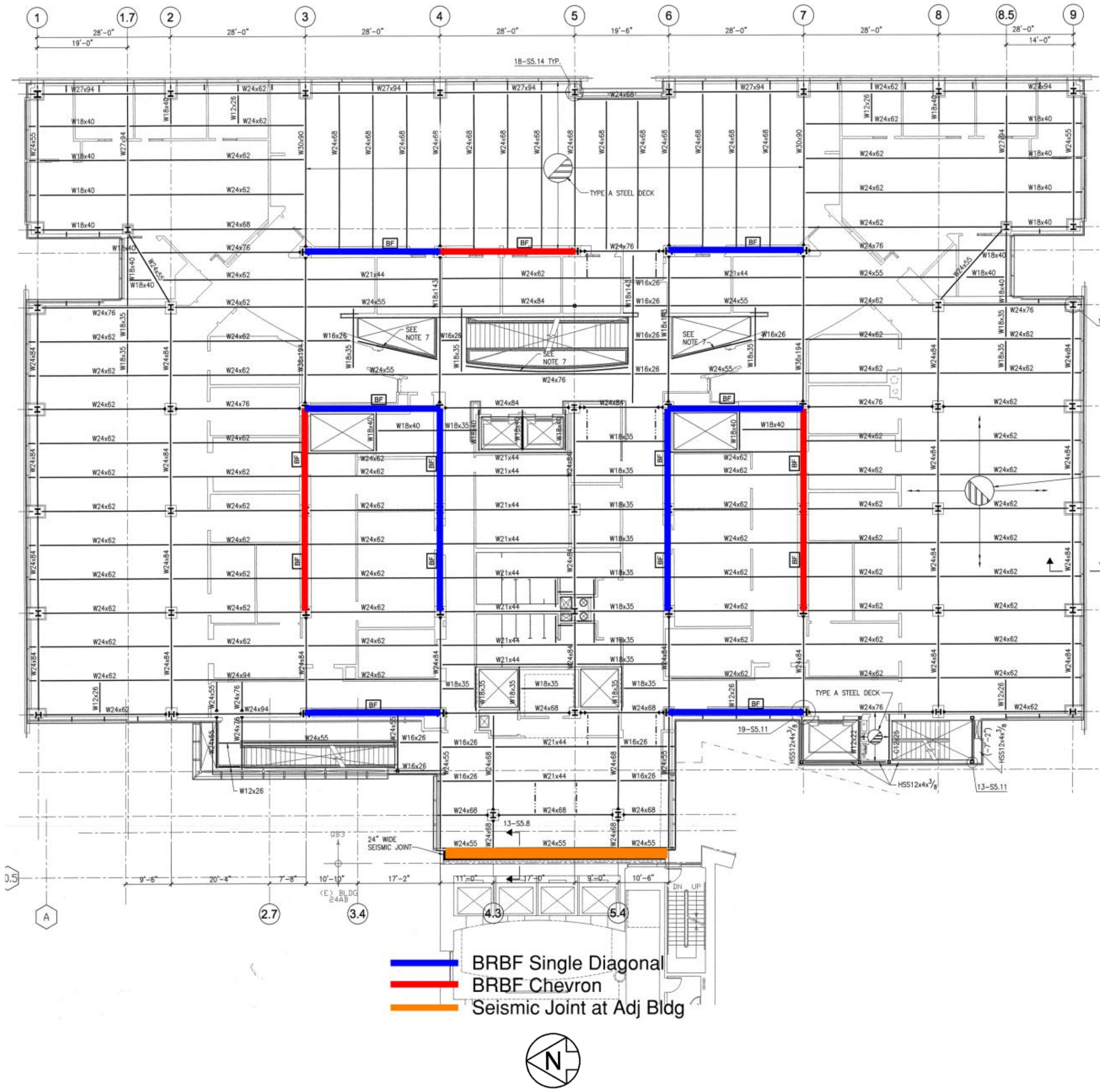
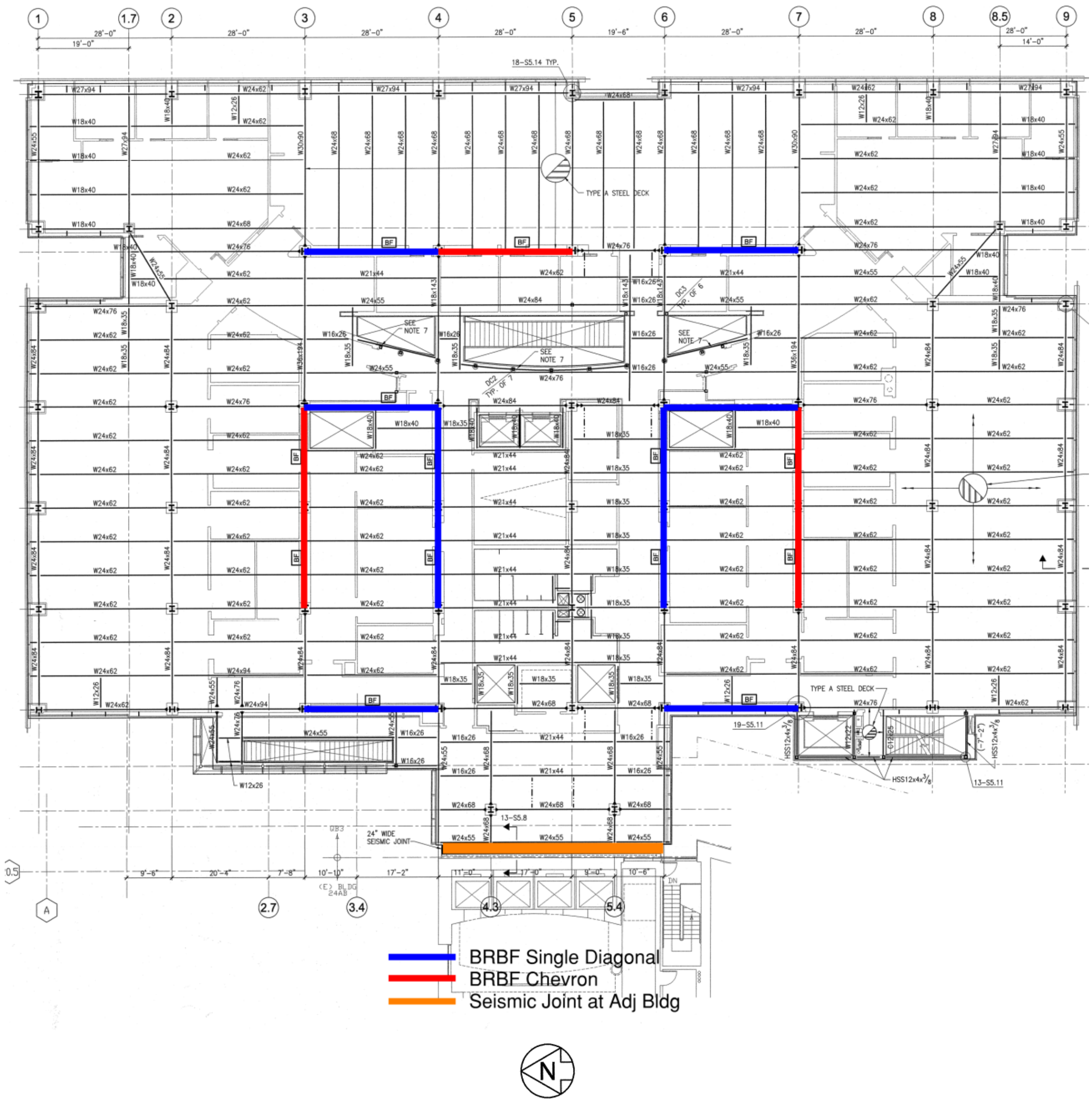
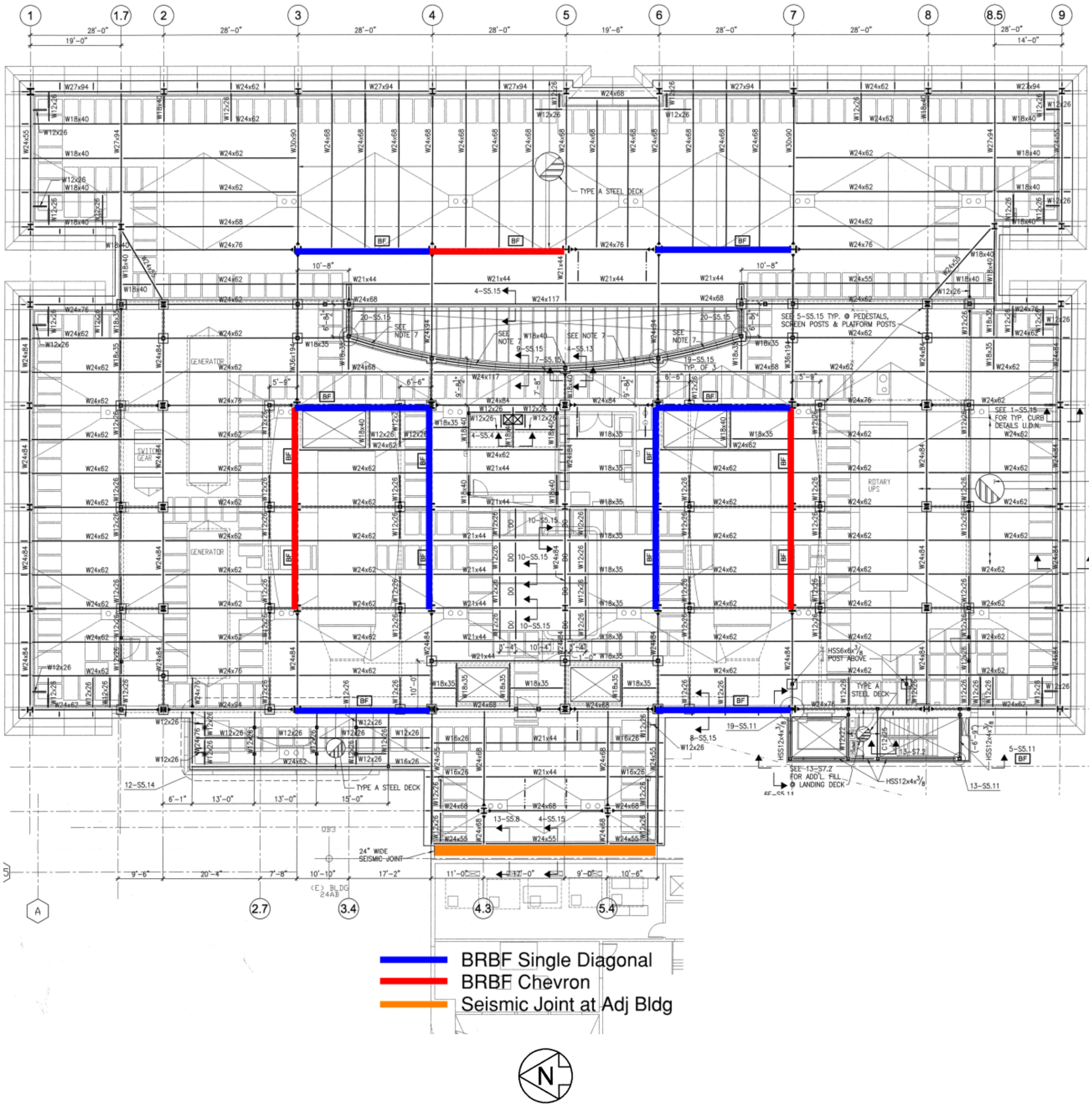


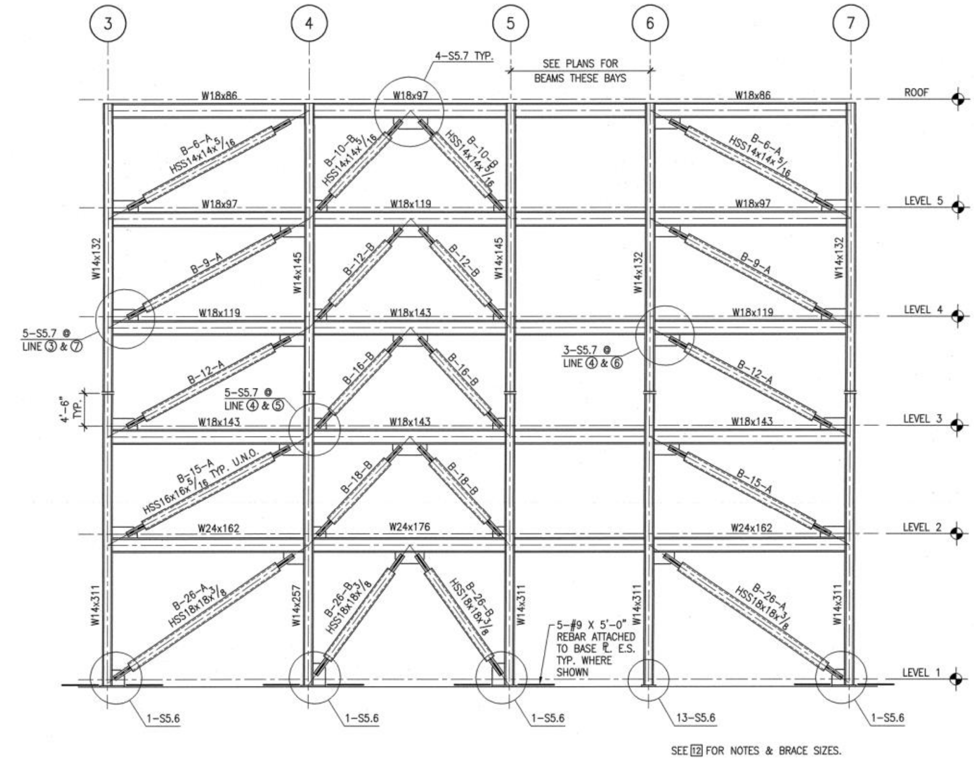
Level 4 Framing Plan



Level 5 Framing Plan

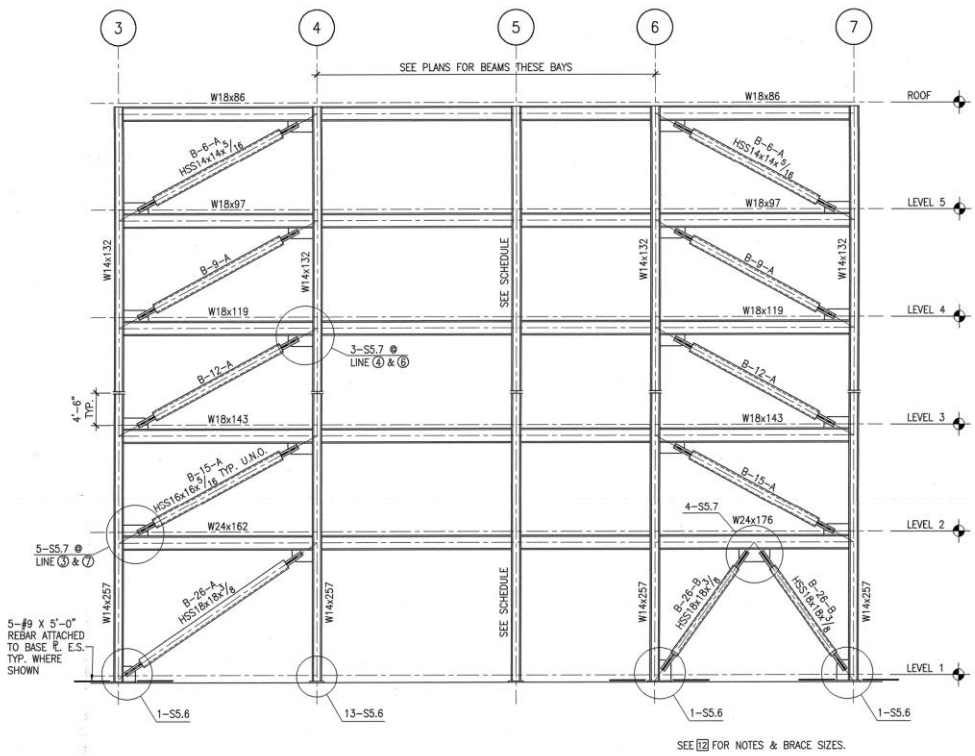


Roof Framing Plan



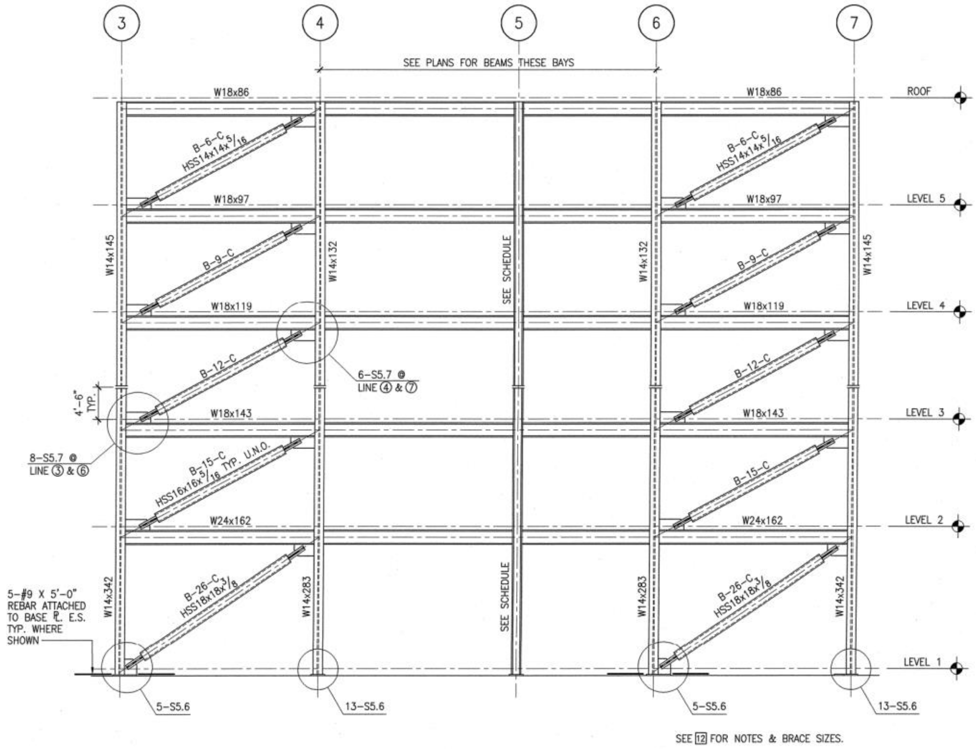
UNBONDED BRACES @ LINES B

1



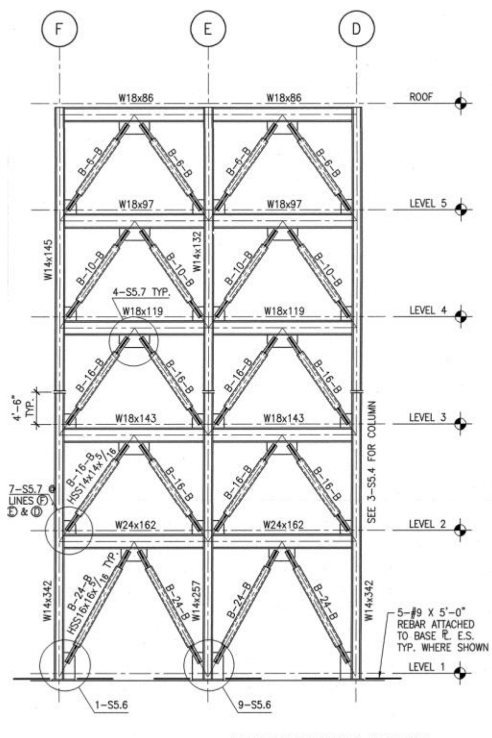
UNBONDED BRACES @ LINES G

9



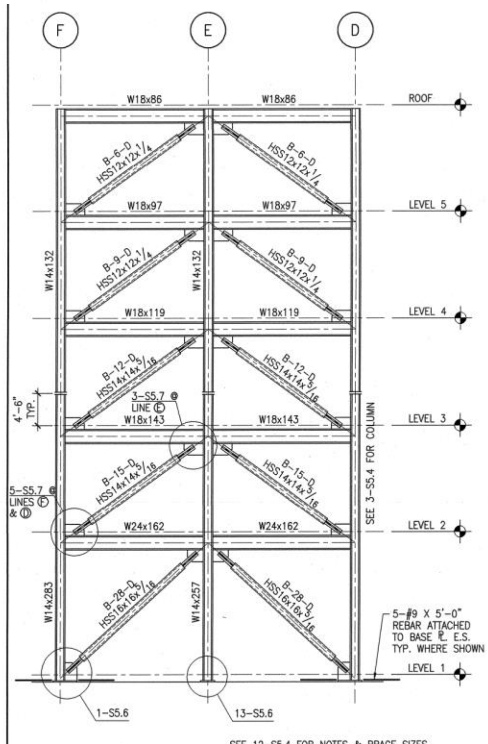
UNBONDED BRACES @ LINES D

3



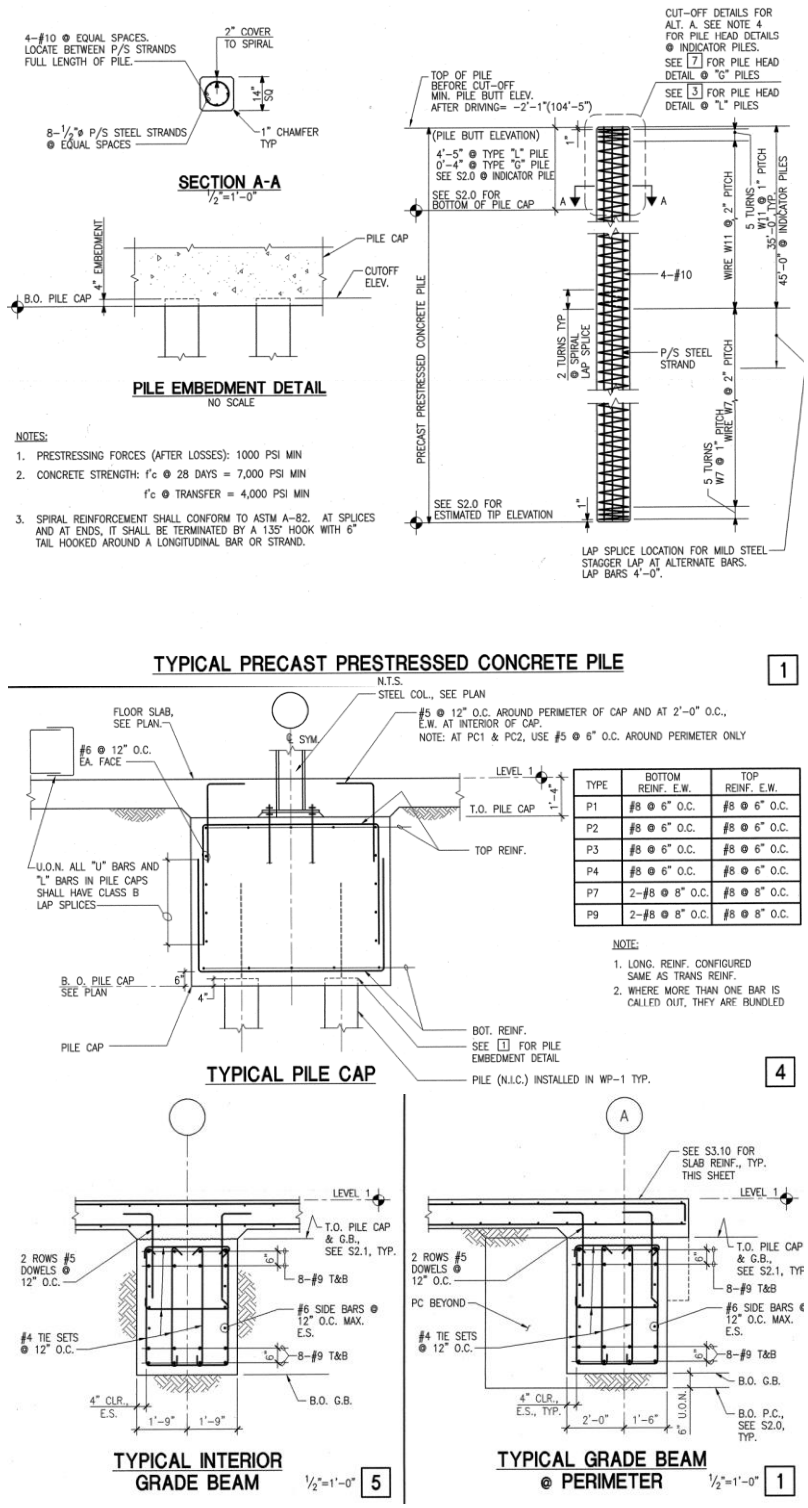
UNBONDED BRACES @ LINES 3 & 7

5

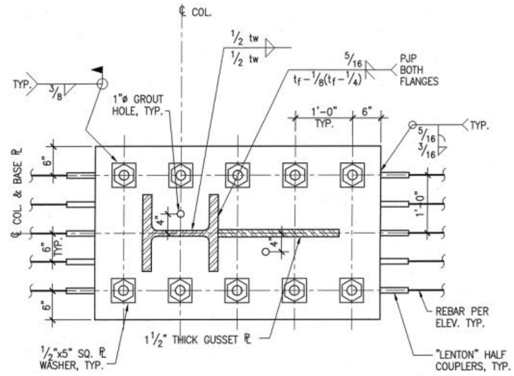


UNBONDED BRACES @ LINES 4 & 6

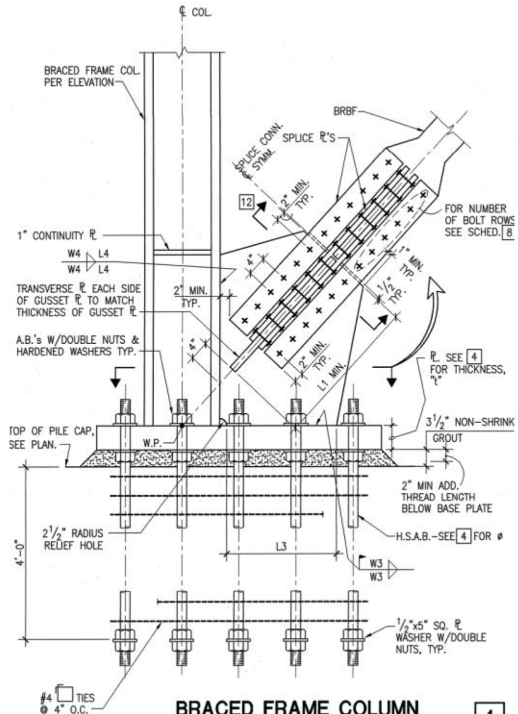
1



TYPE	BOTTOM REINF. E.W.	TOP REINF. E.W.
P1	#8 @ 6" O.C.	#8 @ 6" O.C.
P2	#8 @ 6" O.C.	#8 @ 6" O.C.
P3	#8 @ 6" O.C.	#8 @ 6" O.C.
P4	#8 @ 6" O.C.	#8 @ 6" O.C.
P7	2-#8 @ 8" O.C.	#8 @ 8" O.C.
P9	2-#8 @ 8" O.C.	#8 @ 8" O.C.

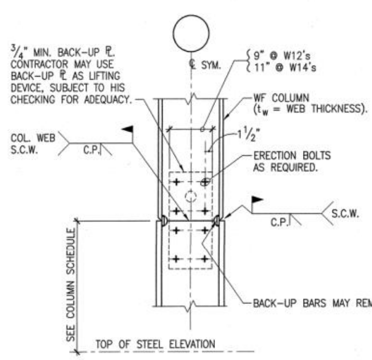


PLAN



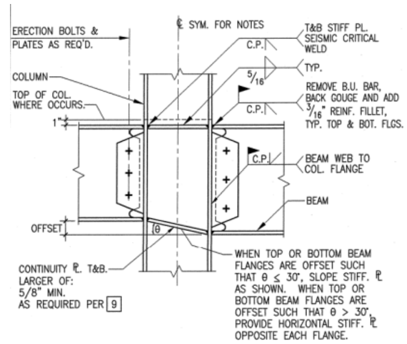
BRACED FRAME COLUMN
BASE PLATE

1



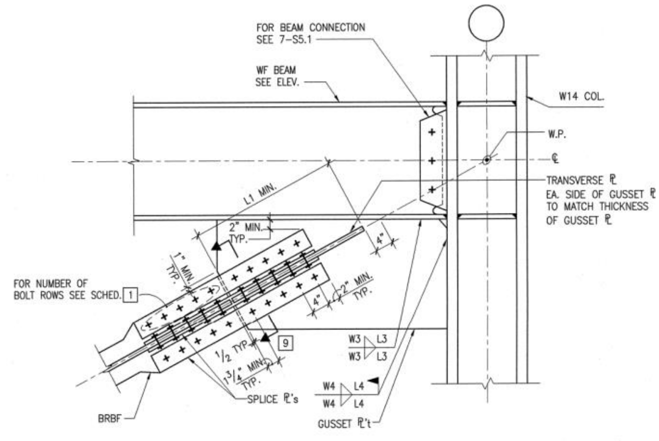
COLUMN SPLICE
BRACE FRAME COLUMNS

15

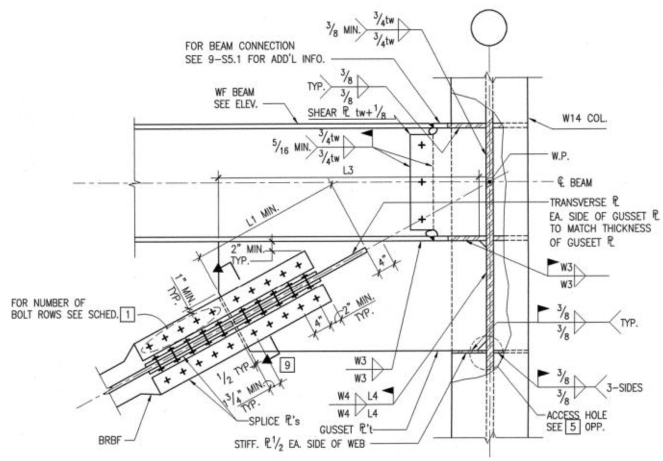


MOMENT CONNECTION OF BEAM
TO COLUMN FLANGE

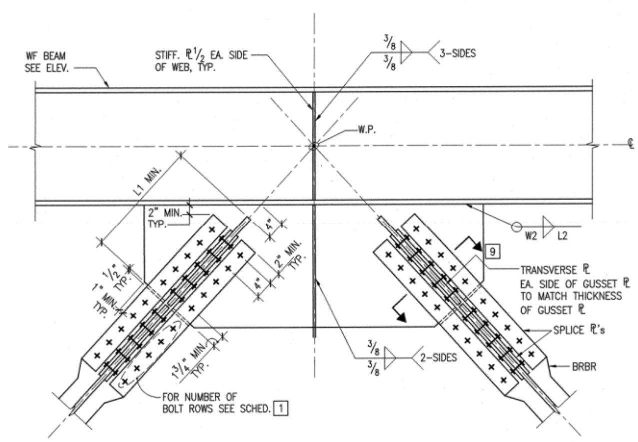
7



TYP. BRACE TO COLUMN FLANGE CONNECTION 1"=1'-0" 3



TYP. BRACE TO COLUMN WEB CONNECTION 1"=1'-0" 6



TYP. CHEVRON BRACE CONNECTION 1"=1'-0" 4

Appendix B

IBC 2006/CBC 2007/ASCE 7-05 Calculations

Flat Loads

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CLIENT UCSF

SUBJECT Byers Hall: Flat Loads

SHEET NO.

PROJECT NO. 197042.00-UCSF

DATE 13 March 2020

BY LZhou

CHECKED BY

Typical Laboratory + Office

Material	Self-Weight (psf)	SDL (psf)	Gravity (psf)	Seismic (psf)	Remarks
4.5" N.W. Concrete on W2, 20 ga.	68.5	-	68.5	68.5	*Self-weight & mass included in model*
Steel Beams/Girders	14.0	14.0	14.0	14.0	*Self-weight & mass included in model*
Steel Columns	*	*	*	*	*Self-weight & mass included in model*
Buckling-Restained Braces	*	*	*	*	*Self-weight & mass included in model*
Finishes & MEP	-	12.0	12.0	12.0	
Partitions	-	20.0	20.0	20.0	
Sum of Dead Loads	**82.5**	**46.0**	**114.5**	**114.5**	
Sum of Live Loads	**-**	**-**	**100.0**	**-**	*Reducible*
Sum of Dead Plus Live Loads	**-**	**-**	**214.5**	**114.5**	

Typical Roof

Material	Self-Weight (psf)	SDL (psf)	Gravity (psf)	Seismic (psf)	Remarks
4.5" N.W. Concrete on W2, 20 ga.	68.5	-	68.5	68.5	*Self-weight & mass included in model*
Steel Beams	14.0	14.0	14.0	14.0	*Self-weight & mass included in model*
Steel Columns	*	*	*	*	*Self-weight & mass included in model*
Buckling-Restained Braces	*	*	*	*	*Self-weight & mass included in model*
Roofing/Insulation	-	8.0	8.0	8.0	
Finishes & MEP	-	12.0	12.0	12.0	
Partitions	-	-	-	10.0	
Sum of Dead Loads	**82.5**	**34.0**	**102.5**	**112.5**	
Sum of Live Loads	**-**	**-**	**20.0**	**-**	*Roof, Reducible*
Sum of Dead Plus Live Loads	**-**	**-**	**122.5**	**112.5**	

Roof Mechanical Area

Material	Self-Weight (psf)	SDL (psf)	Gravity (psf)	Seismic (psf)	Remarks
4.5" N.W. Concrete on W2, 20 ga.	68.5	-	68.5	68.5	*Self-weight & mass included in model*
Steel Beams	14.0	14.0	14.0	14.0	*Self-weight & mass included in model*
Steel Columns	*	*	*	*	*Self-weight & mass included in model*
Buckling-Restained Braces	*	*	*	*	*Self-weight & mass included in model*
Roofing/Insulation	-	8.0	8.0	8.0	
Finishes & MEP	-	12.0	12.0	12.0	
Partitions	-	-	-	10.0	
Sum of Dead Loads	**82.5**	**34.0**	**102.5**	**112.5**	
Sum of Live Loads	**-**	**-**	**100.0**	**100.0**	*Non-reducible (incl'd mech equip. & pads)*
Sum of Dead Plus Live Loads	**-**	**-**	**202.5**	**212.5**	

Mechanical Platform

Material	Self-Weight (psf)	SDL (psf)	Gravity (psf)	Seismic (psf)	Remarks
Grating (1 1/4" x 1/8" Bars @ 1 3/16" o.c.)	6.0	-	6.0	6.0	
Steel Beams	15.0	15.0	15.0	15.0	*Self-weight & mass included in model*
Steel Columns	*	*	*	*	*Self-weight & mass included in model*
Braces	*	*	*	*	*Self-weight & mass included in model*
Sum of Dead Loads	**21.0**	**15.0**	**21.0**	**21.0**	
Sum of Live Loads	**-**	**-**	**50.0**	**50.0**	*Non-reducible (incl'd mech equip)*
Sum of Dead Plus Live Loads	**-**	**-**	**71.0**	**71.0**	

SIMPSON GUMPERTZ & HEGER

Engineering of Structures
and Building Enclosures

CLIENT UCSF

SUBJECT Byers Hall: Flat Loads

SHEET NO.

PROJECT NO. 197042.00-UCSF

DATE 13 March 2020

BY LZhou

CHECKED BY

Terrace

Material	Self-Weight (psf)	SDL (psf)	Gravity (psf)	Seismic (psf)	Remarks
4.5" N.W. Concrete on W2, 20 ga.	68.5	-	68.5	68.5	*Self-weight & mass included in model*
Steel Beams	14.5	14.5	14.5	14.5	*Self-weight & mass included in model*
Steel Columns	*	*	*	*	*Self-weight & mass included in model*
Buckling-Restained Braces	*	*	*	*	*Self-weight & mass included in model*
Terrazzo & Concrete Fill	-	32.0	32.0	32.0	
Finishes & MEP	-	12.0	12.0	12.0	
Partitions	-	-	-	10.0	
Sum of Dead Loads	**83.0**	**58.5**	**127.0**	**137.0**	
Sum of Live Loads	**-**	**-**	**100.0**	**-**	*Reducible*
Sum of Dead Plus Live Loads	**-**	**-**	**227.0**	**137.0**	

Wall Weights

Material	Self-Weight (psf)	SDL (psf)	Gravity (psf)	Seismic (psf)	Remarks
Typical Exterior Wall	15.0	-	15.0	15.0	
Roof Screen Wall	15.0	-	15.0	15.0	
Stack Shroud	15.0	-	15.0	15.0	

Seismic Mass

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	Mass Type	Area (sf)	Length (ft)	Load (psf)	Load (plf)	Mass (kips)	ETABS (kips)
Roof	Typ Roof Area (Slab, Beams, SDL)	15,435		113		1736	
h = 15.00'	Mechanical Area (Slab, Beams, SDL)	13,930		213		2960	
	Mechanical Platform	3,500		61		214	
	Stack Shroud Framing					63	
	Stack Shroud Cladding		254		450	114	
	Screen Wall		535		350	187	
	Parapet Above		795		53	42	
	Exterior Wall Below		795		113	89	
	Columns Below					45	
	Braces Below					32	
					Σ =	**5,406**	**5,517**
					ETABS/Calculation =		102%
Level 5	Typical Area (Slab, SDL)	29,365		115		3362	
h = 15.00'	Exterior Wall Above		795		113	89	
	Exterior Wall Below		795		113	89	
	Columns Above					45	
	Columns Below					45	
	Braces Above					32	
	Braces Below					40	
					Σ =	**3,541**	**3,642**
					ETABS/Calculation =		103%
Level 4	Typical Area (Slab, SDL)	29,365		115		3362	
h = 15.00'	Exterior Wall Above		795		113	89	
	Exterior Wall Below		795		113	89	
	Columns Above					45	
	Columns Below					45	
	Braces Above					40	
	Braces Below					49	
					Σ =	**3,541**	**3,656**
					ETABS/Calculation =		103%
Level 3	Typical Area (Slab, SDL)	29,735		115		3405	
h = 15.00'	Exterior Wall Above		795		113	89	
	Exterior Wall Below		795		113	89	
	Columns Above					45	
	Columns Below					83	
	Braces Above					49	
	Braces Below					54	
					Σ =	**3,711**	**3,782**
					ETABS/Calculation =		102%

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Engineering of Structures
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CLIENT	UCSF
SUBJECT	Byers Hall - Seismic Weight
SHEET NO.	
PROJECT NO.	197042.00-UCSF
DATE	13 March 2020
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	Mass Type	Area (sf)	Length (ft)	Load (psf)	Load (plf)	Mass (kips)	ETABS (kips)
Level 2	Typical Area (Slab, SDL)	28,900		115		3309	
h = 16.00'	Terrace Area (Slab, SDL)	1,380		137		189	
	Parapet Above		31		53	2	
	Exterior Wall Above		795		113	89	
	Exterior Wall Below		730		120	88	
	Columns Above					83	
	Columns Below					118	
	Braces Above					54	
	Braces Below					96	
					Σ =	**3,877**	**3,953**
					ETABS/Calculation =		102%

Base

Calculated Total Σ =	**20,076**	
ETABS Total Σ =		**20,550**
ETABS/Calculation =		102%

Calculated & ETABS Mass Within 5%. O.K.

ASCE 7-05
Seismic Base Shear

Hazards by Location

Search Information

Address: 600 16th St, San Francisco, CA 94158, USA

Coordinates: 37.76725479999999, -122.3923029

Elevation: 20 ft

Timestamp: 2020-03-14T01:49:48.324Z

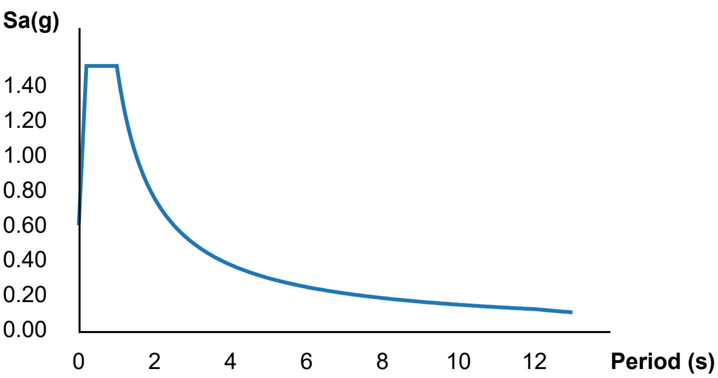
Hazard Type: Seismic

Reference Document: ASCE7-05

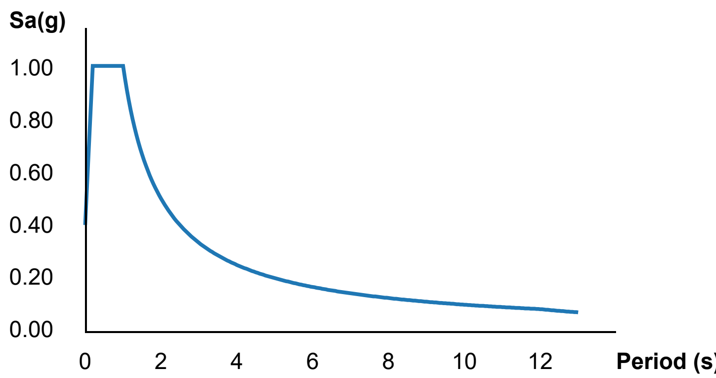
Risk Category: II

Site Class: E

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	1.5	MCE_R ground motion (period=0.2s)
S_1	0.634	MCE_R ground motion (period=1.0s)
S_{MS}	1.35	Site-modified spectral acceleration value
S_{M1}	1.521	Site-modified spectral acceleration value
S_{DS}	0.9	Numeric seismic design value at 0.2s SA
S_{D1}	1.014	Numeric seismic design value at 1.0s SA

▾Additional Information

Name	Value	Description
SDC	D	Seismic design category
F_a	0.9	Site amplification factor at 0.2s

F_v	2.4	Site amplification factor at 1.0s
T_L	12	Long-period transition period (s)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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Engineering of Structures and Building Enclosures

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SUBJECT Byers Hall: CBC 2007 Base Shear X

Building Data

Occupancy Category = **II** *Table 1604.5, 2007 CBC* Importance Factor, I = **1.00** *Table 11.5-1*

Seismic Ground Motion Values *Section 11.4*

S_S =	**1.500**	*From geotech or Figs.22-1 to 14*	F_a =	**0.90**	*Table 11.4-1*
S_1 =	**0.634**	*From geotech or Figs.22-1 to 14*	F_v =	**2.40**	*Table 11.4-2*
Site Class =	**E**	*From geotech or Table 20.3-1*	$S_{MS} = F_aS_S$ =	**1.350**	*Eq. 11.4-1*
T_L =	**12 sec**	*Figs.22-15 to 20*	$S_{M1} = F_vS_1$ =	**1.522**	*Eq. 11.4-2*
			$S_{DS} = (2/3)S_{MS}$ =	**0.900**	*Eq. 11.4-3*
			$S_{D1} = (2/3)S_{M1}$ =	**1.014**	*Eq. 11.4-4*

Seismic Design Category **D** *Tables 11.6-1 & 2*

Building Period *Section 12.8.2*

C_t =	**0.02**	*Table 12.8-2*	$T_a = C_t h_n^x$ =	0.535 sec	*Eq. 12.8-7*
x =	**0.75**	*Table 12.8-2*	C_u =	1.40	*Table 12.8-1*
h_n =	**80.00 ft**	*Height of Building*	$T_{a,\,max} = C_u T_a$ =	0.749 sec	*Section. 12.8.2*
T_b =	**0.895 sec**	*From Analysis*			
		(Input zero to use Ta)	**Period =**	**0.749 sec**	**«-- used for design**
				0.895 sec	**«-- used for drift**
					Section. 12.8.6.2

Base Shear *Section 12.8*

W =	**20,550 kips**	*Total Structure Weight*
R =	**8**	*Table 12.2-1*
C_d =	**5**	*Table 12.2-1*

For Design Only				**For Drift Only**	
$C_{s,\,max}$ =	S_{DS} / (R/I) =	0.113	*Eq. 12.8-2*	$C_{s,\,max}$ =	0.113
C_s =	S_{D1} / [T (R/I)] =	0.169	*Eq. 12.8-3, for $T \le T_L$*	C_s =	0.142
C_s =	$S_{D1}\ T_L$ / [T^2 (R/I)] =	N/A	*Eq. 12.8-4, for $T > T_L$*	C_s =	N/A
$C_{s,\,min}$ =		0.040	*Eq.12.8-5(per Suppl. No.2 to ASCE 7-(*	$C_{s,\,min}$ =	0.040
$C_{s,\,min}$ =	$0.5S_1/(R/I)$ =	0.040	*Eq. 12.8-6, if $S_1 \ge 0.6g$*	$C_{s,\,min}$ =	0.040
	Use, C_s =	**0.113**		**Use, C_s =**	**0.113**
	$V_{design} = C_sW$ =	**2,312 kips**	*Eq. 12.8-1*	**$V_{drift} = C_sW$ =**	**2,312 kips**
				Allowable Drift =	**0.020** ***h_{sx}***
					Table 12.12-1

* *Note : All references are from ASCE 7-05 unless noted otherwise.*

Building Data

Occupancy Category =	II	*Table 1604.5, 2007 CBC*	Importance Factor, I =	**1.00**	*Table 11.5-1*

Seismic Ground Motion Values

Section 11.4

S_S =	**1.500**	*From geotech or Figs.22-1 to 14*	F_a =	**0.90**	*Table 11.4-1*
S_1 =	**0.634**	*From geotech or Figs.22-1 to 14*	F_v =	**2.40**	*Table 11.4-2*
Site Class =	**E**	*From geotech or Table 20.3-1*	$S_{MS} = F_aS_S$ =	**1.350**	*Eq. 11.4-1*
T_L =	**12 sec**	*Figs.22-15 to 20*	$S_{M1} = F_vS_1$ =	**1.522**	*Eq. 11.4-2*
			$S_{DS} = (2/3)S_{MS}$ =	**0.900**	*Eq. 11.4-3*
			$S_{D1} = (2/3)S_{M1}$ =	**1.014**	*Eq. 11.4-4*

Seismic Design Category

D *Tables 11.6-1 & 2*

Building Period

Section 12.8.2

C_t =	**0.02**	*Table 12.8-2*	$T_a = C_t\, h_n^{\,x}$ =	0.535 sec	*Eq. 12.8-7*
x =	**0.75**	*Table 12.8-2*	C_u =	1.40	*Table 12.8-1*
h_n =	**80.00 ft**	*Height of Building*	$T_{a,\,max} = C_u\, T_a$ =	0.749 sec	*Section. 12.8.2*
T_b =	**0.806 sec**	*From Analysis*			
		(Input zero to use Ta)	**Period =**	**0.749 sec**	**«-- used for design**
				0.806 sec	**«-- used for drift**
					Section. 12.8.6.2

Base Shear

Section 12.8

W =	**20,550 kips**	*Total Structure Weight*
R =	**8**	*Table 12.2-1*
C_d =	**5**	*Table 12.2-1*

For Design Only

$C_{s,\,max}$ =	S_{DS} / (R/I) =	0.113	*Eq. 12.8-2*	$C_{s,\,max}$ =	0.113
C_s =	S_{D1} / [T (R/I)] =	0.169	*Eq. 12.8-3, for $T \le T_L$*	C_s =	0.157
C_s =	$S_{D1}\, T_L$ / [T^2 (R/I)] =	N/A	*Eq. 12.8-4, for $T > T_L$*	C_s =	N/A
$C_{s,\,min}$ =		0.040	*Eq.12.8-5(per Suppl. No.2 to ASCE 7-(*	$C_{s,\,min}$ =	0.040
$C_{s,\,min}$ =	$0.5S_1$/(R/I) =	0.040	*Eq. 12.8-6, if $S_1 \ge 0.6g$*	$C_{s,\,min}$ =	0.040
	Use, C_s =	**0.113**		**Use, C_s =**	**0.113**

For Drift Only

$V_{design} = C_sW$ = 2,312 kips *Eq. 12.8-1*

$V_{drift} = C_sW$ = 2,312 kips

Allowable Drift = 0.020 h_{sx}

Table 12.12-1

** Note: All references are from ASCE 7-05 unless noted otherwise.*

Scale Factor

Dynamic Analysis:

Rotate Axes = 0 (degree)
Modal Combination = CQC
Damping Ratio = 5 %
Occupancy Category = II
Site Class = E
F_a = 0.9
F_v = 2.4

For Design Force:

Direction	Base Shear: V_{static} (Sec. 12.8, ASCE 7-05) (kips)	Base Shear: $V_{dynamic}$ (Sec. 12.9, ASCE 7-05) (kips)	R	I	$V_{dynamic}$ (I/R) (kips)	$0.85V_{static}$ (kips)	**Design Base Shear** (Sec. 12.9.4, ASCE 7-05) Max of $V_{dynamic}$ (I/R) and $0.85V_{static}$ (kips)	**Scale Factor**
X	2,312	12,815	8.0	1.0	1,602	1,965	**1,965**	**0.1533**
Y	2,312	14,390	8.0	1.0	1,799	1,965	**1,965**	**0.1366**

Note: If $V_{dynamic}$ (I/R) ≥ $0.85V_{static}$, Scale factor = I/R
If $V_{dynamic}$ (I/R) < $0.85V_{static}$, Scale factor = $0.85V_{static}/V_{dynamic}$
Per Section 12.9.4, ASCE 7-05

For Drift :

Direction	$V_{dynamic}$ (Sec. 12.9, ASCE 7-(kips)	R	I	**Scale Factor (= I/R)**	**Design Base Shear** (Sec. 12.9, ASCE 7-05) (kips)
X	12,815	8.0	1.0	**0.1250**	**1,602**
Y	14,390	8.0	1.0	**0.1250**	**1,799**

Note: Scale factor = I/R for all cases

ETABS Model

1. LATERAL ANALYSIS MODEL DESCRIPTION

- Lateral analysis is performed using a three-dimensional structural model in the computer analysis program ETABS v18.0.2.
- Response Spectrum Analysis per 2007 California Building Code and ASCE 7-05 is used for seismic loads, including actual & accidental torsion effects. At least 90 percent of the participating mass of the structure is included in the calculation of the response for each principal horizontal direction. The modal response is combined using the CQC (Complete Quadratic Combination) procedure.
    - Inherent torsion is automatically included by ETABS in dynamic analysis.
    - Accidental torsion is applied in the response spectrum load cases as an Ax*5% eccentricity to the diaphragm
    - The ETABS non-iterative P-Delta option based on mass is used.
    - A modal damping ratio of 5% is used.
- All elements believed to contribute to the lateral force resistance of the structure are modeled, including braced frames and slab diaphragms.
- All floors above Level 1 are modeled as rigid diaphragms.
- Slab on grades are not modeled.
- All floor heights are modeled as the distance from top-of-slab at one floor to top-of-slab at the next adjacent floor.

2. SEISMIC MASS

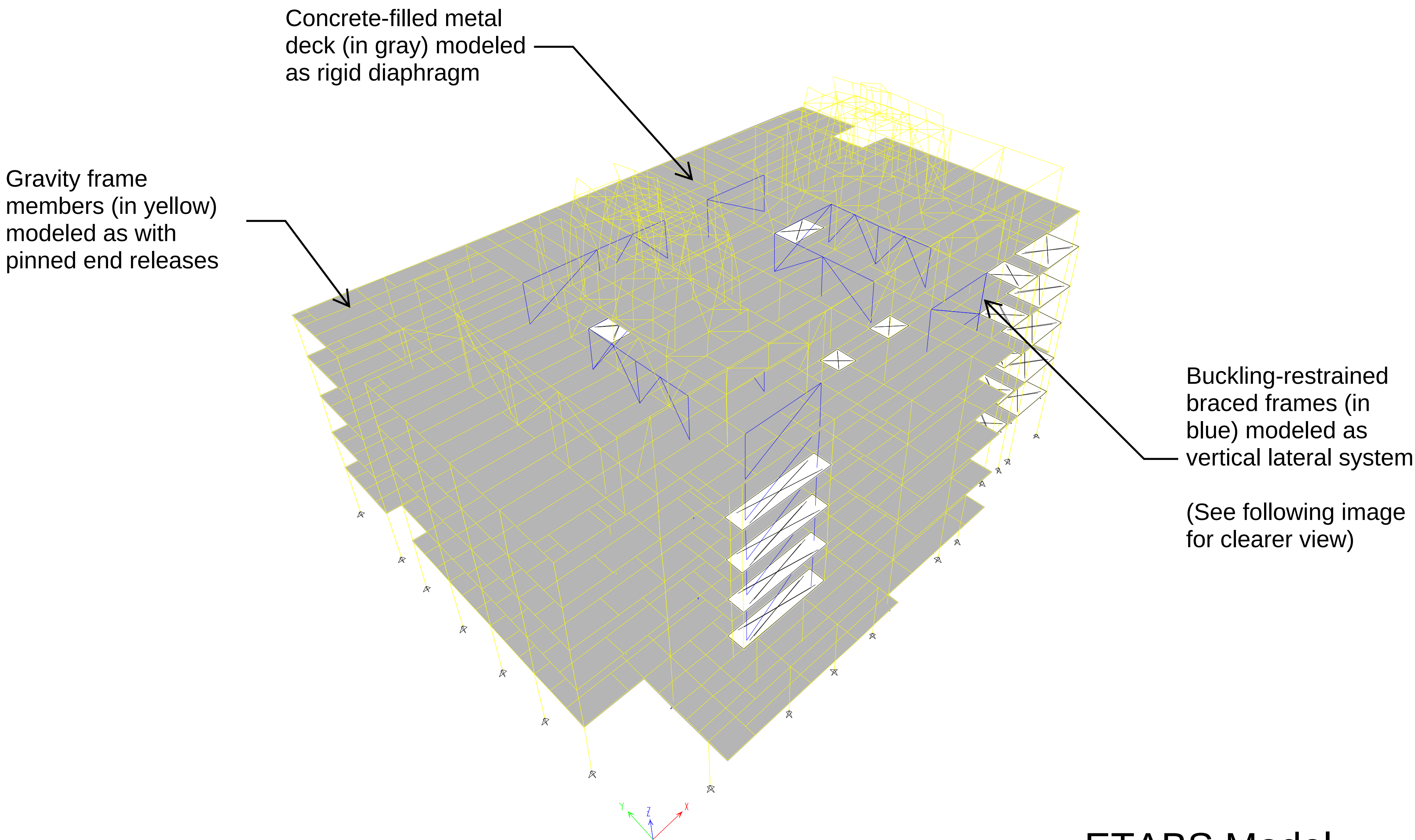
All seismic mass related to slabs, framing, and superimposed loads is applied as uniformly distributed area loads at the appropriate areas. No mass is modeled at the slab on grades.

The distributed area mass does not include the façade. The façade mass is applied as a uniform line load where it occurs at each level with half of the mass attributed to the level above and half to the level below.

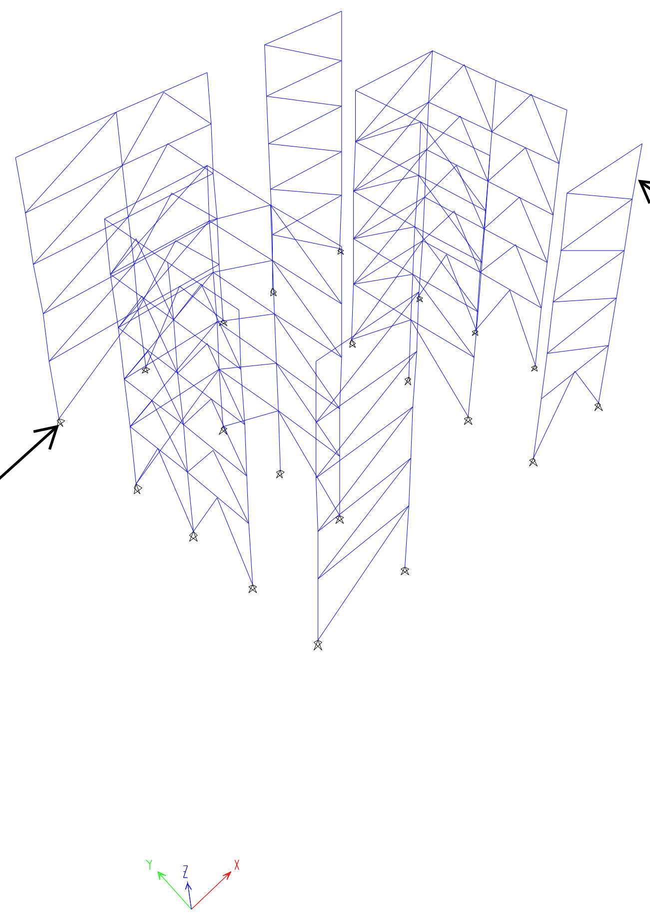
3. MODELING CRITERIA

3.1 Steel Buckling Restrained Braced Frame Modeling

- Column, beam and brace members are modeled using frame elements.
- Column and beam stiffnesses are based on the actual cross-sectional properties of the members.
- Brace ends are modeled as pinned.
- Brace stiffness is modeled using an equivalent stiffness based on the cross-sectional area and geometry of the core plate and the geometry of the connections.
- Braces are modeled from work point-to-work point using the area of the core plate.
- Modulus of elasticity is assigned to be that of steel (E=29,000 ksi) multiplied by an equivalent stiffness factor to account for the actual stiffness of the brace.
- The beam-column connections are modeled as fixed.



ETABS Model
3D Overall View



Buckling-restrained braced frames (in blue) modeled as vertical lateral system

Column bases modeled as pinned restraints

ETABS Model
3D Overall View
LFRS Only

ASCE 7-05
Torsional Irregularity Check

SIMPSON GUMPERTZ & HEGER

Engineering of Structures
and Building Enclosures

CLIENT UCSF

SUBJECT Byers Hall: Torsional Irregularity & Amplification

SHEET NO. ______

PROJECT NO. 197042.00-UCSF

DATE 03/13/2020

BY LZhou

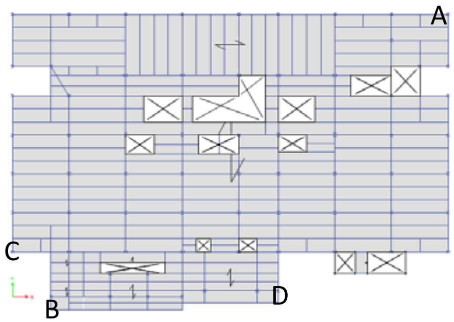
CHECKED BY ______

X DIRECTION (Inherent Torsion)

$drift_{allowable}$ = 2.0% (ASCE 7-05, Table 12.12-1)

I = 1.00 (ASCE 7-05, §11.5.1)

C_d = 5 (ASCE 7-05, Table 12.2-1)

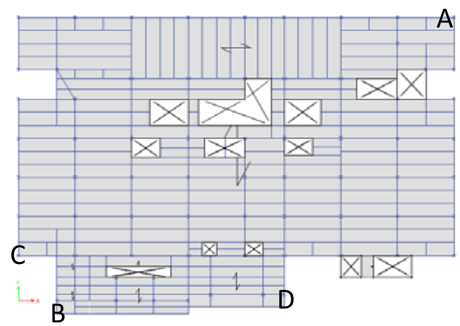


POINT ID	POINT	Location
697	A	N-E
13	B	S-W
76	C	S-W
26	D	S

Level	Story Height	Corner Point ID	Corner Point	Load Case	Displ, δ (in.)	Drift, Δ	Δ_{max}	Δ_{avg}	$\Delta_{max}/\Delta_{avg.}$	Check	δ_{max}	δ_{avg}	A_x	Ecc. (%) $5\%xA_x$
Roof	15.0	697	A	ELF X	1.29	0.265	0.429	0.347	1.24	Ax required	1.861	1.575	1.00	5.00
	15.0	26	D		1.86	0.429								
Level 5	15.0	697	A	ELF X	1.02	0.297	0.457	0.377	1.21	Ax required	1.432	1.227	1.00	5.00
	15.0	26	D		1.43	0.457								
Level 4	15.0	697	A	ELF X	0.73	0.270	0.403	0.337	1.20	Ax not required	0.975	0.850	1.00	5.00
	15.0	26	D		0.97	0.403								
Level 3	15.0	697	A	ELF X	0.46	0.237	0.334	0.285	1.17	Ax not required	0.572	0.514	1.00	5.00
	15.0	26	D		0.57	0.334								
Level 2	20.0	697	A	ELF X	0.22	0.219	0.238	0.229	1.04	Ax not required	0.238	0.229	1.00	5.00
	20.0	13	B		0.24	0.238								

Y DIRECTION (Inherent Torsion)

$drift_{allowable}$ = 2.0% (ASCE 7-05, Table 12.12-1)
I = 1.00 (ASCE 7-05, §11.5.1)
C_d = 5 (ASCE 7-05, Table 12.2-1)

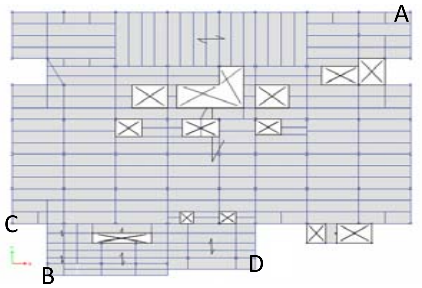


POINT ID	POINT	Location
697	A	N-E
13	B	S-W
76	C	S-W
26	D	S

Level	Story Height	Corner Point ID	Corner Point	Load Case	Displ, δ (in.)	Drift, Δ	Δ_{max}	Δ_{avg}	$\Delta_{max}/\Delta_{avg.}$	Check	δ_{max}	δ_{avg}	A_x	Ecc. (%) $5\%xA_x$
Roof	15.0	697	A	ELF Y	1.17	0.289	0.289	0.286	1.01	Ax not required	1.165	1.160	1.00	5.00
	15.0	76	C		1.16	0.283								
Level 5	15.0	697	A	ELF Y	0.88	0.275	0.275	0.274	1.01	Ax not required	0.876	0.875	1.00	5.00
	15.0	76	C		0.87	0.272								
Level 4	15.0	697	A	ELF Y	0.60	0.217	0.217	0.216	1.01	Ax not required	0.601	0.601	1.00	5.00
	15.0	76	C		0.60	0.215								
Level 3	15.0	697	A	ELF Y	0.38	0.197	0.197	0.196	1.00	Ax not required	0.386	0.385	1.00	5.00
	15.0	76	C		0.39	0.195								
Level 2	20.0	697	A	ELF Y	0.19	0.187	0.191	0.189	1.01	Ax not required	0.191	0.189	1.00	5.00
	20.0	76	C		0.19	0.191								

X DIRECTION

$drift_{allowable}$ = 2.0% (ASCE 7-05, Table 12.12-1)
I = 1.00 (ASCE 7-05, §11.5.1)
C_d = 5 (ASCE 7-05, Table 12.2-1)



POINT ID	POINT	Location
697	A	N-E
13	B	S-W
76	C	S-W
26	D	S

Level	Story Height	Corner Point ID	Corner Point	Load Case	Displ, δ (in.)	Drift, Δ	Δ_{max}	Δ_{avg}	$\Delta_{max}/\Delta_{avg.}$	Check	δ_{max}	δ_{avg}	A_x	Ecc. (%) $5\%xA_x$
Roof	15.0	697	A	ELF X+ECC	1.48	0.309	0.359	0.334	1.07	Ax not required	1.582	1.529	1.00	5.00
	15.0	76	C		1.58	0.359								
	15.0	697	A	ELF X-ECC	1.10	0.221	0.453	0.337	1.34	Ax required	1.979	1.540	1.15	5.74
	15.0	76	C		1.98	0.453								
Level 5	15.0	697	A	ELF X+ECC	1.17	0.340	0.387	0.364	1.06	Ax not required	1.223	1.195	1.00	5.00
	15.0	76	C		1.22	0.387								
	15.0	697	A	ELF X-ECC	0.88	0.253	0.483	0.368	1.31	Ax required	1.526	1.202	1.12	5.59
	15.0	76	C		1.53	0.483								
Level 4	15.0	697	A	ELF X+ECC	0.83	0.305	0.345	0.325	1.06	Ax not required	0.836	0.831	1.00	5.00
	15.0	76	C		0.84	0.345								
	15.0	697	A	ELF X-ECC	0.63	0.235	0.424	0.330	1.29	Ax required	1.043	0.834	1.09	5.43
	15.0	76	C		1.04	0.424								
Level 3	15.0	697	A	ELF X+ECC	0.52	0.269	0.285	0.277	1.03	Ax not required	0.521	0.506	1.00	5.00
	15.0	76	C		0.49	0.285								
	15.0	697	A	ELF X-ECC	0.39	0.205	0.355	0.280	1.27	Ax required	0.619	0.505	1.05	5.23
	15.0	76	C		0.62	0.355								
Level 2	20.0	697	A	ELF X+ECC	0.25	0.252	0.252	0.229	1.10	Ax not required	0.253	0.229	1.00	5.00
	20.0	76	C		0.21	0.206								
	20.0	697	A	ELF X-ECC	0.19	0.185	0.264	0.225	1.18	Ax not required	0.264	0.225	1.00	5.00
	20.0	76	C		0.26	0.264								

$\Delta_{max}/\Delta_{avg}$ does not exceed 1.4, therefore, Extreme Torsional Irregularity (Horizontal Irregularity Type 1b) does not exist.
Removal of a brace does not result in a 33% loss in story strength and the resulting system does not have an Extreme Torsional Irregularity.
The redundancy factor, ρ, for the X direction may be taken as 1.0.

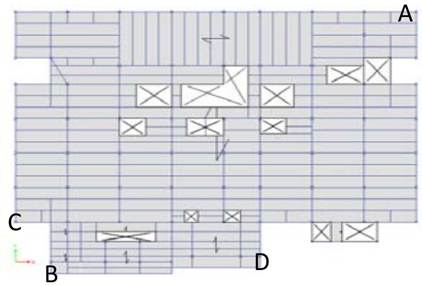
SIMPSON GUMPERTZ & HEGER
Engineering of Structures
and Building Enclosures

CLIENT UCSF
SUBJECT Byers Hall: Torsional Irregularity & Amplification

SHEET NO.
PROJECT NO. 197042.00-UCSF
DATE 03/13/2020
BY LZhou
CHECKED BY

Y DIRECTION

$drift_{allowable}$ = 2.0% (ASCE 7-05, Table 12.12-1)
I = 1.00 (ASCE 7-05, §11.5.1)
C_d = 5 (ASCE 7-05, Table 12.2-1)



POINT ID	POINT	Location
697	A	N-E
13	B	S-W
76	C	S-W
26	D	S

Level	Story Height	Corner Point ID	Corner Point	Load Case	Displ, δ (in.)	Drift, Δ	Δ_{max}	Δ_{avg}	$\Delta_{max}/\Delta_{avg.}$	Check	δ_{max}	δ_{avg}	A_x	Ecc. (%) $5\%xA_x$
Roof	15.0	697	A	ELF Y+ECC	1.60	0.391	0.391	0.286	1.37	Ax required	1.602	1.160	1.32	6.62
	15.0	76	C		0.72	0.180								
	15.0	697	A	ELF Y-ECC	0.73	0.186	0.386	0.286	1.35	Ax required	1.593	1.161	1.31	6.54
	15.0	76	C		1.59	0.386								
Level 5	15.0	697	A	ELF Y+ECC	1.21	0.378	0.378	0.274	1.38	Ax required	1.211	0.874	1.33	6.66
	15.0	76	C		0.54	0.169								
	15.0	697	A	ELF Y-ECC	0.54	0.173	0.375	0.274	1.37	Ax required	1.208	0.875	1.32	6.62
	15.0	76	C		1.21	0.375								
Level 4	15.0	697	A	ELF Y+ECC	0.83	0.302	0.302	0.216	1.40	Ax required	0.833	0.601	1.33	6.67
	15.0	76	C		0.37	0.130								
	15.0	697	A	ELF Y-ECC	0.37	0.133	0.300	0.216	1.39	Ax required	0.833	0.601	1.33	6.67
	15.0	76	C		0.83	0.300								
Level 3	15.0	697	A	ELF Y+ECC	0.53	0.272	0.272	0.196	1.39	Ax required	0.531	0.385	1.32	6.61
	15.0	76	C		0.24	0.120								
	15.0	697	A	ELF Y-ECC	0.24	0.121	0.271	0.196	1.38	Ax required	0.533	0.385	1.33	6.67
	15.0	76	C		0.53	0.271								
Level 2	20.0	697	A	ELF Y+ECC	0.26	0.258	0.258	0.189	1.37	Ax required	0.258	0.189	1.30	6.50
	20.0	76	C		0.12	0.120								
	20.0	697	A	ELF Y-ECC	0.12	0.115	0.262	0.189	1.39	Ax required	0.262	0.189	1.34	6.70
	20.0	76	C		0.26	0.262								

$\Delta_{max}/\Delta_{avg}$ does not exceed 1.4, therefore, Extreme Torsional Irregularity (Horizontal Irregularity Type 1b) does not exist.
Removal of a brace does not result in a 33% loss in story strength. The resulting system borders on having an Extreme Torsional Irregularity.
By engineering judgement due to the number of braces and low inherent torsion, the redundancy factor, ρ, for the X direction may be taken as 1.0.

ASCE 7-05
Drift Check

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

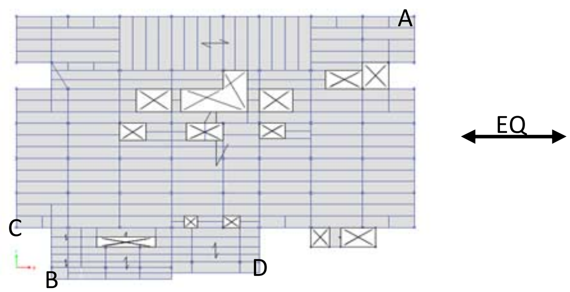
CLIENT UCSF
SUBJECT Byers Hall: Drift Check

SHEET NO.
PROJECT NO. 197042.00-UCSF
DATE 03/13/2020
BY LZhou
CHECKED

X DIRECTION

Lateral System = Buckling Restrained Braced Frame
Occupancy Category = II
Importance Factor, I = 1.00
Seismic Design Category = D
Redundancy Factor, ρ = 1.0
Response Modification Coeff., R = 8.0
Deflection Amp. Factor, C_d = 5.0
Allowable Story Drift, Δ_a = 2.0%
Design Allowable Story Drift, Δ_a = 2.0%

Check Drift at: Corner Points



Point A

Load Case	Story	Story Height Below (ft)	Point ID	Elastic Deflection δxe (in.)	Elastic Drift Ratio Δxe	Ultimate Deflection δ*Cd/I	Ultimate Drift Ratio Δ*Cd/I	CHECK
DRIFTX-ECC	Roof	15.0	697	1.569	0.20%	7.847	0.98%	OK
DRIFTX-ECC	Level 5	15.0	697	1.225	0.20%	6.123	1.02%	OK
DRIFTX-ECC	Level 4	15.0	697	0.864	0.17%	4.322	0.87%	OK
DRIFTX-ECC	Level 3	15.0	697	0.554	0.15%	2.772	0.77%	OK
DRIFTX-ECC	Level 2	20.0	697	0.277	0.12%	1.383	0.58%	OK

Point B or D

Load Case	Story	Story Height Below (ft)	Point ID	Elastic Deflection δxe (in.)	Elastic Drift Ratio Δxe	Ultimate Deflection δ*Cd/I	Ultimate Drift Ratio Δ*Cd/I	CHECK
DRIFTX-ECC	Roof	15.0	26	1.472	0.20%	7.358	1.01%	OK
DRIFTX-ECC	Level 5	15.0	26	1.115	0.20%	5.577	1.02%	OK
DRIFTX-ECC	Level 4	15.0	26	0.754	0.17%	3.771	0.86%	OK
DRIFTX-ECC	Level 3	15.0	26	0.449	0.14%	2.244	0.71%	OK
DRIFTX-ECC	Level 2	20.0	13	0.199	0.08%	0.996	0.42%	OK

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SHEET NO.	
PROJECT NO.	197042.00-UCSF
DATE	03/12/2020
BY	LZhou
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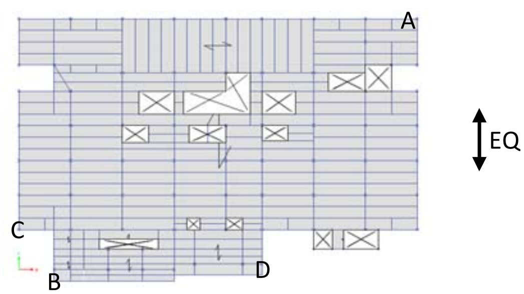
CLIENT UCSF

SUBJECT Byers Hall: Drift Check

Y DIRECTION

Lateral System = Buckling Restrained Braced Frame
Occupancy Category = II
Importance Factor, I = 1.00
Seismic Design Category = D
Redundancy Factor, ρ = 1.0
Response Modification Coeff., R = 8.0
Deflection Amp. Factor, C_d = 5.0
Allowable Story Drift, Δ_a = 2.0%
Design Allowable Story Drift, Δ_a = 2.0%

Check Drift at: Corner Points



Point A								
Load Case	Story	Story Height Below (ft)	Point ID	Elastic Deflection δxe (in.)	Elastic Drift Ratio Δxe	Ultimate Deflection δ*Cd/I	Ultimate Drift Ratio Δ*Cd/I	CHECK
DRIFTY-ECC	Roof	15.0	697	1.578	0.23%	7.889	1.13%	OK
DRIFTY-ECC	Level 5	15.0	697	1.178	0.21%	5.890	1.05%	OK
DRIFTY-ECC	Level 4	15.0	697	0.804	0.16%	4.021	0.82%	OK
DRIFTY-ECC	Level 3	15.0	697	0.511	0.15%	2.557	0.73%	OK
DRIFTY-ECC	Level 2	20.0	697	0.249	0.10%	1.247	0.52%	OK

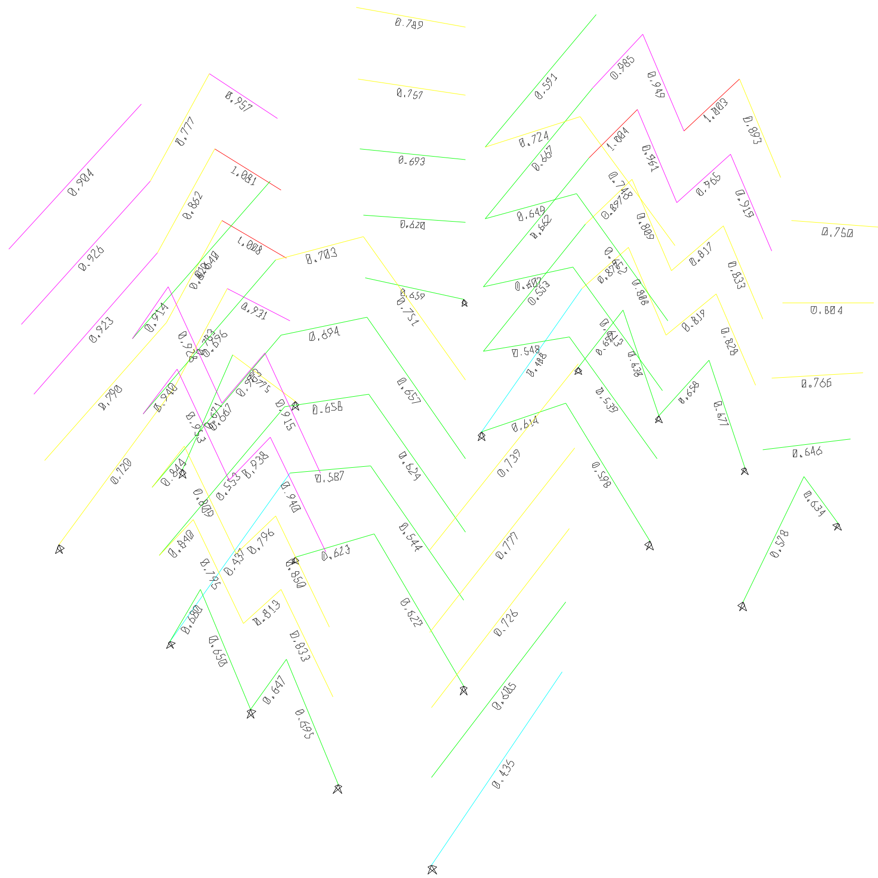
Point C								
Load Case	Story	Story Height Below (ft)	Point ID	Elastic Deflection δxe (in.)	Elastic Drift Ratio Δxe	Ultimate Deflection δ*Cd/I	Ultimate Drift Ratio Δ*Cd/I	CHECK
DRIFTY-ECC	Roof	15.0	76	1.568	0.22%	7.838	1.12%	OK
DRIFTY-ECC	Level 5	15.0	76	1.171	0.21%	5.857	1.04%	OK
DRIFTY-ECC	Level 4	15.0	76	0.800	0.16%	3.999	0.81%	OK
DRIFTY-ECC	Level 3	15.0	76	0.509	0.14%	2.543	0.72%	OK
DRIFTY-ECC	Level 2	20.0	76	0.249	0.10%	1.244	0.52%	OK

ASCE 7-05/AISC 360-05
LFRS Standard Provisions

Strength Checks

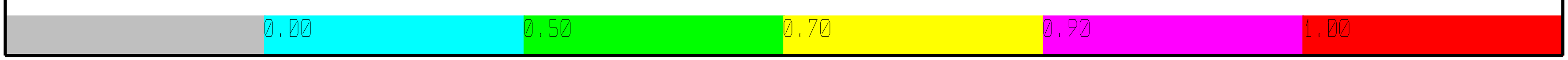
Max DCR = 1.08

Note: DCRs shown conservatively consider gravity loads on chevron bracing and seismic 100+30 directional combinations.



BRACE DCRs

See braced frame elevations in following pages for detailed results

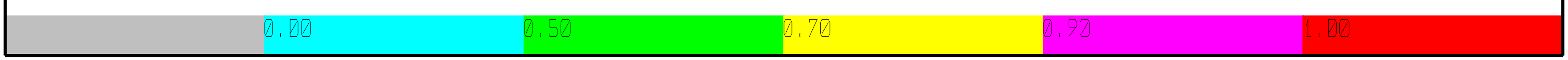


Max DCR = 1.05

Note: DCRs shown here do not include load combinations including mechanisms due to adjusted brace strength as required per AISC 341-05. See detailed frame calculations.

Column DCRs

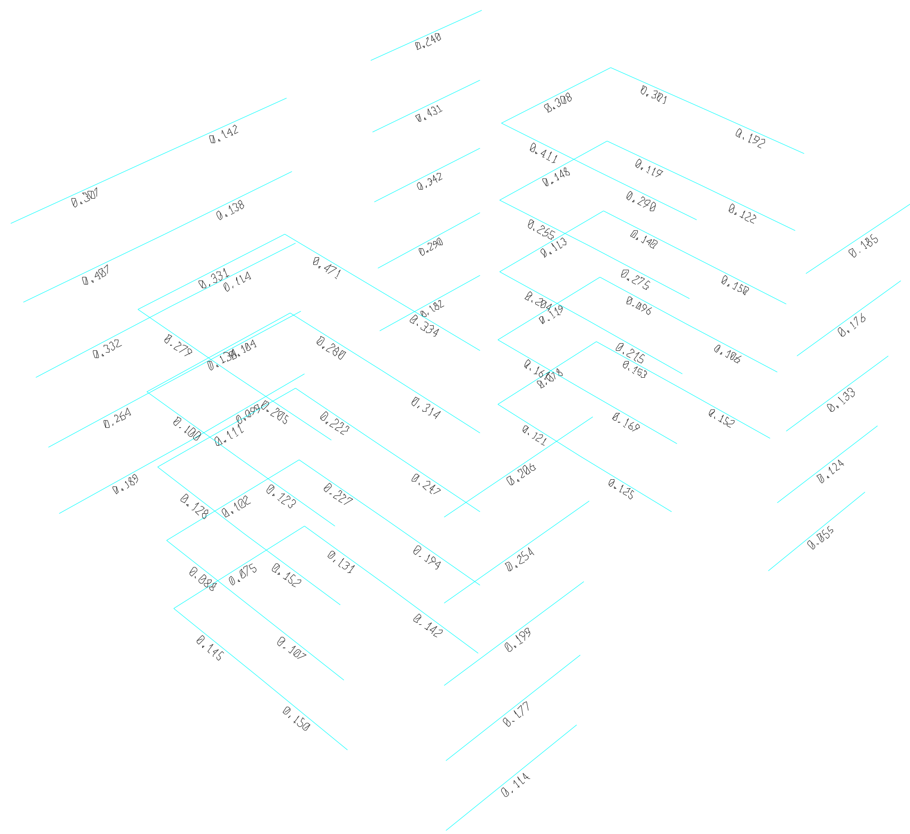
See braced frame elevations in following pages for detailed results



Max DCR < 0.5

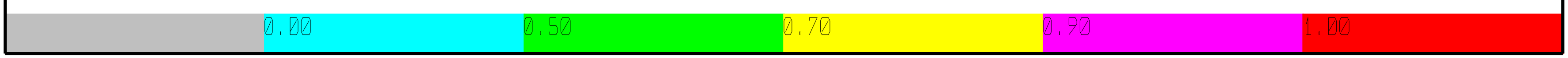
Notes:
1. DCRs shown conservatively consider seismic 100+30 directional combinations.

2. DCRs shown here do not include load combinations including mechanisms due to adjusted brace strength as required per AISC 341-05. See detailed frame calculations.



BEAM DCRs

See braced frame elevations in following pages for detailed results

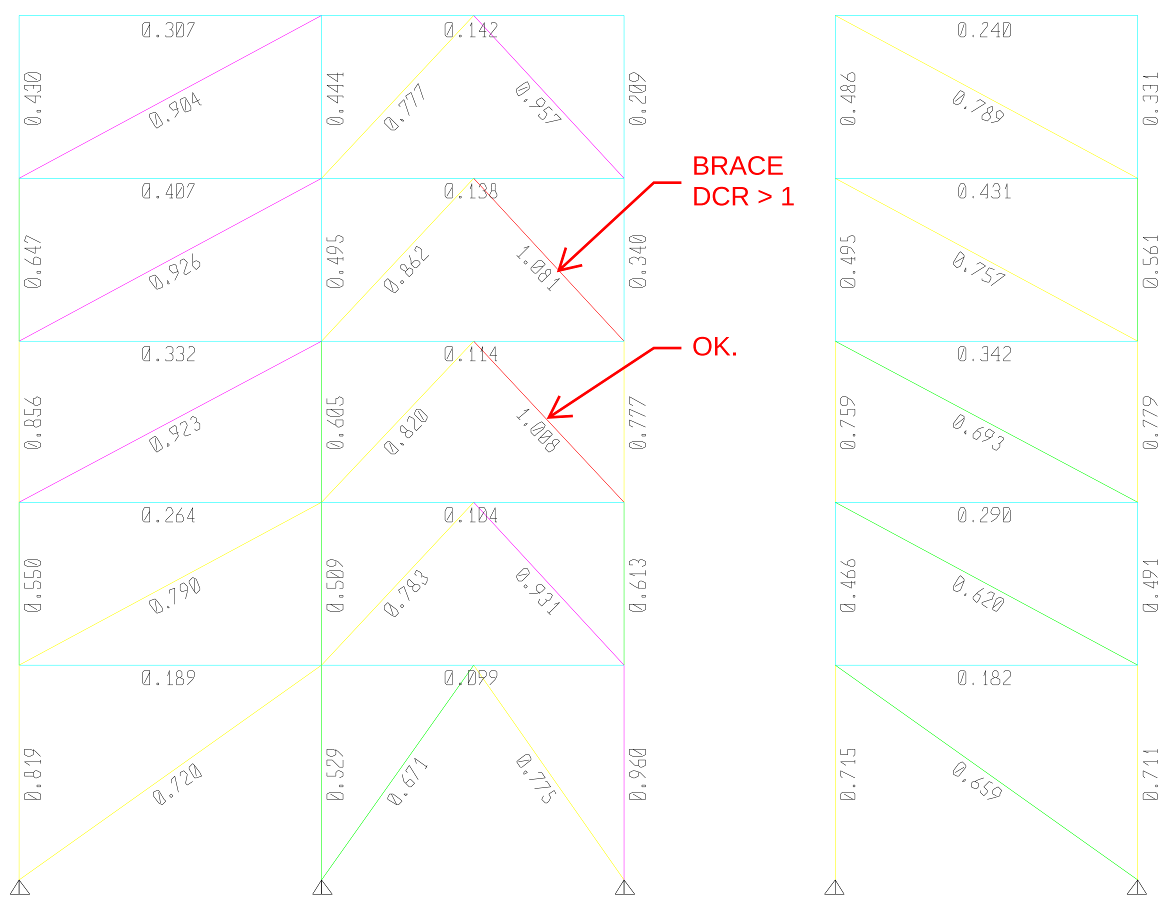


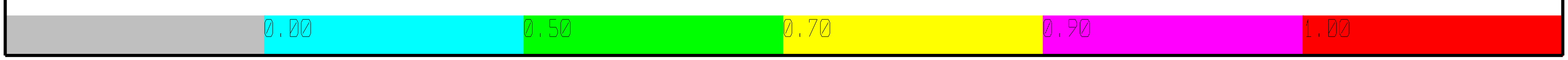
FRAME ELEVATION
GRID LINE B DCRs

Notes:
1. DCRs shown conservatively consider gravity loads on bracing.

2. DCRs shown conservatively consider seismic 100+30 directional combinations on all elements, including beams and braces.

3. DCRs shown here do not include load combinations including mechanisms due to adjusted brace strength as required per AISC 341-05. See detailed frame calculations.





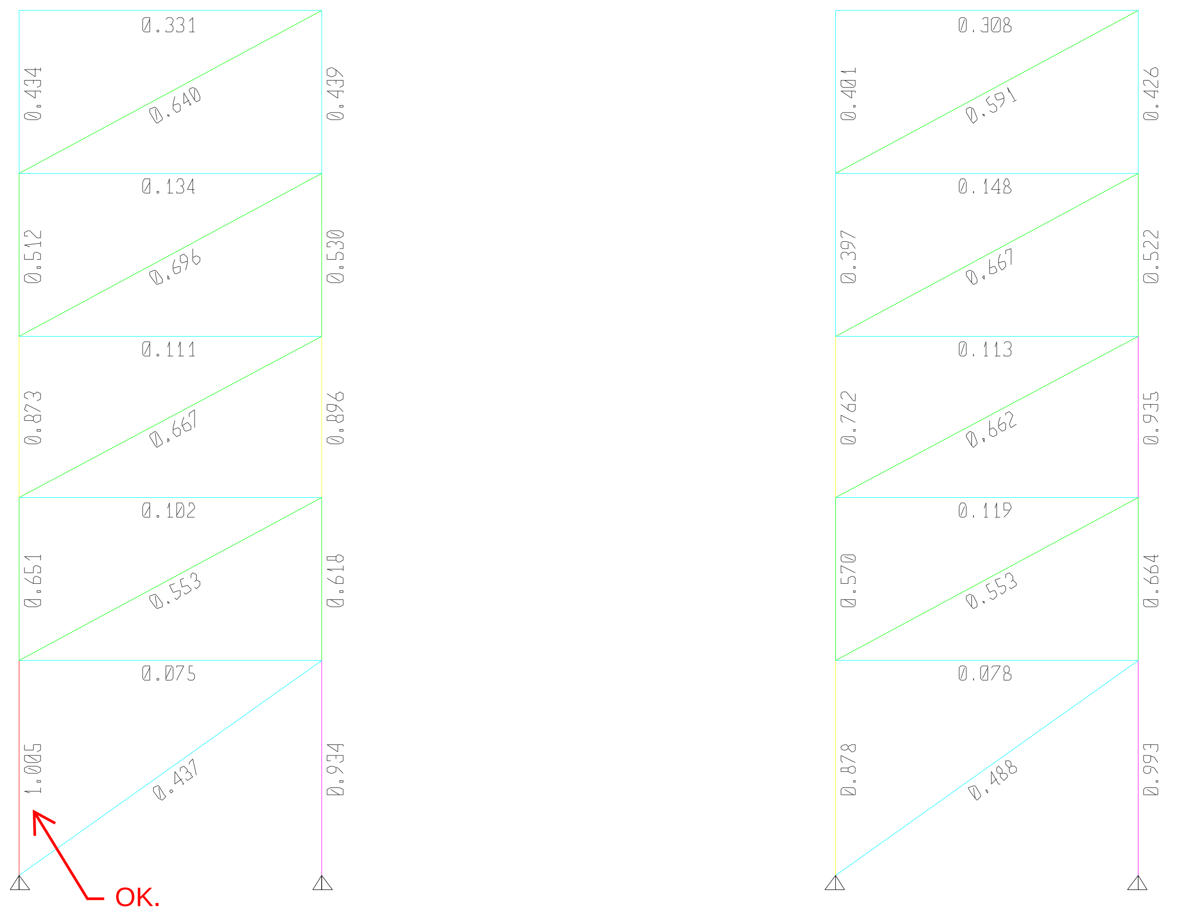
FRAME ELEVATION
GRID LINE D DCRs

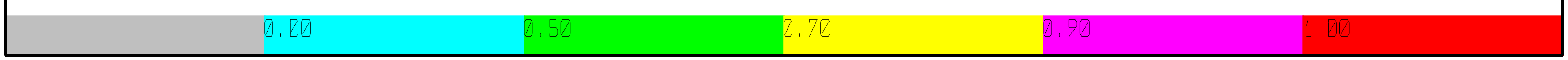
Notes:

1. DCRs shown conservatively consider gravity loads on bracing.

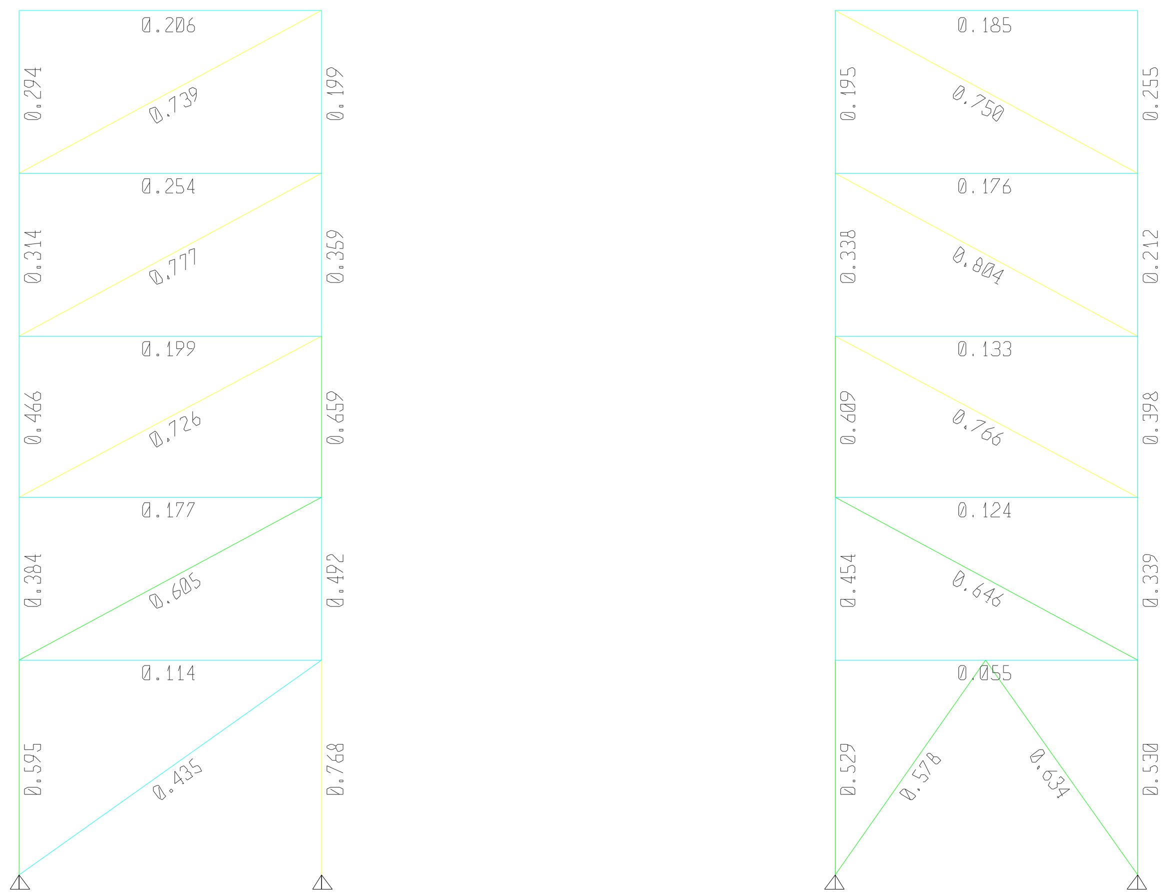
2. DCRs shown conservatively consider seismic 100+30 directional combinations on all elements, including beams and braces.

3. DCRs shown here do not include load combinations including mechanisms due to adjusted brace strength as required per AISC 341-05. See detailed frame calculations.





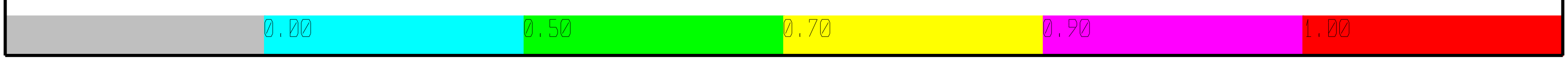
FRAME ELEVATION
GRID LINE G DCRs



Notes:
1. DCRs shown conservatively consider gravity loads on bracing.

2. DCRs shown conservatively consider seismic 100+30 directional combinations on all elements, including beams and braces.

3. DCRs shown here do not include load combinations including mechanisms due to adjusted brace strength as required per AISC 341-05. See detailed frame calculations.

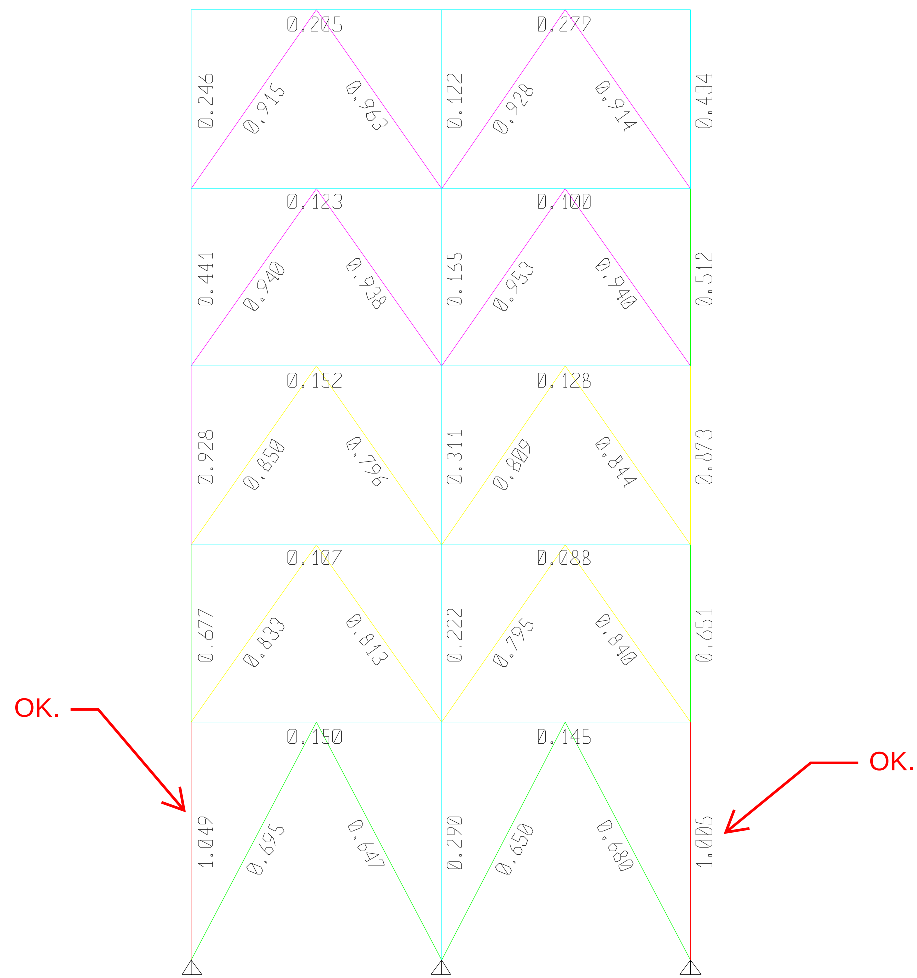


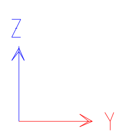
FRAME ELEVATION
GRID LINE 3 DCRs

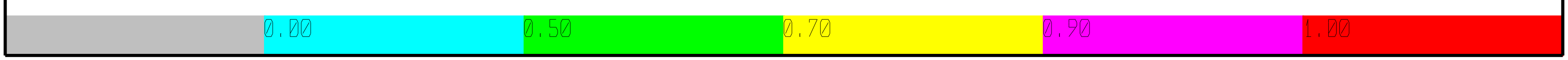
Notes:
1. DCRs shown conservatively consider gravity loads on bracing.

2. DCRs shown conservatively consider seismic 100+30 directional combinations on all elements, including beams and braces.

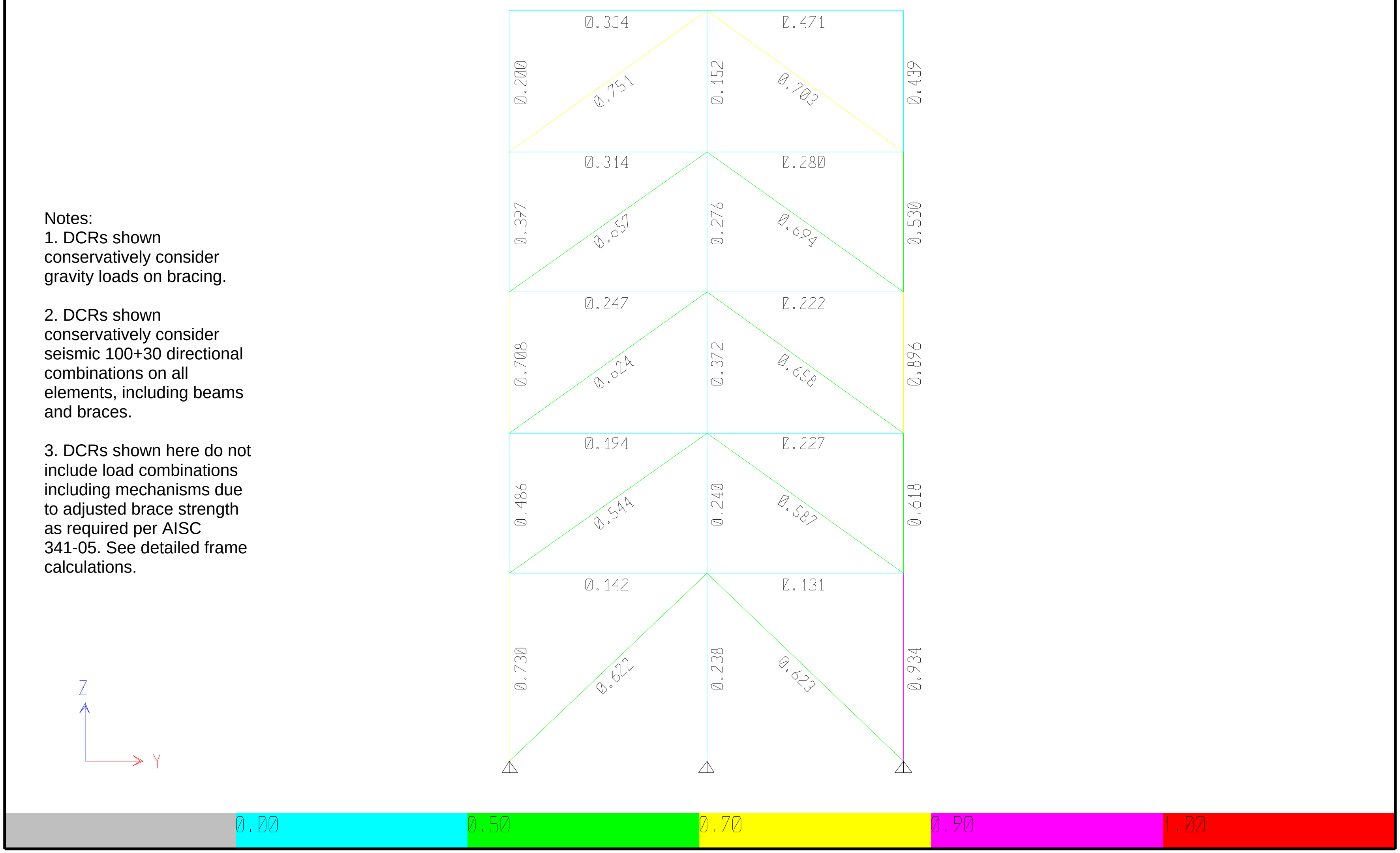
3. DCRs shown here do not include load combinations including mechanisms due to adjusted brace strength as required per AISC 341-05. See detailed frame calculations.







FRAME ELEVATION
GRID LINE 4 DCRs



Notes:
1. DCRs shown conservatively consider gravity loads on bracing.

2. DCRs shown conservatively consider seismic 100+30 directional combinations on all elements, including beams and braces.

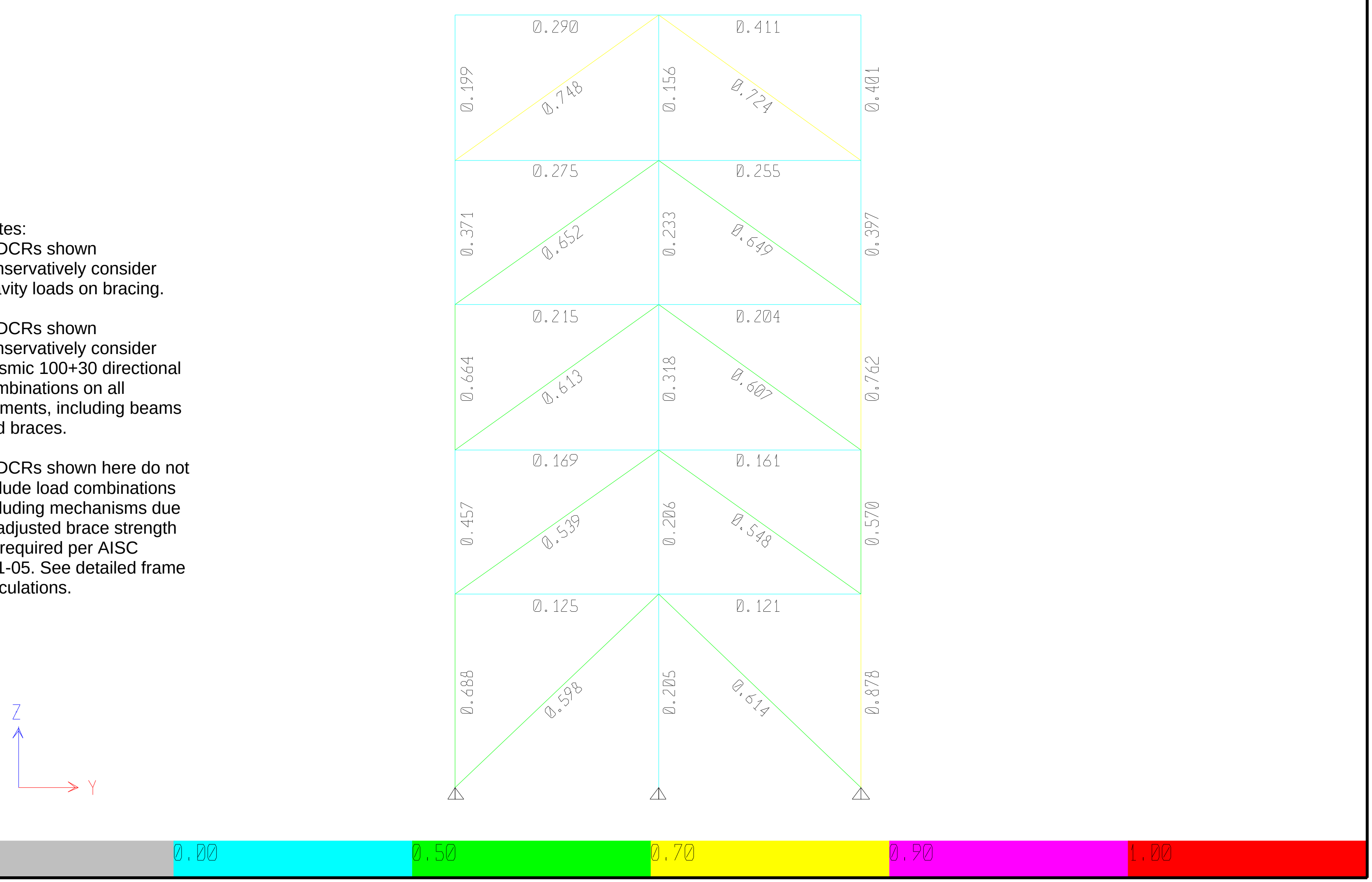
3. DCRs shown here do not include load combinations including mechanisms due to adjusted brace strength as required per AISC 341-05. See detailed frame calculations.

FRAME ELEVATION
GRID LINE 6 DCRs

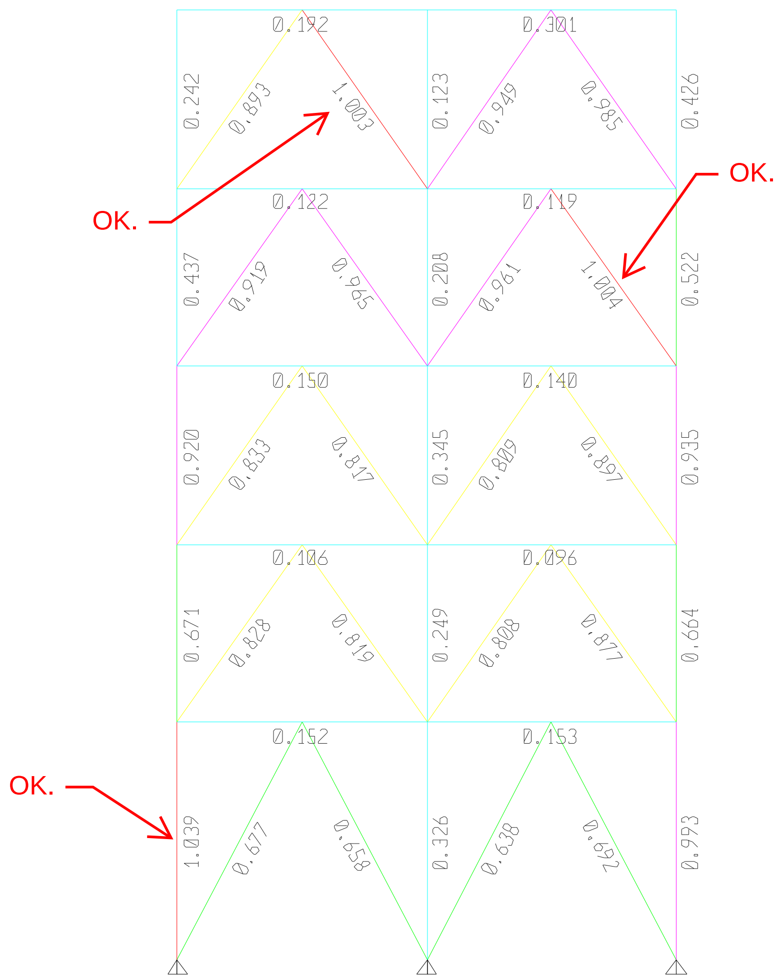
Notes:
1. DCRs shown conservatively consider gravity loads on bracing.

2. DCRs shown conservatively consider seismic 100+30 directional combinations on all elements, including beams and braces.

3. DCRs shown here do not include load combinations including mechanisms due to adjusted brace strength as required per AISC 341-05. See detailed frame calculations.



FRAME ELEVATION
GRID LINE 7 DCRs

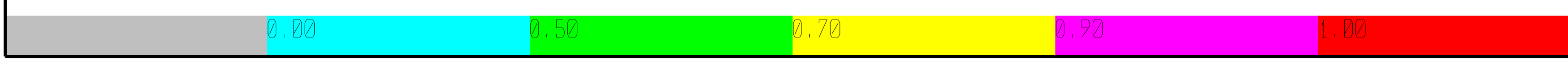


Notes:
1. DCRs shown conservatively consider gravity loads on bracing.

2. DCRs shown conservatively consider seismic 100+30 directional combinations on all elements, including beams and braces.

3. DCRs shown here do not include load combinations including mechanisms due to adjusted brace strength as required per AISC 341-05. See detailed frame calculations.

Z
Y



ASCE 7-05/AISC 341-05
LFRS Seismic Provisions

Detailed Frame Checks

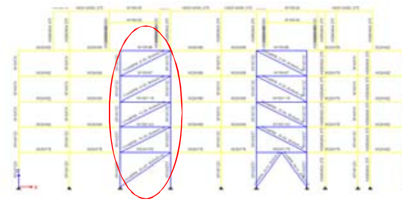
GL G/3-4 Diagonal

SIMPSON GUMPERTZ & HEGER
Engineering of Structures
and Building Enclosures

CLIENT UCSF
SUBJECT Byers Hall: BRBF Design

SHEET NO. L
PROJECT NO. 197042.00-UCSF
DATE 03/17/2020
BY LZhou
CHECKED

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL G/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BRBF GEOMETRY:

	Level 2	Level 3	Level 4	Level 5	Roof	
L(ft) =	28.00	28.00	28.00	28.00	28.00	Bay Width (Column C-C)
h_i (ft) =	20.00	15.00	15.00	15.00	15.00	Story Height
L_{diag} (ft)=	34.4	31.8	31.8	31.8	31.8	Work Point - Work Point
cos ψ =	0.814	0.881	0.881	0.881	0.881	ψ = angle between brace and horizontal axis
sin ψ =	0.581	0.472	0.472	0.472	0.472	

BRACE DESIGN

AISC 341-05 Section 16.2a - Brace Strength

	D1	D1	D1	D1	D1	ETABS Brace ID
F_{ysc} =	34 ksi	34 ksi	34 ksi	34 ksi	34 ksi	Minimum yield stress of the steel core
F_{ymax} =	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	Max yield stress of the steel core (R_yF_{ysc}; R_y = 1.1, F_{ysc} per dwg)
Dead Load (k) =	0.0	0.0	0.0	0.0	0.0	Gravity load on brace neglected
Live Load (k) =	0.0	0.0	0.0	0.0	0.0	Gravity load on brace neglected
Seismic Load (k) =	272.4	239.3	226.6	189.8	123.3	Primary direction (from ETABS analysis)
	86.9	68.5	61.7	55.3	37.0	Perpendicular direction (from ETABS analysis)
Combined Axial Load, P_u (k) =	298	260	245	206	134	(1.2+0.2S_{DS})D + 0.5 L + ρ E (include 100%+30% effects)
Steel Core Area (sq.in.) =	26.0	15.0	12.0	9.0	6.0	
ϕ P_{ysc} (k) =	796	459	367	275	184	$\phi F_{ysc}A_{sc}$ (AISC 341-05 Equation 16-1)
DCR =	0.38	0.57	0.67	0.75	0.73	$P_u/\phi P_{ysc}$
	Brace OK	*Brace OK*	*Brace OK*	*Brace OK*	*Brace OK*	

AISC 341-05 Section 16.2d - Adjusted Brace Strength

ω=	1.25	1.25	1.25	1.25	1.25	Strain hardening adjustment factor (assumed)
β=	1.35	1.35	1.35	1.35	1.35	Comp.strength adjustment factor (assumed)
$\beta\omega$=	1.69	1.69	1.69	1.69	1.69	
$\omega F_{ymax}A_{sc}$=	1466	846	677	507	338	Adjusted Brace Strength in Tension
$\beta\omega F_{ymax}A_{sc}$=	1979	1142	913	685	457	Adjusted Brace Strength in Compression

BEAM DESIGN

Beam Demands

	B94	B94	B94	B94	B94	ETABS Beam ID
$P_{ubm,c}$ (k) =	1193	745	596	447	298	Max. comp. due to brace tens., $P_{ubm,c}$ = cos(ψ_b) $\omega F_{ymax}A_{sc,b}$
$P_{ubm,t}$ (k) =	1610	1006	805	604	403	Max. tens. due to brace comp., $P_{ubm,t}$ = cos(ψ_b) $\beta\omega F_{ymax}A_{sc,b}$
$M_{E,drift}$ (k-ft) =	0	0	0	0	0	Drift-induced ETABS seismic moment neglected
$M_{Em,br}$ (k-ft) =	0	0	0	0	0	Seis. moment due to adj. brace strength, 0 for single diag. config.
M_{ug-} (k-ft) =	110	147	151	149	90	Factored gravity moment (from ETABS Analysis)
M_u (k-ft) =	110	147	151	149	90	M_{ug-} + $M_{Em,br}$
V_{Emh} (k) =	0.0	0.0	0.0	0.0	0.0	Seis. shear due to adj. brace strength, 0 for single diag. config.
V_{ug} (k) =	22	15	15	15	15	Factored gravity shear (from ETABS Analysis)
V_u (k) =	22	15	15	15	15	V_{ug} + V_{Em}

Beam Geometric Properties

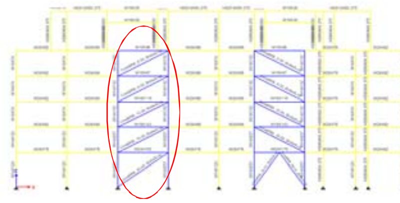
F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Beam Size =	W24x162	W18x143	W18x119	W18x97	W18x86
A_g (in^2) =	47.8	42.0	35.1	28.5	25.3
t_f(in) =	1.22	1.32	1.06	0.87	0.77
t_w (in) =	0.71	0.73	0.66	0.54	0.48
d (in) =	25.0	19.5	19.0	18.6	18.4
b_f (in) =	13.0	11.2	11.3	11.1	11.1
Z_x (in^3) =	468	322	262	211	186
r_x(in) =	10.40	8.09	7.90	7.82	7.77
r_y(in) =	3.05	2.72	2.69	2.65	2.63
r_{ts}(in) =	3.57	3.17	3.13	3.08	3.05
h_o(in) =	23.80	18.20	17.90	17.70	17.60
J (in^4) =	18.50	19.20	10.60	5.86	4.10

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

SHEET NO.
PROJECT NO. 197042.00-UCSF
DATE 03/17/2020
BY LZhou
CHECKED

CLIENT UCSF
SUBJECT Byers Hall: BRBF Design

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL G/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BEAM DESIGN (CONT'D)

Seismic Compactness Per AISC 341-05 Section16.5a/8.2b

Beam Compact Flange $b_f/2t_f$ =	5.3	4.2	5.3	6.4	7.2	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.2	7.2	7.2	7.2	7.2	
$b_f/2t_f \le (b/2t)_{max}$=	*Beam OK*	*Beam OK*	*Beam OK*	*Beam OK*	*Beam OK*	
Beam Compact Web $(d-2t_f)/t_w$ =	32.0	23.1	25.8	31.5	35.1	
$C_a = P_u/\phi P_y$ =	0.55	0.39	0.38	0.35	0.26	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	28.6	37.4	38.3	39.9	44.6	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	44.1	47.0	47.3	47.9	49.5	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	44.1	47.0	47.3	47.9	49.5	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	*Beam OK*	*Beam OK*	*Beam OK*	*Beam OK*	*Beam OK*	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	2151	1890	1580	1283	1139	AISC 360-05 Equation D2-1
DCR =	0.75	0.53	0.51	0.47	0.35	
	Beam OK	*Beam OK*	*Beam OK*	*Beam OK*	*Beam OK*	

AISC 360-05 Section E3 - Compression

L_x(ft) =	19.42	19.42	19.42	19.42	19.42	Strong axis unbraced length
L_y(ft) =	19.42	19.42	19.42	19.42	19.42	Weak axis unbraced length
k_x =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_x$ =	22.4	28.8	29.5	29.8	30.0	
k_y =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_y$ =	76.4	85.7	86.6	87.9	88.6	
F_e (ksi) =	49.04	39.01	38.15	37.02	36.47	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	32.6	29.2	28.9	28.4	28.2	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	1404	1105	913	729	641	AISC 360-05 Equation E3-1
DCR =	0.85	0.67	0.65	0.61	0.46	
	Beam OK	*Beam OK*	*Beam OK*	*Beam OK*	*Beam OK*	

AISC 360-05 Section F2 - Flexure

L_p(ft) =	10.8	9.6	9.5	9.4	9.3	AISC 360-05 Equation F2-5
L_r (ft) =	35.8	39.6	34.3	30.4	28.6	AISC 360-05 Equation F2-6
C_b =	1.00	1.00	1.00	1.00	1.00	
S_x(in^3) =	414.0	282.0	231.0	188.0	166.0	
M_p (k-ft) =	1950	1342	1092	879	775	Z_xF_y
M_n (k-ft) =	1693	1172	924	721	622	AISC 360-05 Equation F2-2
$\phi_b M_n$ (k-ft) =	1524	1055	832	649	560	
DCR =	0.07	0.14	0.18	0.23	0.16	
	Beam OK	*Beam OK*	*Beam OK*	*Beam OK*	*Beam OK*	

AISC 360-05 Section H1 - Combined Compression & Flexure

P_u (k) =	1193	745	596	447	298	
M_u (k-ft) =	110	147	151	149	90	
$P_u/\phi_c P_{nc}$ =	0.85	0.67	0.65	0.61	0.46	
combined equation =	0.91	0.80	0.81	0.82	0.61	AISC 360-05 Equation H1-1a or H1-1b
	Beam OK	*Beam OK*	*Beam OK*	*Beam OK*	*Beam OK*	

AISC 360-05 Section H2 - Combined Tension & Flexure

P_u (k) =	1610	1006	805	604	403	
M_u (k-ft) =	110	147	151	149	90	
$P_u/\phi_c P_{nc}$ =	0.75	0.53	0.51	0.47	0.35	
combined equation =	0.81	0.66	0.67	0.68	0.50	AISC 360-05 Equation H1-1a or H1-1b
	Beam OK	*Beam OK*	*Beam OK*	*Beam OK*	*Beam OK*	

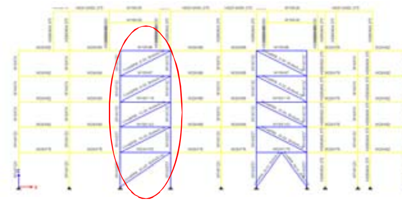
AISC 360-05 Section G2 - Shear

$\phi_v V_n$ (k) =	429	332	299	244	219	AISC 360-05 Equation G2-1
DCR =	0.05	0.05	0.05	0.06	0.07	
	Beam OK	*Beam OK*	*Beam OK*	*Beam OK*	*Beam OK*	

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

CLIENT	UCSF	SHEET NO.	
SUBJECT	Byers Hall: BRBF Design	PROJECT NO.	197042.00-UCSF
		DATE	03/17/2020
		BY	LZhou
		CHECKED	

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL G/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (RIGHT)

Column Demands

	C26	C26	C26	C26	C26	ETABS Column ID
P_{DL} (k) =	317	254	190	129	63	(from ETABS gravity analysis)
P_{LL} (k) =	208	165	123	81	36	(from ETABS gravity analysis)
1.2DL + f_1LL + E_v=	542	433	324	218	105	$E_V = (0.2)(S_{DS})(DL)$
0.9DL - E_v=	228	183	137	93	46	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	

Brace in Tension - Beam in Compression - Column in Compression

$V_{t,br}$ (k) =	852	399	319	240	160	Vert.component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perpendicular frames
$\Sigma P_{em} + 0.3{*}\Sigma P_{em,perp}$ (k) =	1970	1118	719	399	160	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	2512	1551	1042	617	265	

Brace in Compression - Beam in Tension - Column in Tension

$V_{c,br}$ (k) =	1150	539	431	323	216	Vertical component of the adj. brace force in compression
$V_{c,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perpendicular frames
$\Sigma P_{em} + 0.3{*}\Sigma P_{em,perp}$ (k) =	2660	1509	970	539	216	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	2431	1326	834	446	170	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x257	W14x257	W14x132	W14x132	W14x132
A_g (in^2) =	75.6	75.6	38.8	38.8	38.8
t_f (in) =	1.89	1.89	1.03	1.03	1.03
t_w (in) =	1.18	1.18	0.65	0.65	0.65
d (in) =	16.4	16.4	14.7	14.7	14.7
b_f (in) =	16	16	14.7	14.7	14.7
Z_x (in^3) =	487	487	234	234	234
Z_y (in^3) =	246	246	113	113	113
r_x (in) =	6.71	6.71	6.28	6.28	6.28
r_y (in) =	4.13	4.13	3.76	3.76	3.76
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	32.0	23.9	25.6	25.7	25.7
$(kL/r)_y$ =	52.1	38.9	42.8	42.9	43.0

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column Compact Flange $b_f/2t_f$ =	4.23	4.23	7.14	7.14	7.14	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	
Column Compact Web $(d-2t_f)/t_w$ =	10.7	10.7	19.6	19.6	19.6	
$C_a = P_u/\phi P_y$ =	0.74	0.46	0.60	0.35	0.15	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	18.5	34.0	26.2	39.6	50.7	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	40.6	45.9	43.3	47.8	51.5	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	40.6	45.9	43.3	47.8	51.5	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

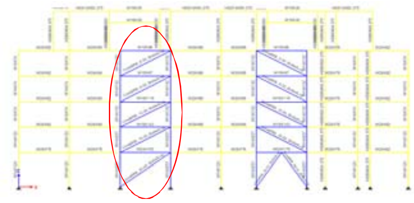
AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	3402.0	3402.0	1746.0	1746.0	1746.0	AISC 360-05 Equation D2-1
DCR =	0.71	0.39	0.48	0.26	0.10	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	105.61	189.52	156.11	155.33	154.95	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.0	44.8	43.7	43.7	43.7	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2790	3046	1527	1526	1525	AISC 360-05 Equation E3-1
DCR =	0.90	0.51	0.68	0.40	0.17	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL G/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (LEFT)

Column Demands

	C23	C23	C23	C23	C23	ETABS Column ID
P_{DL} (k) =	354	291	218	152	87	
P_{LL} (k) =	217	177	132	90	50	
1.2DL + f_1LL + E_v =	597	489	367	254	144	$E_V = (0.2)(S_{DS})(DL)$
0.9DL - E_v =	255	209	157	109	62	
Column Orientation =	Weak	Weak	Weak	Weak	Weak	

Brace in Compression - Beam in Tension - Column in Compression

$V_{c,br}$ (k) =	539	431	323	216	0	Vertical component of the adj. brace force in comp.
$V_{c,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perpendicular frames
$\Sigma P_{em} + 0.3^*\Sigma P_{em,perp}$ (k) =	1509	970	539	216	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	2106	1460	907	470	144	

Brace in Tension - Beam in Compression - Column in Tension

$V_{t,br}$ (k) =	399	319	240	160	0	Vertical component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perpendicular frames
$\Sigma P_{em} + 0.3^*\Sigma P_{em,perp}$ (k) =	1118	719	399	160	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	863	510	242	51	-62	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x257	W14x257	W14x132	W14x132	W14x132
A_g (in^2) =	75.6	75.6	38.8	38.8	38.8
t_f (in) =	1.89	1.89	1.03	1.03	1.03
t_w (in) =	1.18	1.18	0.65	0.65	0.65
d (in) =	16.4	16.4	14.7	14.7	14.7
b_f (in) =	16	16	14.7	14.7	14.7
Z_x (in^3) =	487	487	234	234	234
Z_y (in^3) =	246	246	113	113	113
r_x (in) =	6.71	6.71	6.28	6.28	6.28
r_y (in) =	4.13	4.13	3.76	3.76	3.76
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	32.0	23.9	25.6	25.7	25.7
$(kL/r)_y$ =	52.1	38.9	42.8	42.9	43.0

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column shall satisfy highly ductile requirements

Column Compact Flange $b_f/2t_f$ =	4.23	4.23	7.14	7.14	7.14	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	
Column Compact Web $(d-2t_f)/t_w$ =	10.7	10.7	19.6	19.6	19.6	
$C_a = P_u/\phi P_y$ =	0.62	0.43	0.52	0.27	0.08	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	25.0	35.5	30.5	44.2	54.5	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	42.9	46.4	44.7	49.3	52.8	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	42.9	46.4	44.7	49.3	54.5	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	3402.0	3402.0	1746.0	1746.0	1746.0	AISC 360-05 Equation D2-1
DCR =	0.25	0.15	0.14	0.03	No Tension	
	Column OK	Column OK	Column OK	Column OK	Column OK	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	105.61	189.52	156.11	155.33	154.95	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.0	44.8	43.7	43.7	43.7	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2790	3046	1527	1526	1525	AISC 360-05 Equation E3-1
DCR =	0.75	0.48	0.59	0.31	0.09	
	Column OK	Column OK	Column OK	Column OK	Column OK	

ASCE 7-05/AISC 341-05
LFRS Seismic Provisions

Detailed Frame Checks

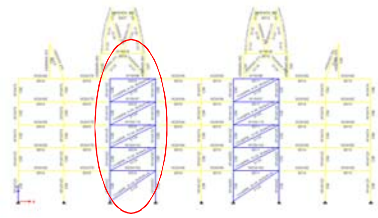
GL D/3-4 Diagonal

SIMPSON GUMPERTZ & HEGER

Engineering of Structures and Building Enclosures

CLIENT	UCSF	SHEET NO.	
SUBJECT	Byers Hall: BRBF Design	PROJECT NO.	197042.00-UCSF
		DATE	03/17/2020
		BY	LZhou
		CHECKED	

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL D/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BRBF GEOMETRY:	Level 2	Level 3	Level 4	Level 5	Roof	
L(ft) =	28.00	28.00	28.00	28.00	28.00	Bay Width (Column C-C)
h_i (ft) =	20.00	15.00	15.00	15.00	15.00	Story Height
L_{diag} (ft)=	34.4	31.8	31.8	31.8	31.8	Work Point - Work Point
cos ψ =	0.814	0.881	0.881	0.881	0.881	ψ = angle between brace and horizontal axis
sin ψ =	0.581	0.472	0.472	0.472	0.472	

BRACE DESIGN

AISC 341-05 Section 16.2a - Brace Strength

	D126	D126	D126	D126	D126	ETABS Brace ID
F_{ysc} =	34 ksi	34 ksi	34 ksi	34 ksi	34 ksi	Minimum yield stress of the steel core
F_{ymax} =	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	Max yield stress of the steel core (R_yF_{ysc}; R_y = 1.1, F_{ysc} per dwg)
Dead Load (k) =	0.0	0.0	0.0	0.0	0.0	Gravity load on brace neglected
Live Load (k) =	0.0	0.0	0.0	0.0	0.0	Gravity load on brace neglected
Seismic Load (k) =	269.4	215.2	194.3	156.9	95.5	Primary direction (from ETABS analysis)
	46.2	14.5	13.5	11.2	8.3	Perpendicular direction (from ETABS analysis)
Combined Axial Load, P_u (k) =	283	220	198	160	98	$(1.2+0.2S_{DS})D + 0.5\,L + \rho\,E$ (include 100%+30% effects)
Steel Core Area (sq.in.) =	26.0	15.0	12.0	9.0	6.0	
ϕ P_{ysc} (k) =	796	459	367	275	184	$\phi F_{ysc}A_{sc}$ (AISC 341-05 Equation 16-1)
DCR =	0.36	0.48	0.54	0.58	0.53	$P_u/\phi P_{ysc}$
	Brace OK	*Brace OK*	*Brace OK*	*Brace OK*	*Brace OK*	

AISC 341-05 Section 16.2d - Adjusted Brace Strength

ω=	1.25	1.25	1.25	1.25	1.25	Strain hardening adjustment factor (assumed)
β=	1.35	1.35	1.35	1.35	1.35	Comp.strength adjustment factor (assumed)
$\beta\omega$=	1.69	1.69	1.69	1.69	1.69	
$\omega F_{ymax}A_{sc}$=	1466	846	677	507	338	Adjusted Brace Strength in Tension
$\beta\omega F_{ymax}A_{sc}$=	1979	1142	913	685	457	Adjusted Brace Strength in Compression

BEAM DESIGN

Beam Demands

	B508	B508	B508	B508	B508	ETABS Beam ID
$P_{ubm,c}$ (k) =	1193	745	596	447	298	Max. comp. due to brace tens., $P_{ubm,c}$ = $\cos(\psi_b)\ \omega F_{ymax}A_{sc,b}$
$P_{ubm,t}$ (k) =	1610	1006	805	604	403	Max. tens. due to brace comp., $P_{ubm,t}$ = $\cos(\psi_b)\ \beta\omega F_{ymax}A_{sc,b}$
$M_{E,drift}$ (k-ft) =	0	0	0	0	0	Drift-induced ETABS seismic moment neglected
$M_{Em,br}$ (k-ft) =	0	0	0	0	0	Seis. moment due to adj. brace strength, 0 for single diag. config.
M_{ug-} (k-ft) =	48	58	49	59	66	Factored gravity moment (from ETABS Analysis)
M_u (k-ft) =	48	58	49	59	66	M_{ug-} + $M_{Em,br}$
V_{Emh} (k) =	0.0	0.0	0.0	0.0	0.0	Seis. shear due to adj. brace strength, 0 for single diag. config.
V_{ug} (k) =	14	15	15	15	15	Factored gravity shear (from ETABS Analysis)
V_u (k) =	14	15	15	15	15	V_{ug} + V_{Em}

Beam Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Beam Size =	W24x162	W18x143	W18x119	W18x97	W18x86
A_g (in^2) =	47.8	42.0	35.1	28.5	25.3
t_f (in) =	1.22	1.32	1.06	0.87	0.77
t_w (in) =	0.71	0.73	0.66	0.54	0.48
d (in) =	25.0	19.5	19.0	18.6	18.4
b_f (in) =	13.0	11.2	11.3	11.1	11.1
Z_x (in^3) =	468	322	262	211	186
r_x(in) =	10.40	8.09	7.90	7.82	7.77
r_y(in) =	3.05	2.72	2.69	2.65	2.63
r_{ts}(in) =	3.57	3.17	3.13	3.08	3.05
h_o(in) =	23.80	18.20	17.90	17.70	17.60
J (in^4) =	18.50	19.20	10.60	5.86	4.10

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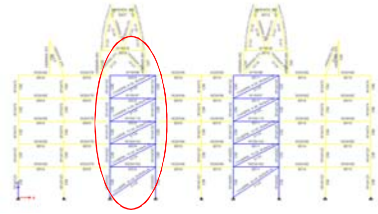
CLIENT	UCSF	SHEET NO.	L
SUBJECT	Byers Hall: BRBF Design	PROJECT NO.	197042.00-UCSF
		DATE	03/17/2020
		BY	LZhou
		CHECKED	

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL D/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BEAM DESIGN (CONT'D)

Seismic Compactness Per AISC 341-05 Section16.5a/8.2b

Beam Compact Flange $b_f/2t_f$ =	5.3	4.2	5.3	6.4	7.2	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.2	7.2	7.2	7.2	7.2	
$b_f/2t_f \le (b/2t)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	
Beam Compact Web $(d-2t_f)/t_w$ =	32.0	23.1	25.8	31.5	35.1	
$C_a = P_u/\phi P_y$ =	0.55	0.39	0.38	0.35	0.26	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	28.6	37.4	38.3	39.9	44.6	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	44.1	47.0	47.3	47.9	49.5	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	44.1	47.0	47.3	47.9	49.5	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	2151	1890	1580	1283	1139	AISC 360-05 Equation D2-1
DCR =	0.75	0.53	0.51	0.47	0.35	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section E3 - Compression

L_x(ft) =	14.00	14.00	14.00	14.00	14.00	Strong axis unbraced length
L_y(ft) =	14.00	14.00	14.00	14.00	14.00	Weak axis unbraced length
k_x =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_x$ =	16.2	20.8	21.3	21.5	21.6	
k_y =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_y$ =	55.1	61.8	62.5	63.4	63.9	
F_e (ksi) =	94.34	75.03	73.38	71.21	70.14	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	40.1	37.8	37.6	37.3	37.1	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	1723	1430	1188	956	845	AISC 360-05 Equation E3-1
DCR =	0.69	0.52	0.50	0.47	0.35	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section F2 - Flexure

L_p(ft) =	10.8	9.6	9.5	9.4	9.3	AISC 360-05 Equation F2-5
L_r (ft) =	35.8	39.6	34.3	30.4	28.6	AISC 360-05 Equation F2-6
C_b =	1.00	1.00	1.00	1.00	1.00	
S_x(in^3) =	414.0	282.0	231.0	188.0	166.0	
M_p (k-ft) =	1950	1342	1092	879	775	Z_xF_y
M_n (k-ft) =	1854	1266	1016	806	704	AISC 360-05 Equation F2-2
$\phi_b M_n$ (k-ft) =	1669	1139	914	725	634	
DCR =	0.03	0.05	0.05	0.08	0.10	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section H1 - Combined Compression & Flexure

P_u (k) =	1193	745	596	447	298	
M_u (k-ft) =	48	58	49	59	66	
$P_u/\phi_c P_{nc}$ =	0.69	0.52	0.50	0.47	0.35	
combined equation =	0.72	0.57	0.55	0.54	0.45	AISC 360-05 Equation H1-1a or H1-1b
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section H2 - Combined Tension & Flexure

P_u (k) =	1610	1006	805	604	403	
M_u (k-ft) =	48	58	49	59	66	
$P_u/\phi_c P_{nc}$ =	0.75	0.53	0.51	0.47	0.35	
combined equation =	0.77	0.58	0.56	0.54	0.45	AISC 360-05 Equation H1-1a or H1-1b
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section G2 - Shear

$\phi_v V_n$ (k) =	429	332	299	244	219	AISC 360-05 Equation G2-1
DCR =	0.03	0.05	0.05	0.06	0.07	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

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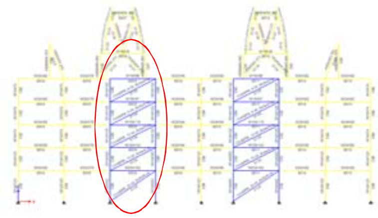
Engineering of Structures and Building Enclosures

CLIENT	UCSF	SHEET NO.	L
SUBJECT	Byers Hall: BRBF Design	PROJECT NO.	197042.00-UCSF
		DATE	03/17/2020
		BY	LZhou
		CHECKED	

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL D/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (RIGHT)

Column Demands

	C88	C88	C88	C88	C88	ETABS Column ID
P_{DL} (k) =	379	318	228	168	111	(from ETABS gravity analysis)
P_{LL} (k) =	287	245	175	129	85	(from ETABS gravity analysis)
$1.2DL + f_1LL + E_v$ =	666	562	401	297	195	$E_V = (0.2)(S_{DS})(DL)$
$0.9DL - E_v$ =	273	229	164	121	80	
Column Orientation =	Weak	Weak	Weak	Weak	Weak	

Brace in Tension - Beam in Compression - Column in Compression

$V_{t,br}$ (k) =	852	399	319	240	160	Vert.component of the adj. brace force in tension
$V_{c,br,perp}$ (k) =	664	531	398	265	0	Vert. component of the adj. brace force from perpendicular frames
$\Sigma P_{em} + 0.3{*}\Sigma P_{em,perp}$ (k) =	2527	1476	918	479	160	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	3193	2038	1319	776	355	

Brace in Compression - Beam in Tension - Column in Tension

$V_{c,br}$ (k) =	1150	539	431	323	216	Vertical component of the adj. brace force in compression
$V_{t,br,perp}$ (k) =	492	393	295	197	0	Vert. component of the adj. brace force from perpendicular frames
$\Sigma P_{em} + 0.3{*}\Sigma P_{em,perp}$ (k) =	3072	1775	1118	598	216	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	2800	1546	954	477	136	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x283	W14x283	W14x132	W14x132	W14x132
A_g (in^2) =	83.3	83.3	38.8	38.8	38.8
t_f (in) =	2.07	2.07	1.03	1.03	1.03
t_w (in) =	1.29	1.29	0.65	0.65	0.65
d (in) =	16.7	16.7	14.7	14.7	14.7
b_f (in) =	16.1	16.1	14.7	14.7	14.7
Z_x (in^3) =	542	542	234	234	234
Z_y (in^3) =	274	274	113	113	113
r_x (in) =	6.79	6.79	6.28	6.28	6.28
r_y (in) =	4.17	4.17	3.76	3.76	3.76
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	31.7	23.6	25.6	25.7	25.7
$(kL/r)_y$ =	51.6	38.5	42.8	42.9	43.0

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column Compact Flange $b_f/2t_f$ =	3.89	3.89	7.14	7.14	7.14	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	
Column Compact Web $(d-2t_f)/t_w$ =	9.7	9.7	19.6	19.6	19.6	
$C_a = P_u/\phi P_y$ =	0.85	0.54	0.76	0.44	0.20	
$2.45\ (E/F_y)^{0.5}\ (1 - 0.93C_a)$ =	12.3	29.2	17.5	34.6	47.8	if $C_a \le 0.125$
$0.77\ (E/F_y)^{0.5}\ (2.93 - C_a)$ =	38.5	44.3	40.3	46.1	50.6	if $C_a > 0.125$
$1.49\ (E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	38.5	44.3	40.3	46.1	50.6	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

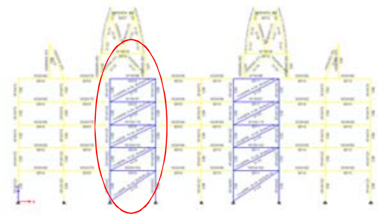
AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	3748.5	3748.5	1746.0	1746.0	1746.0	AISC 360-05 Equation D2-1
DCR =	0.75	0.41	0.55	0.27	0.08	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	107.67	193.21	156.11	155.33	154.95	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.2	44.9	43.7	43.7	43.7	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	3086	3364	1527	1526	1525	AISC 360-05 Equation E3-1
DCR =	1.03	0.61	0.86	0.51	0.23	
	Revise	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL D/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (LEFT)

Column Demands						
	C86	C86	C86	C86	C86	ETABS Column ID
P_{DL} (k) =	420	337	238	163	94	
P_{LL} (k) =	317	257	182	124	69	
$1.2DL + f_1LL + E_v$ =	738	594	420	287	164	$E_V = (0.2)(S_{DS})(DL)$
$0.9DL - E_v$ =	303	243	172	117	67	
Column Orientation =	Weak	Weak	Weak	Weak	Weak	
Brace in Compression - Beam in Tension - Column in Compression						
$V_{c,br}$ (k) =	539	431	323	216	0	Vertical component of the adj. brace force in comp.
$V_{c,br,perp}$ (k) =	998	998	623	374	0	Vert. component of the adj. brace force from perpendicular frames
$\Sigma P_{em} + 0.3{*}\Sigma P_{em,perp}$ (k) =	2407	1569	838	328	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	3146	2163	1258	615	164	
Brace in Tension - Beam in Compression - Column in Tension						
$V_{t,br}$ (k) =	399	319	240	160	0	Vertical component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	739	739	462	277	0	Vert. component of the adj. brace force from perpendicular frames
$\Sigma P_{em} + 0.3{*}\Sigma P_{em,perp}$ (k) =	1783	1162	621	243	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	1481	919	449	125	-67	
Column Geometric Properties						
F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi	
Column Size =	W14x342	W14x342	W14x145	W14x145	W14x145	
A_g (in^2) =	101.0	101.0	42.7	42.7	42.7	
t_f (in) =	2.47	2.47	1.09	1.09	1.09	
t_w (in) =	1.54	1.54	0.68	0.68	0.68	
d (in) =	17.5	17.5	14.8	14.8	14.8	
b_f (in) =	16.4	16.4	15.5	15.5	15.5	
Z_x (in^3) =	672	672	260	260	260	
Z_y (in^3) =	338	338	133	133	133	
r_x (in) =	6.98	6.98	6.33	6.33	6.33	
r_y (in) =	4.24	4.24	3.98	3.98	3.98	
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5	
k_x =	1.00	1.00	1.00	1.00	1.00	
k_y =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_x$ =	30.8	23.0	25.4	25.5	25.5	
$(kL/r)_y$ =	50.7	37.9	40.5	40.6	40.6	
Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b						Column shall satisfy highly ductile requirements
Column Compact Flange $b_f/2t_f$ =	3.32	3.32	7.11	7.11	7.11	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	
Column Compact Web $(d-2t_f)/t_w$ =	8.2	8.2	18.6	18.6	18.6	
$C_a = P_u/\phi P_y$ =	0.69	0.48	0.65	0.32	0.09	
$2.45\ (E/F_y)^{0.5}\ (1 - 0.93C_a)$ =	21.0	32.9	23.1	41.4	54.3	if $C_a \le 0.125$
$0.77\ (E/F_y)^{0.5}\ (2.93 - C_a)$ =	41.5	45.5	42.2	48.4	52.8	if $C_a > 0.125$
$1.49\ (E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	41.5	45.5	42.2	48.4	54.3	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	
AISC 360-05 Section D2 - Tension						
$\phi_t P_{nt}$ (k) =	4545.0	4545.0	1921.5	1921.5	1921.5	AISC 360-05 Equation D2-1
DCR =	0.33	0.20	0.23	0.07	No Tension	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	
AISC 360-05 Section E3 - Compression						
F_e (ksi) =	111.31	199.75	174.91	174.04	173.61	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.4	45.0	44.4	44.3	44.3	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	3766	4093	1705	1704	1703	AISC 360-05 Equation E3-1
DCR =	0.84	0.53	0.74	0.36	0.10	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

ASCE 7-05/AISC 341-05
LFRS Seismic Provisions

Detailed Frame Checks

GL 4/D-E Diagonal

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

SHEET NO.	
PROJECT NO.	197042.00-UCSF
DATE	03/17/2020
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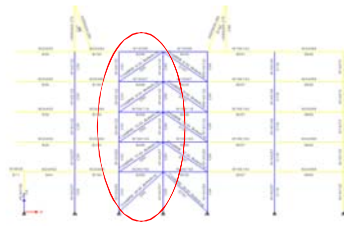
CLIENT UCSF
SUBJECT Byers Hall: BRBF Design

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL 4/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BRBF GEOMETRY:

	Level 2	Level 3	Level 4	Level 5	Roof	
L(ft) =	21.00	21.00	21.00	21.00	21.00	Bay Width (Column C-C)
h_i (ft) =	20.00	15.00	15.00	15.00	15.00	Story Height
L_{diag} (ft)=	29.0	25.8	25.8	25.8	25.8	Work Point - Work Point
cos ψ =	0.724	0.814	0.814	0.814	0.814	ψ = angle between brace and horizontal axis
sin ψ =	0.690	0.581	0.581	0.581	0.581	

BRACE DESIGN

AISC 341-05 Section 16.2a - Brace Strength

	D50	D50	D50	D50	D50	ETABS Brace ID
F_{ysc} =	34 ksi	34 ksi	34 ksi	34 ksi	34 ksi	Minimum yield stress of the steel core
F_{ymax} =	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	Max yield stress of the steel core (R_yF_{ysc}; R_y = 1.1, F_{ysc} per dwg)
Dead Load (k) =	0.0	0.0	0.0	0.0	0.0	Gravity load on brace neglected
Live Load (k) =	0.0	0.0	0.0	0.0	0.0	Gravity load on brace neglected
Seismic Load (k) =	377.9	216.1	186.4	163.5	114.0	Primary direction (from ETABS analysis)
	133.1	73.2	61.8	50.4	31.1	Perpendicular direction (from ETABS analysis)
Combined Axial Load, P_u (k) =	418	238	205	179	123	$(1.2+0.2S_{DS})$D + 0.5 L + ρ E (include 100%+30% effects)
Steel Core Area (sq.in.) =	28.0	15.0	12.0	9.0	6.0	
ϕ P_{ysc} (k) =	857	459	367	275	184	$\phi F_{ysc}A_{sc}$ (AISC 341-05 Equation 16-1)
DCR =	0.49	0.52	0.56	0.65	0.67	$P_u/\phi P_{ysc}$
	Brace OK	*Brace OK*	*Brace OK*	*Brace OK*	*Brace OK*	

AISC 341-05 Section 16.2d - Adjusted Brace Strength

ω=	1.25	1.25	1.25	1.25	1.25	Strain hardening adjustment factor (assumed)
β=	1.35	1.35	1.35	1.35	1.35	Comp.strength adjustment factor (assumed)
$\beta\omega$=	1.69	1.69	1.69	1.69	1.69	
$\omega F_{ymax}A_{sc}$=	1579	846	677	507	338	Adjusted Brace Strength in Tension
$\beta\omega F_{ymax}A_{sc}$=	2131	1142	913	685	457	Adjusted Brace Strength in Compression

BEAM DESIGN

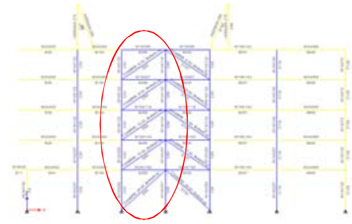
Beam Demands

	B268	B268	B268	B268	B268	ETABS Beam ID
$P_{ubm,c}$ (k) =	1143	688	550	413	275	Max. comp. due to brace tens., $P_{ubm,c}$ = cos(ψ_b) $\omega F_{ymax}A_{sc,b}$
$P_{ubm,t}$ (k) =	1543	929	743	557	372	Max. tens. due to brace comp., $P_{ubm,t}$ = cos(ψ_b) $\beta\omega F_{ymax}A_{sc,b}$
$M_{E,drift}$ (k-ft) =	0	0	0	0	0	Drift-induced ETABS seismic moment neglected
$M_{Em,br}$ (k-ft) =	0	0	0	0	0	Seis. moment due to adj. brace strength, 0 for single diag. config.
M_{ug-} (k-ft) =	165	163	167	162	159	Factored gravity moment (from ETABS Analysis)
M_u (k-ft) =	165	163	167	162	159	M_{ug-} + $M_{Em,br}$
V_{Emh} (k) =	0.0	0.0	0.0	0.0	0.0	Seis. shear due to adj. brace strength, 0 for single diag. config.
V_{ug} (k) =	41	15	15	15	15	Factored gravity shear (from ETABS Analysis)
V_u (k) =	41	15	15	15	15	V_{ug} + V_{Em}

Beam Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Beam Size =	W24x162	W18x143	W18x119	W18x97	W18x86
A_g (in^2) =	47.8	42.0	35.1	28.5	25.3
t_f (in) =	1.22	1.32	1.06	0.87	0.77
t_w (in) =	0.71	0.73	0.66	0.54	0.48
d (in) =	25.0	19.5	19.0	18.6	18.4
b_f (in) =	13.0	11.2	11.3	11.1	11.1
Z_x (in^3) =	468	322	262	211	186
r_x(in) =	10.40	8.09	7.90	7.82	7.77
r_y(in) =	3.05	2.72	2.69	2.65	2.63
r_{ts}(in) =	3.57	3.17	3.13	3.08	3.05
h_o(in) =	23.80	18.20	17.90	17.70	17.60
J (in^4) =	18.50	19.20	10.60	5.86	4.10

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL 4/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BEAM DESIGN (CONT'D)

Seismic Compactness Per AISC 341-05 Section16.5a/8.2b

Beam Compact Flange $b_f/2t_f$ =	5.3	4.2	5.3	6.4	7.2	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.2	7.2	7.2	7.2	7.2	
$b_f/2t_f \le (b/2t)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	
Beam Compact Web $(d-2t_f)/t_w$ =	32.0	23.1	25.8	31.5	35.1	
$C_a = P_u/\phi P_y$ =	0.53	0.36	0.35	0.32	0.24	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	29.8	39.0	39.9	41.3	45.7	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	44.5	47.6	47.9	48.4	49.9	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	44.5	47.6	47.9	48.4	49.9	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	2151	1890	1580	1283	1139	AISC 360-05 Equation D2-1
DCR =	0.72	0.49	0.47	0.43	0.33	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section E3 - Compression

L_x(ft) =	7.00	7.00	7.00	7.00	7.00	Strong axis unbraced length
L_y(ft) =	7.00	7.00	7.00	7.00	7.00	Weak axis unbraced length
k_x =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_x$ =	8.1	10.4	10.6	10.7	10.8	
k_y =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_y$ =	27.5	30.9	31.2	31.7	31.9	
F_e (ksi) =	377.35	300.11	293.52	284.86	280.58	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	47.3	46.6	46.6	46.5	46.4	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2035	1763	1471	1192	1057	AISC 360-05 Equation E3-1
DCR =	0.56	0.39	0.37	0.35	0.26	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section F2 - Flexure

L_p(ft) =	10.8	9.6	9.5	9.4	9.3	AISC 360-05 Equation F2-5
L_r (ft) =	35.8	39.6	34.3	30.4	28.6	AISC 360-05 Equation F2-6
C_b =	1.00	1.00	1.00	1.00	1.00	
S_x (in^3) =	414.0	282.0	231.0	188.0	166.0	
M_p (k-ft) =	1950	1342	1092	879	775	$Z_x F_y$
M_n (k-ft) =	1950	1342	1092	879	775	AISC 360-05 Equation F2-2
$\phi_b M_n$ (k-ft) =	1755	1208	983	791	698	
DCR =	0.09	0.13	0.17	0.20	0.23	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section H1 - Combined Compression & Flexure

P_u (k) =	1143	688	550	413	275	
M_u (k-ft) =	165	163	167	162	159	
$P_u/\phi_c P_{nc}$ =	0.56	0.39	0.37	0.35	0.26	
combined equation =	0.65	0.51	0.53	0.53	0.46	AISC 360-05 Equation H1-1a or H1-1b
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

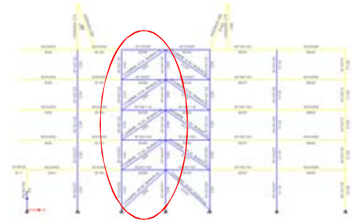
AISC 360-05 Section H2 - Combined Tension & Flexure

P_u (k) =	1543	929	743	557	372	
M_u (k-ft) =	165	163	167	162	159	
$P_u/\phi_c P_{nc}$ =	0.72	0.49	0.47	0.43	0.33	
combined equation =	0.80	0.61	0.62	0.62	0.53	AISC 360-05 Equation H1-1a or H1-1b
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section G2 - Shear

$\phi_v V_n$ (k) =	429	332	299	244	219	AISC 360-05 Equation G2-1
DCR =	0.10	0.05	0.05	0.06	0.07	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL 4/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (RIGHT)

Column Demands

	C69	C69	C69	C69	C69	ETABS Column ID
P_{DL} (k) =	240	251	194	144	78	(from ETABS gravity analysis)
P_{LL} (k) =	193	206	161	121	67	(from ETABS gravity analysis)
$1.2DL + f_1LL + E_v$ =	427	449	348	259	141	$E_V = (0.2)(S_{DS})(DL)$
$0.9DL - E_v$ =	173	181	140	104	56	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	

Brace in Tension - Beam in Compression - Column in Compression

$V_{t,br}$ (k) =	1089	492	393	295	197	Vert.component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perpendicular frames
$V_{c,br,adj}$ (k) =	-1470	-664	-531	-398	-265	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	-863	-482	-310	-172	-69	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	-436	-33	38	87	72	

Brace in Compression - Beam in Tension - Column in Tension

$V_{c,br}$ (k) =	1470	664	531	398	265	Vertical component of the adj. brace force in compression
$V_{c,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perpendicular frames
$V_{t,br,adj}$ (k) =	-1089	-492	-393	-295	-197	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	863	482	310	172	69	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	690	301	170	68	13	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x257	W14x257	W14x132	W14x132	W14x132
A_g (in^2) =	75.6	75.6	38.8	38.8	38.8
t_f (in) =	1.89	1.89	1.03	1.03	1.03
t_w (in) =	1.18	1.18	0.65	0.65	0.65
d (in) =	16.4	16.4	14.7	14.7	14.7
b_f (in) =	16	16	14.7	14.7	14.7
Z_x (in^3) =	487	487	234	234	234
Z_y (in^3) =	246	246	113	113	113
r_x (in) =	6.71	6.71	6.28	6.28	6.28
r_y (in) =	4.13	4.13	3.76	3.76	3.76
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	32.0	23.9	25.6	25.7	25.7
$(kL/r)_y$ =	52.1	38.9	42.8	42.9	43.0

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column Compact Flange $b_f/2t_f$ =	4.23	4.23	7.14	7.14	7.14	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	
Column Compact Web $(d-2t_f)/t_w$ =	10.7	10.7	19.6	19.6	19.6	
$C_a = P_u/\phi P_y$ =	-0.13	-0.01	0.02	0.05	0.04	
$2.45\,(E/F_y)^{0.5}\,(1 - 0.93C_a)$ =	66.0	59.5	57.8	56.3	56.7	if $C_a \le 0.125$
$0.77\,(E/F_y)^{0.5}\,(2.93 - C_a)$ =	56.7	54.5	53.9	53.4	53.6	if $C_a > 0.125$
$1.49\,(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	66.0	59.5	57.8	56.3	56.7	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	3402.0	3402.0	1746.0	1746.0	1746.0	AISC 360-05 Equation D2-1
DCR =	0.20	0.09	0.10	0.04	0.01	
	Column OK	Column OK	Column OK	Column OK	Column OK	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	105.61	189.52	156.11	155.33	154.95	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.0	44.8	43.7	43.7	43.7	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2790	3046	1527	1526	1525	AISC 360-05 Equation E3-1
DCR =	No Compress.	No Compress.	0.03	0.06	0.05	
	Column OK	Column OK	Column OK	Column OK	Column OK	

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

SHEET NO.	
PROJECT NO.	197042.00-UCSF
DATE	03/17/2020
BY	LZhou
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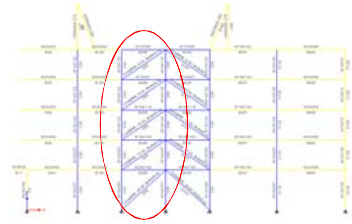
CLIENT UCSF
SUBJECT Byers Hall: BRBF Design

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL 4/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (LEFT)

Column Demands

	C50	C50	C50	C50	C50	ETABS Column ID
P_{DL} (k) =	394	311	226	152	83	
P_{LL} (k) =	333	267	197	135	75	
1.2DL + f_1LL + E_v=	710	563	411	277	152	E_V = (0.2)(S_{DS})(DL)
0.9DL - E_v=	283	224	163	109	59	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	

Brace in Compression - Beam in Tension - Column in Compression

$V_{c,br}$ (k) =	664	531	398	265	0	Vertical component of the adj. brace force in comp.
$V_{c,br,perp}$ (k) =	852	399	319	240	160	Vert. component of the adj. brace force from perpendicular frames
$V_{c,br,adj}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from adjacent frames
ΣP_{em} + $\Sigma P_{em,adj}$ + 0.3*$\Sigma P_{em,perp}$ (k) =	2449	1530	879	385	48	Sum of axial forces in column due to adj. brace force at all levels
P_{uc} = ΣP_{em} + $P_{u,grav}$ (k) =	3159	2093	1290	662	200	

Brace in Tension - Beam in Compression - Column in Tension

$V_{t,br}$ (k) =	492	393	295	197	0	Vertical component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	1150	539	431	323	216	Vert. component of the adj. brace force from perpendicular frames
$V_{t,br,adj}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from adjacent frames
ΣP_{em} + $\Sigma P_{em,adj}$ + 0.3*$\Sigma P_{em,perp}$ (k) =	2174	1338	783	358	65	Sum of axial forces in column due to adj. brace force at all levels
P_{ut} = ΣP_{em} - $P_{u,grav}$ (k) =	1891	1114	620	249	5	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x283	W14x283	W14x132	W14x132	W14x132
A_g (in^2) =	83.3	83.3	38.8	38.8	38.8
t_f (in) =	2.07	2.07	1.03	1.03	1.03
t_w (in) =	1.29	1.29	0.65	0.65	0.65
d (in) =	16.7	16.7	14.7	14.7	14.7
b_f (in) =	16.1	16.1	14.7	14.7	14.7
Z_x (in^3) =	542	542	234	234	234
Z_y (in^3) =	274	274	113	113	113
r_x (in) =	6.79	6.79	6.28	6.28	6.28
r_y (in) =	4.17	4.17	3.76	3.76	3.76
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	31.7	23.6	25.6	25.7	25.7
$(kL/r)_y$ =	51.6	38.5	42.8	42.9	43.0

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column shall satisfy highly ductile requirements

Column Compact Flange $b_f/2t_f$ =	3.89	3.89	7.14	7.14	7.14	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	
Column Compact Web $(d-2t_f)/t_w$ =	9.7	9.7	19.6	19.6	19.6	
$C_a = P_u/\phi P_y$ =	0.84	0.56	0.74	0.38	0.11	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	12.8	28.4	18.5	38.2	52.7	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	38.7	44.0	40.6	47.3	52.2	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	38.7	44.0	40.6	47.3	52.7	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	3748.5	3748.5	1746.0	1746.0	1746.0	AISC 360-05 Equation D2-1
DCR =	0.50	0.30	0.36	0.14	0.00	
	Column OK	Column OK	Column OK	Column OK	Column OK	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	107.67	193.21	156.11	155.33	154.95	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.2	44.9	43.7	43.7	43.7	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	3086	3364	1527	1526	1525	AISC 360-05 Equation E3-1
DCR =	1.02	0.62	0.84	0.43	0.13	
	Revise	Column OK	Column OK	Column OK	Column OK	

ASCE 7-05/AISC 341-05
LFRS Seismic Provisions

Detailed Frame Checks

GL B/3-4 Diagonal

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

CLIENT	UCSF
SUBJECT	Byers Hall: BRBF Design
SHEET NO.	
PROJECT NO.	197042.00-UCSF
DATE	03/17/2020
BY	LZhou
CHECKED	

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL

BRBF LOCATION : GL B/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BRBF GEOMETRY:

	Level 2	Level 3	Level 4	Level 5	Roof	
L(ft) =	28.00	28.00	28.00	28.00	28.00	Bay Width (Column C-C)
h_i (ft) =	20.00	15.00	15.00	15.00	15.00	Story Height
L_{diag} (ft)=	34.4	31.8	31.8	31.8	31.8	Work Point - Work Point
cos ψ =	0.814	0.881	0.881	0.881	0.881	ψ = angle between brace and horizontal axis
sin ψ =	0.581	0.472	0.472	0.472	0.472	

BRACE DESIGN

AISC 341-05 Section 16.2a - Brace Strength

	D150	D150	D150	D150	D150	ETABS Brace ID
F_{ysc} =	34 ksi	34 ksi	34 ksi	34 ksi	34 ksi	Minimum yield stress of the steel core
F_{ymax} =	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	Maximum yield stress of the steel core (R_yF_{ysc}; R_y = 1.1, F_{ysc} per dwg)
Dead Load (k) =	0.0	0.0	0.0	0.0	0.0	Gravity load on brace neglected
Live Load (k) =	0.0	0.0	0.0	0.0	0.0	Gravity load on brace neglected
Seismic Load (k) =	459.0	326.7	301.2	235.7	141.7	Primary direction (from ETABS analysis)
	84.5	53.6	47.8	40.6	26.2	Perpendicular direction (from ETABS analysis)
Combined Axial Load, P_u (k) =	484	343	315	248	150	$(1.2+0.2S_{DS})$D + 0.5 L + ρ E (include 100%+30% effects)
Steel Core Area (sq.in.) =	26.0	15.0	12.0	9.0	6.0	
ϕ P_{ysc} (k) =	796	459	367	275	184	$\phi F_{ysc}A_{sc}$ (AISC 341-05 Equation 16-1)
DCR =	0.61	0.75	0.86	0.90	0.81	$P_u/\phi P_{ysc}$
	Brace OK	*Brace OK*	*Brace OK*	*Brace OK*	*Brace OK*	

AISC 341-05 Section 16.2d - Adjusted Brace Strength

ω=	1.25	1.25	1.25	1.25	1.25	Strain hardening adjustment factor (assumed)
β=	1.35	1.35	1.35	1.35	1.35	Comp.strength adjustment factor (assumed)
$\beta\omega$=	1.69	1.69	1.69	1.69	1.69	
$\omega F_{ymax}A_{sc}$=	1466	846	677	507	338	Adjusted Brace Strength in Tension
$\beta\omega F_{ymax}A_{sc}$=	1979	1142	913	685	457	Adjusted Brace Strength in Compression

BEAM DESIGN

Beam Demands

	B608	B608	B608	B608	B608	ETABS Beam ID
$P_{ubm,c}$ (k) =	1193	745	596	447	298	Max. comp. due to brace tens., $P_{ubm,c}$ = cos(ψ_b) $\omega F_{ymax}A_{sc,b}$
$P_{ubm,t}$ (k) =	1610	1006	805	604	403	Max. tens. due to brace comp., $P_{ubm,t}$ = cos(ψ_b) $\beta\omega F_{ymax}A_{sc,b}$
$M_{E,drift}$ (k-ft) =	0	0	0	0	0	Drift-induced ETABS seismic moment neglected
$M_{Em,br}$ (k-ft) =	0	0	0	0	0	Seis. moment due to adj. brace strength, 0 for single diag. config.
M_{ug-} (k-ft) =	177	191	172	183	112	Factored gravity moment (from ETABS Analysis)
M_u (k-ft) =	177	191	172	183	112	M_{ug-} + $M_{Em,br}$
V_{Emh} (k) =	0.0	0.0	0.0	0.0	0.0	Seis. shear due to adj. brace strength, 0 for single diag. config.
V_{ug} (k) =	45	15	15	15	15	Factored gravity shear (from ETABS Analysis)
V_u (k) =	45	15	15	15	15	V_{ug} + V_{Em}

Beam Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Beam Size =	W24x162	W18x143	W18x119	W18x97	W18x86
A_g (in^2) =	47.8	42.0	35.1	28.5	25.3
t_f (in) =	1.22	1.32	1.06	0.87	0.77
t_w (in) =	0.71	0.73	0.66	0.54	0.48
d (in) =	25.0	19.5	19.0	18.6	18.4
b_f (in) =	13.0	11.2	11.3	11.1	11.1
Z_x (in^3) =	468	322	262	211	186
r_x(in) =	10.40	8.09	7.90	7.82	7.77
r_y(in) =	3.05	2.72	2.69	2.65	2.63
r_{ts}(in) =	3.57	3.17	3.13	3.08	3.05
h_o(in) =	23.80	18.20	17.90	17.70	17.60
J (in^4) =	18.50	19.20	10.60	5.86	4.10

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Engineering of Structures
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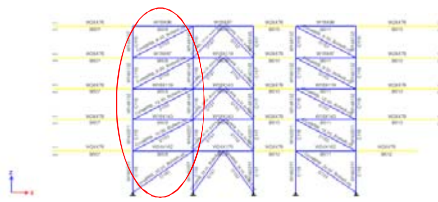
SUBJECT Byers Hall: BRBF Design

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL



BRBF LOCATION : GL B/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BEAM DESIGN (CONT'D)

Seismic Compactness Per AISC 341-05 Section16.5a/8.2b

Beam Compact Flange $b_f/2t_f$ =	5.3	4.2	5.3	6.4	7.2	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.2	7.2	7.2	7.2	7.2	
$b_f/2t_f \le (b/2t)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	
Beam Compact Web $(d-2t_f)/t_w$ =	32.0	23.1	25.8	31.5	35.1	
$C_a = P_u/\phi P_y$ =	0.55	0.39	0.38	0.35	0.26	
$2.45\ (E/F_y)^{0.5}\ (1 - 0.93C_a)$ =	28.6	37.4	38.3	39.9	44.6	if $C_a \le 0.125$
$0.77\ (E/F_y)^{0.5}\ (2.93 - C_a)$ =	44.1	47.0	47.3	47.9	49.5	if $C_a > 0.125$
$1.49\ (E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	44.1	47.0	47.3	47.9	49.5	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	2151	1890	1580	1283	1139	AISC 360-05 Equation D2-1
DCR =	0.75	0.53	0.51	0.47	0.35	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section E3 - Compression

L_x(ft) =	7.00	7.00	7.00	7.00	7.00	Strong axis unbraced length
L_y(ft) =	7.00	7.00	7.00	7.00	7.00	Weak axis unbraced length
k_x =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_x$ =	8.1	10.4	10.6	10.7	10.8	
k_y =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_y$ =	27.5	30.9	31.2	31.7	31.9	
F_e (ksi) =	377.35	300.11	293.52	284.86	280.58	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	47.3	46.6	46.6	46.5	46.4	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2035	1763	1471	1192	1057	AISC 360-05 Equation E3-1
DCR =	0.59	0.42	0.41	0.38	0.28	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section F2 - Flexure

L_p(ft) =	10.8	9.6	9.5	9.4	9.3	AISC 360-05 Equation F2-5
L_r (ft) =	35.8	39.6	34.3	30.4	28.6	AISC 360-05 Equation F2-6
C_b =	1.00	1.00	1.00	1.00	1.00	
S_x (in^3) =	414.0	282.0	231.0	188.0	166.0	
M_p (k-ft) =	1950	1342	1092	879	775	Z_xF_y
M_n (k-ft) =	1950	1342	1092	879	775	AISC 360-05 Equation F2-2
$\phi_b M_n$ (k-ft) =	1755	1208	983	791	698	
DCR =	0.10	0.16	0.18	0.23	0.16	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section H1 - Combined Compression & Flexure

P_u (k) =	1193	745	596	447	298	
M_u (k-ft) =	177	191	172	183	112	
$P_u/\phi_c P_{nc}$ =	0.59	0.42	0.41	0.38	0.28	
combined equation =	0.68	0.56	0.56	0.58	0.43	AISC 360-05 Equation H1-1a or H1-1b
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section H2 - Combined Tension & Flexure

P_u (k) =	1610	1006	805	604	403	
M_u (k-ft) =	177	191	172	183	112	
$P_u/\phi_c P_{nc}$ =	0.75	0.53	0.51	0.47	0.35	
combined equation =	0.84	0.67	0.67	0.68	0.50	AISC 360-05 Equation H1-1a or H1-1b
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section G2 - Shear

$\phi_v V_n$ (k) =	429	332	299	244	219	AISC 360-05 Equation G2-1
DCR =	0.10	0.05	0.05	0.06	0.07	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

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Engineering of Structures and Building Enclosures

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CLIENT UCSF

SUBJECT Byers Hall: BRBF Design

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL

BRBF LOCATION : GL B/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (RIGHT)

Column Demands

	C116	C116	C116	C116	C116	ETABS Column ID
P_{DL} (k) =	423	365	272	189	100	(from ETABS gravity analysis)
P_{LL} (k) =	297	250	174	105	36	(from ETABS gravity analysis)
$1.2DL + f_1LL + E_v$ =	733	629	462	313	157	$E_V = (0.2)(S_{DS})(DL)$
$0.9DL - E_v$ =	305	263	196	136	72	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	

Brace in Tension - Beam in Compression - Column in Compression

$V_{t,br}$ (k) =	852	399	319	240	160	Vert.component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perpendicular frames
$V_{t,br,adj}$ (k) =	-742	-659	-495	-412	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	-338	-448	-188	-13	160	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	395	181	274	300	316	

Brace in Compression - Beam in Tension - Column in Tension

$V_{c,br}$ (k) =	1150	539	431	323	216	Vertical component of the adj. brace force in compression
$V_{c,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perpendicular frames
$V_{c,br,adj}$ (k) =	-1001	-890	-668	-556	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	-456	-605	-254	-17	216	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	-761	-868	-449	-153	143	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x257	W14x257	W14x145	W14x145	W14x145
A_g (in^2) =	75.6	75.6	42.7	42.7	42.7
t_f (in) =	1.89	1.89	1.09	1.09	1.09
t_w (in) =	1.18	1.18	0.68	0.68	0.68
d (in) =	16.4	16.4	14.8	14.8	14.8
b_f (in) =	16	16	15.5	15.5	15.5
Z_x (in^3) =	487	487	260	260	260
Z_y (in^3) =	246	246	133	133	133
r_x (in) =	6.71	6.71	6.33	6.33	6.33
r_y (in) =	4.13	4.13	3.98	3.98	3.98
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	32.0	23.9	25.4	25.5	25.5
$(kL/r)_y$ =	52.1	38.9	40.5	40.6	40.6

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column Compact Flange $b_f/2t_f$ =	4.23	4.23	7.11	7.11	7.11	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	
Column Compact Web $(d-2t_f)/t_w$ =	10.7	10.7	18.6	18.6	18.6	
$C_a = P_u/\phi P_y$ =	0.12	0.05	0.14	0.16	0.16	
$2.45\ (E/F_y)^{0.5}\ (1 - 0.93C_a)$ =	52.6	56.1	51.2	50.4	50.0	if $C_a \le 0.125$
$0.77\ (E/F_y)^{0.5}\ (2.93 - C_a)$ =	52.2	53.3	51.7	51.4	51.3	if $C_a > 0.125$
$1.49\ (E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	52.6	56.1	51.7	51.4	51.3	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	3402.0	3402.0	1921.5	1921.5	1921.5	AISC 360-05 Equation D2-1
DCR =	No Tension	No Tension	No Tension	No Tension	0.07	
	Column OK	Column OK	Column OK	Column OK	Column OK	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	105.61	189.52	174.91	174.04	173.61	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.0	44.8	44.4	44.3	44.3	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2790	3046	1705	1704	1703	AISC 360-05 Equation E3-1
DCR =	0.14	0.06	0.16	0.18	0.19	
	Column OK	Column OK	Column OK	Column OK	Column OK	

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

CLIENT	UCSF	SHEET NO.	
SUBJECT	Byers Hall: BRBF Design	PROJECT NO.	197042.00-UCSF
		DATE	03/17/2020
		BY	LZhou
		CHECKED	

SINGLE BAY BRBF DESIGN - SINGLE DIAGONAL

BRBF LOCATION : GL B/3-4

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (LEFT)

Column Demands

	C115	C115	C115	C115	C115	ETABS Column ID
P_{DL} (k) =	664	524	383	256	133	
P_{LL} (k) =	473	363	249	145	42	
1.2DL + f_1LL + E_v =	1152	905	653	426	205	E_V = (0.2)(S_{DS})(DL)
0.9DL - E_v =	478	377	276	184	96	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	

Brace in Compression - Beam in Tension - Column in Compression

$V_{c,br}$ (k) =	539	431	323	216	0	Vertical component of the adj. brace force in comp.
$V_{c,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perpendicular frames
$V_{c,br,adj}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3^{*}\Sigma P_{em,perp}$ (k) =	1509	970	539	216	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	2662	1875	1192	642	205	

Brace in Tension - Beam in Compression - Column in Tension

$V_{t,br}$ (k) =	399	319	240	160	0	Vertical component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perpendicular frames
$V_{t,br,adj}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3^{*}\Sigma P_{em,perp}$ (k) =	1118	719	399	160	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	640	341	124	-25	-96	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x311	W14x311	W14x132	W14x132	W14x132
A_g (in^2) =	91.4	91.4	38.8	38.8	38.8
t_f (in) =	2.26	2.26	1.03	1.03	1.03
t_w (in) =	1.41	1.41	0.65	0.65	0.65
d (in) =	17.1	17.1	14.7	14.7	14.7
b_f (in) =	16.2	16.2	14.7	14.7	14.7
Z_x (in^3) =	603	603	234	234	234
Z_y (in^3) =	304	304	113	113	113
r_x (in) =	6.88	6.88	6.28	6.28	6.28
r_y (in) =	4.2	4.2	3.76	3.76	3.76
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	31.3	23.3	25.6	25.7	25.7
$(kL/r)_y$ =	51.2	38.2	42.8	42.9	43.0

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column shall satisfy highly ductile requirements

Column Compact Flange $b_f/2t_f$ =	3.58	3.58	7.14	7.14	7.14	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	
Column Compact Web $(d-2t_f)/t_w$ =	8.9	8.9	19.6	19.6	19.6	
$C_a = P_u/\phi P_y$ =	0.65	0.46	0.68	0.37	0.12	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	23.5	34.0	21.5	38.8	52.6	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	42.3	45.9	41.7	47.5	52.2	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	42.3	45.9	41.7	47.5	52.6	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	4113.0	4113.0	1746.0	1746.0	1746.0	AISC 360-05 Equation D2-1
DCR =	0.16	0.08	0.07	No Tension	No Tension	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	109.22	196.00	156.11	155.33	154.95	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.3	44.9	43.7	43.7	43.7	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	3396	3696	1527	1526	1525	AISC 360-05 Equation E3-1
DCR =	0.78	0.51	0.78	0.42	0.13	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

ASCE 7-05/AISC 341-05
LFRS Seismic Provisions

Detailed Frame Checks

GL B/4-5 Chevron

Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL B/4-5

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BRBF GEOMETRY:

	Level 2	Level 3	Level 4	Level 5	Roof	
L(ft) =	28.00	28.00	28.00	28.00	28.00	Bay Width (Column C-C)
h_i (ft) =	20.00	15.00	15.00	15.00	15.00	Story Height
L_{diag} (ft)=	24.4	20.5	20.5	20.5	20.5	Work Point - Work Point
cos ψ =	0.573	0.682	0.682	0.682	0.682	ψ = angle between brace and horizontal axis
sin ψ =	0.819	0.731	0.731	0.731	0.731	

BRACE DESIGN

AISC 341-05 Section 16.2a - Brace Strength

	D152	D152	D152	D152	D152	
F_{ysc} =	34 ksi	34 ksi	34 ksi	34 ksi	34 ksi	Minimum yield stress of the steel core
F_{ymax} =	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	Max yield stress of the steel core (R_yF_{ysc}; R_y = 1.1, F_{ysc} per dwg)
Dead Load (k) =	46.7	43.0	49.4	43.4	37.7	(from ETABS analysis)
Live Load (k) =	34.2	33.1	37.4	32.7	18.4	(from ETABS analysis)
Seismic Load (k) =	505.6	415.6	387.9	304.3	218.6	Primary direction (from ETABS analysis)
	95.0	67.8	59.6	51.8	41.0	Perpendicular direction (from ETABS analysis)
Combined Axial Load, P_u (k) =	616	512	493	396	292	(1.2+0.2S_{DS})D + 0.5 L + ρ E (include 100%+30% effects)
Steel Core Area (sq.in.) =	26.0	18.0	16.0	12.0	10.0	
ϕ P_{ysc} (k) =	796	551	490	367	306	$\phi F_{ysc}A_{sc}$ (AISC 341-05 Equation 16-1)
DCR =	0.77	0.93	1.01	1.08	0.95	$P_u/\phi P_{ysc}$
	Brace OK	***Brace OK***	***Revise***	***Revise***	***Brace OK***	

AISC 341-05 Section 16.2d - Adjusted Brace Strength

ω=	1.25	1.25	1.25	1.25	1.25	Strain hardening adjustment factor (assumed)
β=	1.35	1.35	1.35	1.35	1.35	Comp.strength adjustment factor (assumed)
$\beta\omega$=	1.69	1.69	1.69	1.69	1.69	
$\omega F_{ymax}A_{sc}$=	1466	1015	902	677	564	Adjusted Brace Strength in Tension
$\beta\omega F_{ymax}A_{sc}$=	1979	1370	1218	913	761	Adjusted Brace Strength in Compression

BEAM DESIGN

Beam Demands

	B609	B609	B609	B609	B609	ETABS Beam ID
P_{Emh} (k) =	1231	847	753	565	471	Axial load due to sum of adj brace forces (tension & compression)
$M_{E,drift}$ (k-ft) =	0	0	0	0	0	Drift-induced ETABS seismic moment neglected
P_y (kip) =	420	260	231	173	144	Vertical unbalanced force due to adj. brace strength
M_{Emh} (k-ft) =	2942	1818	1616	1212	1010	$V_{Em,br}$*L/4
V_{Emh} (k) =	210.1	129.8	115.4	86.5	72.1	Seis. shear due to adj. brace strength
V_{ug} (k) =	0	15	15	15	15	Factored gravity shear (from ETABS Analysis)
V_u (k) =	210	145	130	102	87	V_{ug} + V_{Em}

Beam Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Beam Size =	W24x176	W18x143	W18x143	W18x119	W18x97
A_g (in^2) =	51.7	42.0	42.0	35.1	28.5
t_f (in) =	1.34	1.32	1.32	1.06	0.87
t_w (in) =	0.75	0.73	0.73	0.66	0.54
d (in) =	25.2	19.5	19.5	19.0	18.6
b_f (in) =	12.9	11.2	11.2	11.3	11.1
Z_x (in^3) =	511	322	322	262	211
r_x(in) =	10.50	8.09	8.09	7.90	7.82
r_y(in) =	3.04	2.72	2.72	2.69	2.65
r_{ts}(in) =	3.57	3.17	3.17	3.13	3.08
h_o(in) =	23.90	18.20	18.20	17.90	17.70
J (in^4) =	23.90	19.20	19.20	10.60	5.86

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CLIENT	UCSF	SHEET NO.	
		PROJECT NO.	197042.00-UCSF
		DATE	03/16/2020
SUBJECT	Byers Hall: BRBF Chevron Unbalanced Force	BY	LZhou
		CHECKED	

Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL B/4-5

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BEAM DESIGN (CONT'D)

Seismic Compactness Per AISC 341-05 Section16.5a/8.2b

Beam Compact Flange $b_f/2t_f$ =	4.8	4.2	4.2	5.3	6.4	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.2	7.2	7.2	7.2	7.2	
$b_f/2t_f \le (b/2t)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	
Beam Compact Web $(d-2t_f)/t_w$ =	30.0	23.1	23.1	25.8	31.5	
$C_a = P_u/\phi P_y$ =	0.53	0.45	0.40	0.36	0.37	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	30.0	34.4	37.1	39.4	38.9	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	44.5	46.0	46.9	47.7	47.5	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	44.5	46.0	46.9	47.7	47.5	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	2327	1890	1890	1580	1283	AISC 360-05 Equation D2-1
DCR =	0.53	0.45	0.40	0.36	0.37	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section E3 - Compression

L_x(ft) =	7.00	7.00	7.00	7.00	7.00	Strong axis unbraced length
L_y(ft) =	7.00	7.00	7.00	7.00	7.00	Weak axis unbraced length
k_x =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_x$ =	8.0	10.4	10.4	10.6	10.7	
k_y =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_y$ =	27.6	30.9	30.9	31.2	31.7	
F_e (ksi) =	374.87	300.11	300.11	293.52	284.86	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	47.3	46.6	46.6	46.6	46.5	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2200	1763	1763	1471	1192	AISC 360-05 Equation E3-1
DCR =	0.56	0.48	0.43	0.38	0.39	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section F2 - Flexure

L_p(ft) =	10.7	9.6	9.6	9.5	9.4	AISC 360-05 Equation F2-5
L_r (ft) =	37.4	39.6	39.6	34.3	30.4	AISC 360-05 Equation F2-6
C_b =	1.00	1.00	1.00	1.00	1.00	
S_x (in^3) =	450.0	282.0	282.0	231.0	188.0	
M_p (k-ft) =	2129	1342	1342	1092	879	Z_xF_y
M_n (k-ft) =	2129	1342	1342	1092	879	AISC 360-05 Equation F2-2
$\phi_b M_n$ (k-ft) =	1916	1208	1208	983	791	
DCR =	1.54	1.51	1.34	1.23	1.28	
	Revise	Revise	Revise	Revise	Revise	

AISC 360-05 Section H1 - Combined Compression & Flexure

P_u (k) =	1231	847	753	565	471	
M_u (k-ft) =	2942	1818	1616	1212	1010	
$P_u/\phi_c P_{nc}$ =	0.56	0.48	0.43	0.38	0.39	
combined equation =	1.92	1.82	1.62	1.48	1.53	AISC 360-05 Equation H1-1a or H1-1b
	Revise	Revise	Revise	Revise	Revise	

AISC 360-05 Section H2 - Combined Tension & Flexure

P_u (k) =	1231	847	753	565	471	
M_u (k-ft) =	2942	1818	1616	1212	1010	
$P_u/\phi_c P_{nc}$ =	0.53	0.45	0.40	0.36	0.37	
combined equation =	1.89	1.79	1.59	1.45	1.50	AISC 360-05 Equation H1-1a or H1-1b
	Revise	Revise	Revise	Revise	Revise	

AISC 360-05 Section G2 - Shear

$\phi_v V_n$ (k) =	456	332	332	299	244	AISC 360-05 Equation G2-1
DCR =	0.46	0.44	0.39	0.34	0.36	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

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CLIENT UCSF
SUBJECT Byers Hall: BRBF Chevron Unbalanced Force

SHEET NO.
PROJECT NO. 197042.00-UCSF
DATE 03/16/2020
BY LZhou
CHECKED

Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL B/4-5

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (RIGHT)

Column Demands

	C117	C117	C117	C117	C117	ETABS Column ID
P_{DL} (k) =	364	290	201	122	45	(from ETABS gravity analysis)
P_{LL} (k) =	249	194	125	63	10	(from ETABS gravity analysis)
$1.2DL + f_1LL + E_v$ =	627	497	339	200	67	$E_V = (0.2)(S_{DS})(DL)$
$0.9DL - E_v$ =	262	209	144	88	32	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	

Brace in Compression - Column in Compression

$V_{c,br}$ (k) =	1122	890	668	556	0	Vert.component of the adj. brace force in compression
$V_{c,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perp. frames
$V_{c,br,adj}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3^*\Sigma P_{em,perp}$ (k) =	3237	2114	1224	556	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	3864	2611	1563	756	67	

Brace in Tension - Column in Tension

$V_{t,br}$ (k) =	831	659	495	412	0	Vertical component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from perp. frames
$V_{t,br,adj}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3^*\Sigma P_{em,perp}$ (k) =	2397	1566	907	412	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	2135	1357	762	324	-32	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x311	W14x311	W14x145	W14x145	W14x145
A_g (in^2) =	91.4	91.4	42.7	42.7	42.7
t_f (in) =	2.26	2.26	1.09	1.09	1.09
t_w (in) =	1.41	1.41	0.68	0.68	0.68
d (in) =	17.1	17.1	14.8	14.8	14.8
b_f (in) =	16.2	16.2	15.5	15.5	15.5
Z_x (in^3) =	603	603	260	260	260
Z_y (in^3) =	304	304	133	133	133
r_x (in) =	6.88	6.88	6.33	6.33	6.33
r_y (in) =	4.2	4.2	3.98	3.98	3.98
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.4	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	31.2	23.3	25.4	25.4	25.5
$(kL/r)_y$ =	51.1	38.2	40.3	40.5	40.6

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column Compact Flange $b_f/2t_f$ =	3.58	3.58	7.11	7.11	7.11	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	
Column Compact Web $(d-2t_f)/t_w$ =	8.9	8.9	18.6	18.6	18.6	
$C_a = P_u/\phi P_y$ =	0.94	0.63	0.81	0.39	0.03	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	7.5	24.2	14.4	37.4	57.1	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	36.9	42.6	39.2	47.0	53.7	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	36.9	42.6	39.2	47.0	57.1	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	4113.0	4113.0	1921.5	1921.5	1921.5	AISC 360-05 Equation D2-1
DCR =	0.52	0.33	0.40	0.17	No Tension	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	109.43	196.00	176.00	174.91	174.04	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.3	44.9	44.4	44.4	44.3	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	3397	3696	1706	1705	1704	AISC 360-05 Equation E3-1
DCR =	1.14	0.71	0.92	0.44	0.04	
	Revise	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

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Engineering of Structures and Building Enclosures

CLIENT UCSF
SUBJECT Byers Hall: BRBF Chevron Unbalanced Force

SHEET NO.
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DATE 03/16/2020
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CHECKED

Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL B/4-5

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (LEFT)

Column Demands

	C116	C116	C116	C116	C116	ETABS Column ID
P_{DL} (k) =	423	365	272	189	100	
P_{LL} (k) =	297	250	174	105	36	
$1.2DL + f_1LL + E_v$ =	733	629	462	313	157	$E_V = (0.2)(S_{DS})(DL)$
$0.9DL - E_v$ =	305	263	196	136	72	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	

Brace in Compression - Column in Compression

$V_{c,br}$ (k) =	1001	890	668	556	0	Vertical component of the adj. brace force in comp.
$V_{c,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force in comp from perp. frames
$V_{c,br,adj}$ (k) =	-1150	-539	-431	-323	-216	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	456	605	254	17	-216	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	1189	1234	716	331	-59	

Brace in Tension - Column in Tension

$V_{t,br}$ (k) =	742	659	495	412	0	Vertical component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force in comp from perp. frames
$V_{t,br,adj}$ (k) =	-852	-399	-319	-240	-160	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	338	448	188	13	-160	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	33	185	-8	-123	-232	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x257	W14x257	W14x145	W14x145	W14x145
A_g (in^2) =	75.6	75.6	42.7	42.7	42.7
t_f (in) =	1.89	1.89	1.09	1.09	1.09
t_w (in) =	1.18	1.18	0.68	0.68	0.68
d (in) =	16.4	16.4	14.8	14.8	14.8
b_f (in) =	16	16	15.5	15.5	15.5
Z_x (in^3) =	487	487	260	260	260
Z_y (in^3) =	246	246	133	133	133
r_x (in) =	6.71	6.71	6.33	6.33	6.33
r_y (in) =	4.13	4.13	3.98	3.98	3.98
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.4	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	32.0	23.9	25.4	25.4	25.5
$(kL/r)_y$ =	52.0	38.9	40.3	40.5	40.6

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column shall satisfy highly ductile requirements

Column Compact Flange $b_f/2t_f$ =	4.23	4.23	7.11	7.11	7.11	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	
Column Compact Web $(d-2t_f)/t_w$ =	10.7	10.7	18.6	18.6	18.6	
$C_a = P_u/\phi P_y$ =	0.35	0.36	0.37	0.17	-0.03	
$2.45\,(E/F_y)^{0.5}\,(1 - 0.93C_a)$ =	39.8	39.1	38.6	49.6	60.7	if $C_a \le 0.125$
$0.77\,(E/F_y)^{0.5}\,(2.93 - C_a)$ =	47.9	47.6	47.4	51.1	54.9	if $C_a > 0.125$
$1.49\,(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	47.9	47.6	47.4	51.1	60.7	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	3402.0	3402.0	1921.5	1921.5	1921.5	AISC 360-05 Equation D2-1
DCR =	0.01	0.05	No Tension	No Tension	No Tension	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	105.81	189.52	176.00	174.91	174.04	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.0	44.8	44.4	44.4	44.3	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2792	3046	1706	1705	1704	AISC 360-05 Equation E3-1
DCR =	0.43	0.41	0.42	0.19	No Compress.	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

ASCE 7-05/AISC 341-05
LFRS Seismic Provisions

Detailed Frame Checks

GL 3/D-E Chevron

SIMPSON GUMPERTZ & HEGER
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SHEET NO.
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Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL 3/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BRBF GEOMETRY:

	Level 2	Level 3	Level 4	Level 5	Roof	
L(ft) =	21.00	21.00	21.00	21.00	21.00	Bay Width (Column C-C)
h_i (ft) =	20.00	15.00	15.00	15.00	15.00	Story Height
L_{diag} (ft)=	22.6	18.3	18.3	18.3	18.3	Work Point - Work Point
cos ψ =	0.465	0.573	0.573	0.573	0.573	ψ = angle between brace and horizontal axis
sin ψ =	0.885	0.819	0.819	0.819	0.819	

BRACE DESIGN

AISC 341-05 Section 16.2a - Brace Strength

	D32	D32	D32	D32	D32	
F_{ysc} =	34 ksi	34 ksi	34 ksi	34 ksi	34 ksi	Minimum yield stress of the steel core
F_{ymax} =	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	Max yield stress of the steel core (R_yF_{ysc}; R_y = 1.1, F_{ysc} per dwg)
Dead Load (k) =	34.0	23.2	28.1	18.5	10.1	(from ETABS analysis)
Live Load (k) =	26.8	19.4	24.0	16.5	11.0	(from ETABS analysis)
Seismic Load (k) =	373.1	305.2	304.5	213.8	127.3	Primary direction (from ETABS analysis)
	252.9	201.2	200.2	132.3	70.3	Perpendicular direction (from ETABS analysis)
Combined Axial Load, P_u (k) =	509	407	415	287	168	(1.2+0.2S_{DS})D + 0.5 L + ρ E (include 100%+30% effects)
Steel Core Area (sq.in.) =	24.0	16.0	16.0	10.0	6.0	
ϕ P_{ysc} (k) =	734	490	490	306	184	$\phi F_{ysc}A_{sc}$ (AISC 341-05 Equation 16-1)
DCR =	0.69	0.83	0.85	0.94	0.91	$P_u/\phi P_{ysc}$
	Brace OK	*Brace OK*	*Brace OK*	*Brace OK*	*Brace OK*	

AISC 341-05 Section 16.2d - Adjusted Brace Strength

ω=	1.25	1.25	1.25	1.25	1.25	Strain hardening adjustment factor (assumed)
β=	1.35	1.35	1.35	1.35	1.35	Comp.strength adjustment factor (assumed)
$\beta\omega$=	1.69	1.69	1.69	1.69	1.69	
$\omega F_{ymax}A_{sc}$=	1353	902	902	564	338	Adjusted Brace Strength in Tension
$\beta\omega F_{ymax}A_{sc}$=	1827	1218	1218	761	457	Adjusted Brace Strength in Compression

BEAM DESIGN

Beam Demands

	B256	B256	B256	B256	B256	ETABS Beam ID
P_{Emh} (k) =	1123	757	757	473	284	Axial load due to sum of adj brace forces (tension & compression)
$M_{E,drift}$ (k-ft) =	0	0	0	0	0	Drift-induced ETABS seismic moment neglected
P_y (kip) =	419	259	259	162	97	Vertical unbalanced force due to adj. brace strength
M_{Emh} (k-ft) =	2201	1358	1358	849	509	$V_{Em,br}$*L/4
V_{Emh} (k) =	209.6	129.3	129.3	80.8	48.5	Seis. shear due to adj. brace strength
V_{ug} (k) =	0	15	15	15	15	Factored gravity shear (from ETABS Analysis)
V_u (k) =	210	144	144	96	64	V_{ug} + V_{Em}

Beam Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Beam Size =	W24x162	W18x143	W18x119	W18x97	W18x86
A_g (in^2) =	47.8	42.0	35.1	28.5	25.3
t_f (in) =	1.22	1.32	1.06	0.87	0.77
t_w (in) =	0.71	0.73	0.66	0.54	0.48
d (in) =	25.0	19.5	19.0	18.6	18.4
b_f (in) =	13.0	11.2	11.3	11.1	11.1
Z_x (in^3) =	468	322	262	211	186
r_x(in) =	10.40	8.09	7.90	7.82	7.77
r_y(in) =	3.05	2.72	2.69	2.65	2.63
r_{ts}(in) =	3.57	3.17	3.13	3.08	3.05
h_o(in) =	23.80	18.20	17.90	17.70	17.60
J (in^4) =	18.50	19.20	10.60	5.86	4.10

Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL 3/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BEAM DESIGN (CONT'D)

Seismic Compactness Per AISC 341-05 Section16.5a/8.2b

Beam Compact Flange $b_f/2t_f$ =	5.3	4.2	5.3	6.4	7.2	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.2	7.2	7.2	7.2	7.2	
$b_f/2t_f \le (b/2t)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	
Beam Compact Web $(d-2t_f)/t_w$ =	32.0	23.1	25.8	31.5	35.1	
$C_a = P_u/\phi P_y$ =	0.52	0.40	0.48	0.37	0.25	
$2.45\ (E/F_y)^{0.5}\ (1 - 0.93C_a)$ =	30.4	37.0	32.7	38.7	45.3	if $C_a \le 0.125$
$0.77\ (E/F_y)^{0.5}\ (2.93 - C_a)$ =	44.7	46.9	45.4	47.5	49.7	if $C_a > 0.125$
$1.49\ (E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	44.7	46.9	45.4	47.5	49.7	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	2151	1890	1580	1283	1139	AISC 360-05 Equation D2-1
DCR =	0.52	0.40	0.48	0.37	0.25	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section E3 - Compression

L_x(ft) =	7.00	7.00	7.00	7.00	7.00	Strong axis unbraced length
L_y(ft) =	7.00	7.00	7.00	7.00	7.00	Weak axis unbraced length
k_x =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_x$ =	8.1	10.4	10.6	10.7	10.8	
k_y =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_y$ =	27.5	30.9	31.2	31.7	31.9	
F_e (ksi) =	377.35	300.11	293.52	284.86	280.58	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	47.3	46.6	46.6	46.5	46.4	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2035	1763	1471	1192	1057	AISC 360-05 Equation E3-1
DCR =	0.55	0.43	0.51	0.40	0.27	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section F2 - Flexure

L_p(ft) =	10.8	9.6	9.5	9.4	9.3	AISC 360-05 Equation F2-5
L_r (ft) =	35.8	39.6	34.3	30.4	28.6	AISC 360-05 Equation F2-6
C_b =	1.00	1.00	1.00	1.00	1.00	
S_x (in^3) =	414.0	282.0	231.0	188.0	166.0	
M_p (k-ft) =	1950	1342	1092	879	775	Z_xF_y
M_n (k-ft) =	1950	1342	1092	879	775	AISC 360-05 Equation F2-2
$\phi_b M_n$ (k-ft) =	1755	1208	983	791	698	
DCR =	1.25	1.12	1.38	1.07	0.73	
	Revise	Revise	Revise	Revise	Beam OK	

AISC 360-05 Section H1 - Combined Compression & Flexure

P_u (k) =	1123	757	757	473	284	
M_u (k-ft) =	2201	1358	1358	849	509	
$P_u/\phi_c P_{nc}$ =	0.55	0.43	0.51	0.40	0.27	
combined equation =	1.67	1.43	1.74	1.35	0.92	AISC 360-05 Equation H1-1a or H1-1b
	Revise	Revise	Revise	Revise	Beam OK	

AISC 360-05 Section H2 - Combined Tension & Flexure

P_u (k) =	1123	757	757	473	284	
M_u (k-ft) =	2201	1358	1358	849	509	
$P_u/\phi_c P_{nc}$ =	0.52	0.40	0.48	0.37	0.25	
combined equation =	1.64	1.40	1.71	1.32	0.90	AISC 360-05 Equation H1-1a or H1-1b
	Revise	Revise	Revise	Revise	Beam OK	

AISC 360-05 Section G2 - Shear

$\phi_v V_n$ (k) =	429	332	299	244	219	AISC 360-05 Equation G2-1
DCR =	0.49	0.43	0.48	0.39	0.29	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

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SHEET NO.	
PROJECT NO.	197042.00-UCSF
DATE	03/16/2020
BY	LZhou
CHECKED	

CLIENT UCSF
SUBJECT Byers Hall: BRBF Chevron Unbalanced Force

Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL 3/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (RIGHT)

Column Demands

	C65	C65	C65	C65	C65	ETABS Column ID
P_{DL} (k) =	299	249	180	116	55	(from ETABS gravity analysis)
P_{LL} (k) =	238	201	147	96	44	(from ETABS gravity analysis)
$1.2DL + f_1LL + E_v$ =	532	443	322	208	98	$E_V = (0.2)(S_{DS})(DL)$
$0.9DL - E_v$ =	215	179	129	84	40	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	

Brace in Compression - Column in Compression

$V_{c,br}$ (k) =	1078	998	623	374	0	Vert.component of the adj. brace force in compression
$V_{c,br,perp}$ (k) =	539	431	323	216	0	Vert. component of the adj. brace force from perp. frames
$V_{c,br,adj}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	3526	2286	1159	439	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	4058	2730	1481	647	98	

Brace in Tension - Column in Tension

$V_{t,br}$ (k) =	799	739	462	277	0	Vertical component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	399	319	240	160	0	Vert. component of the adj. brace force from perp. frames
$V_{t,br,adj}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	2612	1694	859	325	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	2397	1515	729	241	-40	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x257	W14x257	W14x132	W14x132	W14x132
A_g (in^2) =	75.6	75.6	38.8	38.8	38.8
t_f (in) =	1.89	1.89	1.03	1.03	1.03
t_w (in) =	1.18	1.18	0.65	0.65	0.65
d (in) =	16.4	16.4	14.7	14.7	14.7
b_f (in) =	16	16	14.7	14.7	14.7
Z_x (in^3) =	487	487	234	234	234
Z_y (in^3) =	246	246	113	113	113
r_x (in) =	6.71	6.71	6.28	6.28	6.28
r_y (in) =	4.13	4.13	3.76	3.76	3.76
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	32.0	23.9	25.6	25.7	25.7
$(kL/r)_y$ =	52.1	38.9	42.8	42.9	43.0

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column Compact Flange $b_f/2t_f$ =	4.23	4.23	7.14	7.14	7.14	
$(b/2t)_{max}=0.30\ (E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	
Column Compact Web $(d-2t_f)/t_w$ =	10.7	10.7	19.6	19.6	19.6	
$C_a = P_u/\phi P_y$ =	1.19	0.80	0.85	0.37	0.06	
$2.45\ (E/F_y)^{0.5}\ (1 - 0.93C_a)$ =	-6.4	15.0	12.5	38.7	55.9	if $C_a \le 0.125$
$0.77\ (E/F_y)^{0.5}\ (2.93 - C_a)$ =	32.2	39.5	38.6	47.5	53.3	if $C_a > 0.125$
$1.49\ (E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	35.9	39.5	38.6	47.5	55.9	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	3402.0	3402.0	1746.0	1746.0	1746.0	AISC 360-05 Equation D2-1
DCR =	0.70	0.45	0.42	0.14	No Tension	
	Column OK	Column OK	Column OK	Column OK	Column OK	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	105.61	189.52	156.11	155.33	154.95	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.0	44.8	43.7	43.7	43.7	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2790	3046	1527	1526	1525	AISC 360-05 Equation E3-1
DCR =	1.45	0.90	0.97	0.42	0.06	
	Revise	Column OK	Column OK	Column OK	Column OK	

SIMPSON GUMPERTZ & HEGER
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CLIENT UCSF
SUBJECT Byers Hall: BRBF Chevron Unbalanced Force

SHEET NO.
PROJECT NO. 197042.00-UCSF
DATE 03/16/2020
BY LZhou
CHECKED

Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL 3/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (LEFT)

Column Demands						
	C48	C48	C48	C48	C48	ETABS Column ID
P_{DL} (k) =	379	302	219	148	81	
P_{LL} (k) =	317	256	190	130	72	
1.2DL + f_1LL + E_v =	682	545	398	269	148	E_V = (0.2)(S_{DS})(DL)
0.9DL - E_v =	273	218	158	107	58	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	
Brace in Compression - Column in Compression						
$V_{c,br}$ (k) =	998	998	623	374	0	Vertical component of the adj. brace force in comp.
$V_{c,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force in comp from perp. frames
$V_{c,br,adj}$ (k) =	-739	-739	-462	-277	0	Vert. component of the adj. brace force from adjacent frames
ΣP_{em} + $\Sigma P_{em,adj}$ + 0.3*$\Sigma P_{em,perp}$ (k) =	776	517	259	97	0	Sum of axial forces in column due to adj. brace force at all levels
P_{uc} = ΣP_{em} + $P_{u,grav}$ (k) =	1457	1063	656	366	148	
Brace in Tension - Column in Tension						
$V_{t,br}$ (k) =	739	739	462	277	0	Vertical component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force in comp from perp. frames
$V_{t,br,adj}$ (k) =	-998	-998	-623	-374	0	Vert. component of the adj. brace force from adjacent frames
ΣP_{em} + $\Sigma P_{em,adj}$ + 0.3*$\Sigma P_{em,perp}$ (k) =	-776	-517	-259	-97	0	Sum of axial forces in column due to adj. brace force at all levels
P_{ut} = ΣP_{em} - $P_{u,grav}$ (k) =	-1049	-735	-417	-204	-58	
Column Geometric Properties						
F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi	
Column Size =	W14x342	W14x342	W14x145	W14x145	W14x145	
A_g (in^2) =	101.0	101.0	42.7	42.7	42.7	
t_f (in) =	2.47	2.47	1.09	1.09	1.09	
t_w (in) =	1.54	1.54	0.68	0.68	0.68	
d (in) =	17.5	17.5	14.8	14.8	14.8	
b_f (in) =	16.4	16.4	15.5	15.5	15.5	
Z_x (in^3) =	672	672	260	260	260	
Z_y (in^3) =	338	338	133	133	133	
r_x (in) =	6.98	6.98	6.33	6.33	6.33	
r_y (in) =	4.24	4.24	3.98	3.98	3.98	
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5	
k_x =	1.00	1.00	1.00	1.00	1.00	
k_y =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_x$ =	30.8	23.0	25.4	25.5	25.5	
$(kL/r)_y$ =	50.7	37.9	40.5	40.6	40.6	
Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b						Column shall satisfy highly ductile requirements
Column Compact Flange $b_f/2t_f$ =	3.32	3.32	7.11	7.11	7.11	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	
Column Compact Web $(d-2t_f)/t_w$ =	8.2	8.2	18.6	18.6	18.6	
C_a = $P_u/\phi P_y$ =	0.32	0.23	0.34	0.19	0.08	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	41.4	46.2	40.3	48.5	54.8	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	48.4	50.0	48.0	50.8	52.9	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	48.4	50.0	48.0	50.8	54.8	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Column OK	Column OK	Column OK	Column OK	Column OK	
AISC 360-05 Section D2 - Tension						
$\phi_t P_{nt}$ (k) =	4545.0	4545.0	1921.5	1921.5	1921.5	AISC 360-05 Equation D2-1
DCR =	No Tension	No Tension	No Tension	No Tension	No Tension	
	Column OK	Column OK	Column OK	Column OK	Column OK	
AISC 360-05 Section E3 - Compression						
F_e (ksi) =	111.31	199.75	174.91	174.04	173.61	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.4	45.0	44.4	44.3	44.3	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	3766	4093	1705	1704	1703	AISC 360-05 Equation E3-1
DCR =	0.39	0.26	0.38	0.22	0.09	
	Column OK	Column OK	Column OK	Column OK	Column OK	

ASCE 7-05/AISC 341-05
LFRS Seismic Provisions

Brace Connection Checks

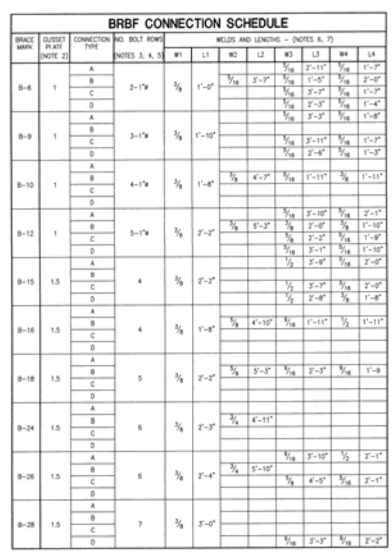
BRBF Brace Connection Checks

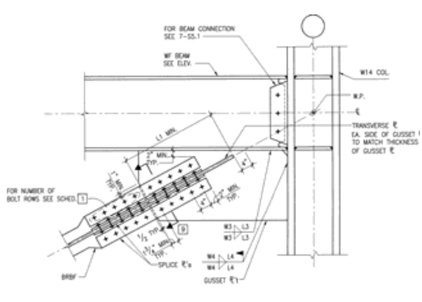
BRB Size, A_{sc}	Adjusted Brace Strength						Bolt Shear						
	F_{ymax}	ω	β	$\beta\omega$	T_{max}	P_{max}	$n_{bolts/leg}$	n_{legs}	n_{bolts}	ϕV_{bolt}	ϕV_n	V_u	DCR
[in^2]	[ksi]				[kip]	[kip]				[kips]	[kips]	[kips]	
6	45.1	1.25	1.35	1.69	338	457	2	4	8	56.5	452	457	1.01
9	45.1	1.25	1.35	1.69	507	685	3	4	12	56.5	678	685	1.01
10	45.1	1.25	1.35	1.69	564	761	4	4	16	56.5	904	761	0.84
12	45.1	1.25	1.35	1.69	677	913	5	4	20	56.5	1130	913	0.81
15	45.1	1.25	1.35	1.69	846	1142	4	4	16	88.4	1414	1142	0.81
16	45.1	1.25	1.35	1.69	902	1218	4	4	16	88.4	1414	1218	0.86
18	45.1	1.25	1.35	1.69	1015	1370	5	4	20	88.4	1768	1370	0.77
24	45.1	1.25	1.35	1.69	1353	1827	6	4	24	88.4	2122	1827	0.86
26	45.1	1.25	1.35	1.69	1466	1979	6	4	24	88.4	2122	1979	0.93
28	45.1	1.25	1.35	1.69	1579	2131	7	4	28	88.4	2475	2131	0.86

BRB Size, A_{sc}	Gusset Plate Yield							Splice Plate Yield						
	t_{GP}	L	$b_{Whitmore}$	F_{yGP}	ϕT_n	T_u	DCR	t_{SP}	b_{SP}	F_{ySP}	n_{SP}	ϕT_n	T_u	DCR
[in^2]	[in]	[in.]	[in]	[ksi]	[kip]	[kips]		[in]	[in]	[ksi]		[kip]	[kips]	
6	1	7.75	17.4	50	872	457	0.52	0.5	5	50	8	1000	457	0.46
9	1	11.75	22.1	50	1103	685	0.62	0.5	5	50	8	1000	685	0.68
10	1	15.75	26.7	50	1334	761	0.57	0.5	5	50	8	1000	761	0.76
12	1	19.75	31.3	50	1565	913	0.58	0.5	5	50	8	1000	913	0.91
15	1.5	15.75	26.7	50	2001	1142	0.57	1	5	50	8	2000	1142	0.57
16	1.5	15.75	26.7	50	2001	1218	0.61	1	5	50	8	2000	1218	0.61
18	1.5	19.75	31.3	50	2348	1370	0.58	1	5	50	8	2000	1370	0.68
24	1.5	23.75	35.9	50	2694	1827	0.68	1	5	50	8	2000	1827	0.91
26	1.5	23.75	35.9	50	2694	1979	0.73	1	5	50	8	2000	1979	0.99
28	1.5	27.75	40.5	50	3041	2131	0.70	1	5	50	8	2000	2131	1.07

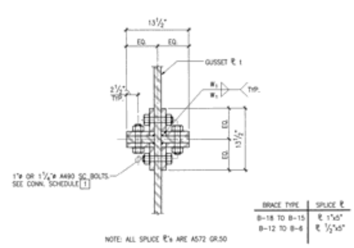
BRB Size, A_{sc}	Wing Plate Welds					
	W1	L1	n_{welds}	ϕV_n	T_u	DCR
[in^2]	[in]	[in]		[kip]	[kips]	
6	0.375	12	4	401	228	0.57
9	0.375	22	4	735	342	0.47
10	0.375	20	4	668	381	0.57
12	0.375	26	4	869	457	0.53
15	0.375	26	4	869	571	0.66
16	0.375	20	4	668	609	0.91
18	0.375	26	4	869	685	0.79
24	0.375	27	4	902	913	1.01
26	0.375	28	4	935	989	1.06
28	0.375	36	4	1203	1065	0.89

Notes:
1. Gusset plate buckling is ok by inspection.
2. Gusset plate block shear is not applicable.
3. Gusset plate to column/base plate welds not checked for Tier 1 analysis.





TYP. BRACE TO COLUMN FLANGE CONNECTION 1"=1'-0" 3



TYPICAL BRACE CONNECTION CROSS SECTION 1 1/2"=1'-0" 9

Appendix C

ASCE 41-17 Tier 1 Checklists

UC Campus:	**San Francisco**			Date:	**12 February 2020**		
Building CAAN:	**3034**	Auxiliary CAAN:		By Firm:	**Simpson Gumpertz & Heger**		
Building Name:	**Byers Hall**			Initials:	***LZ***	Checked:	***KDP***
Building Address:	**1700 4th Street, San Francisco, CA 94158**			Page:	**1**	of	**3**

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

LOW SEISMICITY

BUILDING SYSTEMS - *GENERAL*

	Description
C NC N/A U ☒ ☐ ☐ ☐	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1) **Comments: Composite deck diaphragms transfer load to buckling-restrained braced frames with collector beams. Frame columns are anchored into foundation. All portions of building appear to be well-connected and detailed for transferring seismic forces.**
C NC N/A U ☒ ☐ ☐ ☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2) **Comments: A 24 inch seismic joint exists at the upper levels of the building, narrowing to a 6 inch gap at Level 2.**
C NC N/A U ☒ ☐ ☐ ☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3) **Comments:**

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

	Description
C NC N/A U ☒ ☐ ☐ ☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1) **Comments: Braces increase in size and number continuing down the building. Shear strength of stories are higher than that of stories above.**
C NC N/A U ☒ ☐ ☐ ☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2) **Comments: Braces increase in size and number continuing down the building. Shear stiffness of stories are higher than that of stories above.**

UC Campus:	San Francisco			Date:	12 February 2020		
Building CAAN:	3034	Auxiliary CAAN:		By Firm:	Simpson Gumpertz & Heger		
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ASCE 41-17
Collapse Prevention Basic Configuration Checklist

C NC N/A U	
☒ C ☐ NC ☐ N/A ☐ U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3) **Comments: Braced frame bays stack vertically. Frame columns are continuous to the foundation**
☒ C ☐ NC ☐ N/A ☐ U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4) **Comments: Frames are similar between adjacent stories.**
☒ 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: Masses of adjacent stories are similar.**
☒ C ☐ NC ☐ N/A ☐ U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6) **Comments: Distance between center of mass and center of rigidity is less than 20% of building width.**

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: Building is supported on piles extending to non-liquefiable soils or bedrock.**
☒ 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: Building is located on essentially flat site. No mapped landslides.**

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

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: Surface fault rupture not anticipated.**

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

FOUNDATION CONFIGURATION

	Description
C NC N/A U ☐ ☒ ☐ ☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3) **Comments: The horizontal dimension of the lateral force resisting system in the transverse direction is relatively short compared to the building height. This concern is alleviated by the use of deep pile foundations.**
C NC N/A U ☒ ☐ ☐ ☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4) **Comments: Pile caps are tied together by substantial grade beams at braced frame locations and building perimeter.**

UC Campus:	**San Francisco**		Date:	**12 February 2020**		
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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type S2-S2A

LOW SEISMICITY

SEISMIC-FORCE-RESISTING SYSTEM

C NC N/A U	Description
C ☒ NC ☐ N/A ☐ U ☐	REDUNDANCY: The number of lines of braced frames in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.3.1.1. Tier 2: Sec. 5.5.1.1) **Comments: Each principal direction has 3 or 4 braced frame lines.**
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: Average axial stress due to gravity loads is approximately 0.10Fy.**
C ☐ NC ☒ N/A ☐ U ☐	BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.4.3.4, is less than $0.50F_y$. (Commentary: Sec. A.3.3.1.2. Tier 2: Sec. 5.5.4.1) **Comments: Axial stress calculated by Quick Check procedure is approximately 1.25Fy,**

CONNECTIONS

C NC N/A U	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: Composite deck diaphragm is connected to braced frame & collector beams.**
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: Frame columns are anchored to the foundation.**

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

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type S2-S2A

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 braced bays in each line is greater than 2. (Commentary: Sec. A.3.3.1.1. Tier 2: Sec. 5.5.1.1) **Comments: Each line has 2 or 3 braced bays.**
C ☒ NC ☐ N/A ☐ U ☐	CONNECTION STRENGTH: All the brace connections develop the buckling capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4) **Comments: Minimal overstresses for bolt shear, plate yielding, plate buckling, and wing plate welding. Additional detailed gusset plate to column/base plate welding checks not performed as part of Tier 1 analysis.**
C ☒ NC ☐ N/A ☐ U ☐	COMPACT MEMBERS: All brace elements meet compact section requirements in accordance with AISC 360, Table B4.1. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec. 5.5.4) **Comments: Braces are buckling-restrained braces.**
C ☒ NC ☐ N/A ☐ U ☐	K-BRACING: The bracing system does not include K-braced bays. (Commentary: Sec. A.3.3.2.1. Tier 2: Sec. 5.5.4.6) **Comments: K-bracing not present.**

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

SEISMIC-FORCE-RESISTING SYSTEM

	Description
C ☒ NC ☐ N/A ☐ U ☐	COLUMN SPLICES: All column splice details located in braced frames develop 50% of the tensile strength of the column. (Commentary: Sec. A.3.3.1.3. Tier 2: Sec. 5.5.4.2) **Comments: Splices are CJP welded.**

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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type S2-S2A

C NC N/A U ☒ ☐ ☐ ☐	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression have *Kl/r* ratios less than 200. (Commentary: Sec. A.3.3.1.4. Tier 2: Sec. 5.5.4.3) **Comments: Braces are buckling-restrained braces.**
C NC N/A U ☒ ☐ ☐ ☐	CONNECTION STRENGTH: All the brace connections develop the yield capacity of the diagonals. (Commentary: Sec. A.3.3.1.5. Tier 2: Sec. 5.5.4.4) **Comments: Minimal overstresses for bolt shear, plate yielding, plate buckling, and wing plate welding. Additional detailed gusset plate to column/base plate welding checks not performed as part of Tier 1 analysis.**
C NC N/A U ☒ ☐ ☐ ☐	COMPACT MEMBERS: All brace elements meet section requirements in accordance with AISC 341, Table D1.1, for moderately ductile members. (Commentary: Sec. A.3.3.1.7. Tier 2: Sec.5.5.4) **Comments: Braces are buckling-restrained braces.**
C NC N/A U ☐ ☒ ☐ ☐	CHEVRON BRACING: Beams in chevron, or V-braced, bays are capable of resisting the vertical load resulting from the simultaneous yielding and buckling of the brace pairs. (Commentary: Sec. A.3.3.2.3. Tier 2: Sec. 5.5.4.6) **Comments: Beams at chevron braces are subject to DCRs up to 1.9 when subjected to the unbalanced vertical force from the bracing pair.**
C NC N/A U ☒ ☐ ☐ ☐	CONCENTRICALLY BRACED FRAME JOINTS: All the diagonal braces frame into the beam–column joints concentrically. (Commentary: Sec. A.3.3.2.4. Tier 2: Sec. 5.5.4.8) **Comments: Brace connections are concentric.**

DIAPHRAGMS (STIFF OR FLEXIBLE)

	Description
C NC N/A U ☐ ☒ ☐ ☐	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 25% of the frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3) **Comments: Diaphragm openings extend half the length on one side of frame at some conditions. The presence of collector beams alleviates some concerns associated with this condition.**

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: Not applicable to this building.**

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

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type S2-S2A

C NC N/A U	
☐ ☐ ☒ ☐ (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: Not applicable to this building.**
☐ ☐ ☒ ☐ (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: Not applicable to this building.**
☐ ☐ ☒ ☐ (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: Not applicable to this building.**
☒ ☐ ☐ ☐ (C)	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-filled metal deck.**

Appendix D

ASCE 41-17 Tier 1 Calculations

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

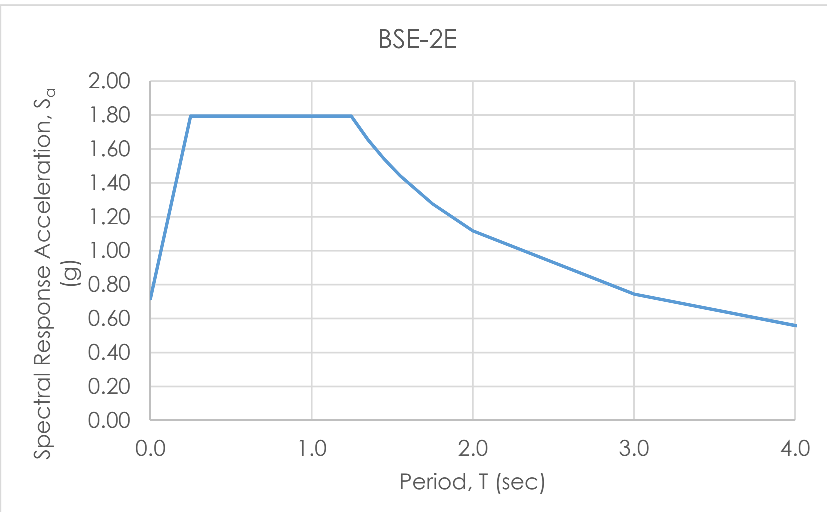
SHEET NO.	
PROJECT NO.	197042.00
DATE	03/16/2020
BY	LZ
CHECKED BY	KDP

CLIENT UCSF
SUBJECT Byers Hall Tier 1 - Quick Checks: BSE-C Hazard

Hazard Level BSE-C (BSE-2E)

MCE_R ground motion (period=0.2s)	S_S	1.380 g
MCE_R ground motion (period=1.0s)	S_1	0.532 g
Site amplification factor at 0.2s	F_a	1.3
Site amplification factor at 1.0s	F_v	4.2
Site modified spectral response (0.2s)	S_{XS}	1.794 g
Site modified spectral response (1.0s)	S_{X1}	2.234 g
Long-period transition period (s)	T_L	12 sec
	T_0	0.249 sec
	T_S	1.245 sec

T	S_a
sec	g
0.0	0.718
0.249	1.794
1.245	1.794
1.25	1.788
1.35	1.655
1.45	1.541
1.55	1.442
1.75	1.277
2.0	1.117
3.0	0.745
4.0	0.559
6.0	0.372
8.0	0.279
10.0	0.223
12.0	0.186



Approximate Period of Structure

System // BRBF

h_n	80.00 ft
β	0.75
C_t	0.020

T	0.535 sec
S_a	1.794 g

SIMPSON GUMPERTZ & HEGER

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SUBJECT Byers Hall Tier 1 - Quick Checks: Seismic Weight

SHEET NO.

PROJECT NO. 197042.00-UCSF

DATE 13 March 2020

BY LZ

CHECKED BY

	Mass Type	Area (sf)	Length (ft)	Load (psf)	Load (plf)	Mass (kips)	ETABS (kips)
Roof	Typ Roof Area (Slab, Beams, SDL)	15,435		113		1736	
h = 15.00'	Mechanical Area (Slab, Beams, SDL)	13,930		213		2960	
	Mechanical Platform	3,500		61		214	
	Stack Shroud Framing					63	
	Stack Shroud Cladding		254		450	114	
	Screen Wall		535		350	187	
	Parapet Above		795		53	42	
	Exterior Wall Below		795		113	89	
	Columns Below					45	
	Braces Below					32	
					Σ =	**5,406**	**5,517**
					ETABS/Calculation =		102%
Level 5	Typical Area (Slab, SDL)	29,365		115		3362	
h = 15.00'	Exterior Wall Above		795		113	89	
	Exterior Wall Below		795		113	89	
	Columns Above					45	
	Columns Below					45	
	Braces Above					32	
	Braces Below					40	
					Σ =	**3,541**	**3,642**
					ETABS/Calculation =		103%
Level 4	Typical Area (Slab, SDL)	29,365		115		3362	
h = 15.00'	Exterior Wall Above		795		113	89	
	Exterior Wall Below		795		113	89	
	Columns Above					45	
	Columns Below					45	
	Braces Above					40	
	Braces Below					49	
					Σ =	**3,541**	**3,656**
					ETABS/Calculation =		103%
Level 3	Typical Area (Slab, SDL)	29,735		115		3405	
h = 15.00'	Exterior Wall Above		795		113	89	
	Exterior Wall Below		795		113	89	
	Columns Above					45	
	Columns Below					83	
	Braces Above					49	
	Braces Below					54	
					Σ =	**3,711**	**3,782**
					ETABS/Calculation =		102%

SIMPSON GUMPERTZ & HEGER

Engineering of Structures and Building Enclosures

SHEET NO.	
PROJECT NO.	197042.00-UCSF
DATE	13 March 2020
BY	LZ
CHECKED BY	

CLIENT UCSF

SUBJECT Byers Hall Tier 1 - Quick Checks: Seismic Weight

	Mass Type	Area (sf)	Length (ft)	Load (psf)	Load (plf)	Mass (kips)	ETABS (kips)
Level 2	Typical Area (Slab, SDL)	28,900		115		3309	
h = 16.00'	Terrace Area (Slab, SDL)	1,380		137		189	
	Parapet Above		31		53	2	
	Exterior Wall Above		795		113	89	
	Exterior Wall Below		730		120	88	
	Columns Above					83	
	Columns Below					118	
	Braces Above					54	
	Braces Below					96	
					Σ =	**3,877**	**3,953**
					ETABS/Calculation =		102%

Base

Calculated Total Σ =	**20,076**	
ETABS Total Σ =		**20,550**
ETABS/Calculation =		102%

Calculated & ETABS Mass Within 5%. O.K.

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

SHEET NO. ______ L______
PROJECT NO. 197042.00
DATE 03/16/2020
BY LZ
CHECKED BY KDP

CLIENT UCSF
SUBJECT Byers Hall Tier 1 - Quick Checks: Pseudo Seismic Force

Floor	W_i [kip]	h_i [ft]	$(h_i)^k$ [ft]	$W_i(h_i)^k$ [kip-ft]	C_{vi}	F_i [kip]	V_i [kip]
Roof	5406	80.0	86.4	466943	0.415	14961	14961
Lvl 5	3541	65.0	69.9	247616	0.220	7934	22895
Lvl 4	3541	50.0	53.5	189602	0.169	6075	28970
Lvl 3	3711	35.0	37.2	138221	0.123	4429	33399
Lvl 2	3877	20.0	21.1	81715	0.073	2618	36017
	20076			1124096	1.00	36017	

T 0.535 sec
k 1.02

W 20076 kip
C 1.0 [Modification factor, buildings 4 stories or greater]
S_a 1.794 g
V 36017 kip

SIMPSON GUMPERTZ & HEGER

Engineering of Structures and Building Enclosures

CLIENT UCSF

SUBJECT Byers Hall Tier 1 - Quick Checks: Column Axial Stress

SHEET NO. L

PROJECT NO. 197042.00

DATE 03/16/2020

BY LZ

CHECKED BY KDP

Column Axial Stress Check due to Gravity

Limiting axial stress per Tier 1 Checklist 0.10Fy = 5 ksi

Floor	P_D [kip]	P_L [kip]	P_{D+L} [kip]	n_{col}	Col Size	A_{col} [in^2]	p_{grav} [ksi]	Check		
PH Floor	3712	1945	5656	89	W14x132	38.8	1.6	≤ 0.10Fy	DCR=0.33	O.K.
Lvl 5	7387	4661	12049	99	W14x132	38.8	3.1	≤ 0.10Fy	DCR=0.63	O.K.
Lvl 4	11036	7378	18414	104	W14x132	38.8	4.6	≤ 0.10Fy	DCR=0.91	O.K.
Lvl 3	14847	10167	25014	104	W14x257	75.6	3.2	≤ 0.10Fy	DCR=0.64	O.K.
Lvl 2	18829	12975	31804	103	W14x257	75.6	4.1	≤ 0.10Fy	DCR=0.82	O.K.

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures

SHEET NO. ______ L
PROJECT NO. 197042.00
DATE 03/16/2020
BY LZ
CHECKED BY KDP

CLIENT UCSF
SUBJECT Byers Hall Tier 1 - Quick Checks: Brace Axial Stress

Brace Axial Stress Check

E-W direction (Y)

Pseudo seismic force base shear	V	=	36017	kip
System modification factor	Ms	=	7	(for collapse prevention)
Limiting axial stress per Tier 1 Checklist	0.50Fy	=	20.5	ksi

Floor	V_j [kip]	L_{br} [ft]	N_{br}	s [ft]	A_{br} [in^2]	f_j [ksi]	Check		
PH Floor	14961	14.7	12	14	6	31.2	> 0.50Fy	DCR=1.52	N.G.
Lvl 5	22895	14.7	12	14	9.7	29.6	> 0.50Fy	DCR=1.44	N.G.
Lvl 4	28970	14.7	12	14	14.7	24.7	> 0.50Fy	DCR=1.20	N.G.
Lvl 3	33399	14.7	12	14	15.7	26.7	> 0.50Fy	DCR=1.30	N.G.
Lvl 2	36017	17.2	12	14	25.3	20.8	> 0.50Fy	DCR=1.01	N.G.

N-S direction (X)

Pseudo seismic force base shear	V	=	36017	kip
System modification factor	Ms	=	7	(for collapse prevention)
Limiting axial stress per Tier 1 Checklist	0.50Fy	=	20.5	ksi

Floor	V_j [kip]	L_{br} [ft]	N_{br}	s [ft]	A_{br} [in^2]	f_j [ksi]	Check		
PH Floor	14961	29.0	8	25	6.0	52.6	> 0.50Fy	DCR=2.57	N.G.
Lvl 5	22895	29.0	8	25	9.8	49.6	> 0.50Fy	DCR=2.42	N.G.
Lvl 4	28970	29.0	8	25	13.0	47.0	> 0.50Fy	DCR=2.29	N.G.
Lvl 3	33399	29.0	8	25	15.8	44.8	> 0.50Fy	DCR=2.18	N.G.
Lvl 2	36017	30.0	9	25	26.0	26.9	> 0.50Fy	DCR=1.31	N.G.

Center of Mass and Center of Rigidity Offset Check

E-W direction (Y)

Floor	Y_{CM}	Y_{CR}	L_Y	$0.2L_Y$	$\lvert Y_{CM}-Y_{CR} \rvert$	Check
Roof	78	90	159	32	11.9	O.K.
Lvl 5	83	90	159	32	6.5	O.K.
Lvl 4	83	88	159	32	5.1	O.K.
Lvl 3	84	86	159	32	2.0	O.K.
Lvl 2	79	81	159	32	2.5	O.K.

N-S direction (X)

Floor	X_{CM}	X_{CR}	L_X	$0.2L_X$	$\lvert X_{CM}-X_{CR} \rvert$	Check
Roof	106	108	216	43	1.3	O.K.
Lvl 5	108	108	216	43	0.1	O.K.
Lvl 4	108	108	216	43	0.1	O.K.
Lvl 3	108	108	216	43	0.0	O.K.
Lvl 2	104	108	216	43	4.1	O.K.

X_{CM}, Y_{CM}: Center of mass location obtained from ETABS model.
X_{CR}, Y_{CR}: Center of rigidity location obtained from ETABS model.
L_X, L_Y: Building plan dimension.

Overturning Check

Overall

LFRS plan dim. @ base	LX =	104	ft				
	LY =	42	ft				
Building Height	H =	80	ft				
Spectral Acceleration	Sa =	1.79	g				
	0.6*Sa =	1.08	g				
Base/Height Ratio	min(LX,LY) / H =			0.53	<	0.6*Sa	N.G.

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CLIENT UCSF
SUBJECT Byers Hall Tier 1 - Quick Checks: BRBF Connections

SHEET NO. L
PROJECT NO. 197042.00-UCSF
DATE 03/16/2020
BY LZhou
CHECKED

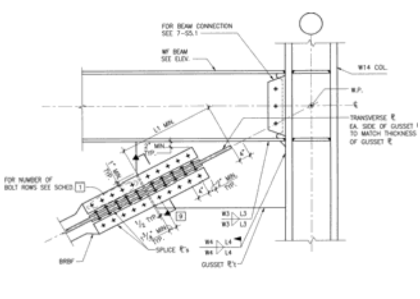
BRBF Brace Connection Checks

	Adjusted Brace Strength						Bolt Shear						
BRB Size, A_{sc}	F_{ymax}	ω	β	$\beta\omega$	T_{max}	P_{max}	$n_{bolts/leg}$	n_{legs}	n_{bolts}	ϕV_{bolt}	ϕV_n	V_u	DCR
[in^2]	[ksi]				[kip]	[kip]				[kips]	[kips]	[kips]	
6	45.1	1.25	1.35	1.69	338	457	2	4	8	56.5	452	457	1.01
9	45.1	1.25	1.35	1.69	507	685	3	4	12	56.5	678	685	1.01
10	45.1	1.25	1.35	1.69	564	761	4	4	16	56.5	904	761	0.84
12	45.1	1.25	1.35	1.69	677	913	5	4	20	56.5	1130	913	0.81
15	45.1	1.25	1.35	1.69	846	1142	4	4	16	88.4	1414	1142	0.81
16	45.1	1.25	1.35	1.69	902	1218	4	4	16	88.4	1414	1218	0.86
18	45.1	1.25	1.35	1.69	1015	1370	5	4	20	88.4	1768	1370	0.77
24	45.1	1.25	1.35	1.69	1353	1827	6	4	24	88.4	2122	1827	0.86
26	45.1	1.25	1.35	1.69	1466	1979	6	4	24	88.4	2122	1979	0.93
28	45.1	1.25	1.35	1.69	1579	2131	7	4	28	88.4	2475	2131	0.86

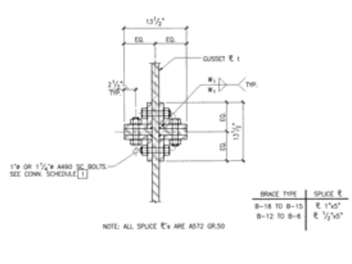
	Gusset Plate Yield							Splice Plate Yield						
BRB Size, A_{sc}	t_{GP}	L	$b_{Whitmore}$	F_{yGP}	ϕT_n	T_u	DCR	t_{SP}	b_{SP}	F_{ySP}	n_{SP}	ϕT_n	T_u	DCR
[in^2]	[in]	[in.]	[in]	[ksi]	[kip]	[kips]		[in]	[in]	[ksi]		[kip]	[kips]	
6	1	7.75	17.4	50	872	457	0.52	0.5	5	50	8	1000	457	0.46
9	1	11.75	22.1	50	1103	685	0.62	0.5	5	50	8	1000	685	0.68
10	1	15.75	26.7	50	1334	761	0.57	0.5	5	50	8	1000	761	0.76
12	1	19.75	31.3	50	1565	913	0.58	0.5	5	50	8	1000	913	0.91
15	1.5	15.75	26.7	50	2001	1142	0.57	1	5	50	8	2000	1142	0.57
16	1.5	15.75	26.7	50	2001	1218	0.61	1	5	50	8	2000	1218	0.61
18	1.5	19.75	31.3	50	2348	1370	0.58	1	5	50	8	2000	1370	0.68
24	1.5	23.75	35.9	50	2694	1827	0.68	1	5	50	8	2000	1827	0.91
26	1.5	23.75	35.9	50	2694	1979	0.73	1	5	50	8	2000	1979	0.99
28	1.5	27.75	40.5	50	3041	2131	0.70	1	5	50	8	2000	2131	1.07

	Wing Plate Welds					
BRB Size, A_{sc}	W1	L1	n_{welds}	ϕV_n	T_u	DCR
[in^2]	[in]	[in]		[kip]	[kips]	
6	0.375	12	4	401	228	0.57
9	0.375	22	4	735	342	0.47
10	0.375	20	4	668	381	0.57
12	0.375	26	4	869	457	0.53
15	0.375	26	4	869	571	0.66
16	0.375	20	4	668	609	0.91
18	0.375	26	4	869	685	0.79
24	0.375	27	4	902	913	1.01
26	0.375	28	4	935	989	1.06
28	0.375	36	4	1203	1065	0.89

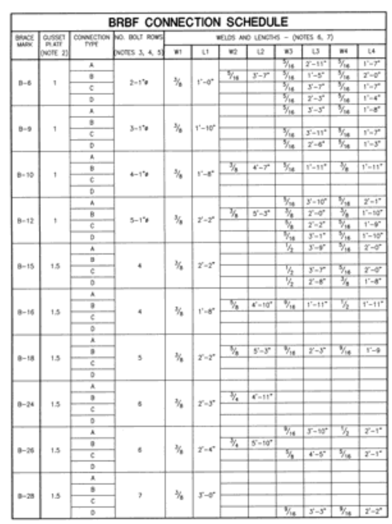
Notes:
1. Gusset plate buckling is ok by inspection.
2. Gusset plate block shear is not applicable.
3. Gusset plate to column/base plate welds not checked for Tier 1 analysis.



TYP. BRACE TO COLUMN FLANGE CONNECTION 1"=1'-0" 3



TYPICAL BRACE CONNECTION CROSS SECTION 1 1/2"=1'-0" 9



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CLIENT UCSF
SUBJECT Byers Hall Tier 1 - Quick Checks: BRBF Chevron Unbalanced Force

SHEET NO.
PROJECT NO. 197042.00-UCSF
DATE 03/16/2020
BY LZhou
CHECKED

Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL 3/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BRBF GEOMETRY:

	Level 2	Level 3	Level 4	Level 5	Roof	
L(ft) =	21.00	21.00	21.00	21.00	21.00	Bay Width (Column C-C)
h_i (ft) =	20.00	15.00	15.00	15.00	15.00	Story Height
L_{diag} (ft)=	22.6	18.3	18.3	18.3	18.3	Work Point - Work Point
cos ψ =	0.465	0.573	0.573	0.573	0.573	ψ = angle between brace and horizontal axis
sin ψ =	0.885	0.819	0.819	0.819	0.819	

BRACE DESIGN

AISC 341-05 Section 16.2a - Brace Strength

	D32	D32	D32	D32	D32	
F_{ysc} =	34 ksi	34 ksi	34 ksi	34 ksi	34 ksi	Minimum yield stress of the steel core
F_{ymax} =	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	45.1 ksi	Max yield stress of the steel core (R_yF_{ysc}; R_y = 1.1, F_{ysc} per dwg)
Dead Load (k) =	34.0	23.2	28.1	18.5	10.1	(from ETABS analysis)
Live Load (k) =	26.8	19.4	24.0	16.5	11.0	(from ETABS analysis)
Seismic Load (k) =	373.1	305.2	304.5	213.8	127.3	Primary direction (from ETABS analysis)
	252.9	201.2	200.2	132.3	70.3	Perpendicular direction (from ETABS analysis)
Combined Axial Load, P_u (k) =	509	407	415	287	168	$(1.2+0.2S_{DS})$D + 0.5 L + ρ E (include 100%+30% effects)
Steel Core Area (sq.in.) =	24.0	16.0	16.0	10.0	6.0	
ϕ P_{ysc} (k) =	734	490	490	306	184	$\phi F_{ysc}A_{sc}$ (AISC 341-05 Equation 16-1)
DCR =	0.69	0.83	0.85	0.94	0.91	$P_u/\phi P_{ysc}$
	Brace OK	*Brace OK*	*Brace OK*	*Brace OK*	*Brace OK*	

AISC 341-05 Section 16.2d - Adjusted Brace Strength

ω=	1.25	1.25	1.25	1.25	1.25	Strain hardening adjustment factor (assumed)
β=	1.35	1.35	1.35	1.35	1.35	Comp.strength adjustment factor (assumed)
$\beta\omega$=	1.69	1.69	1.69	1.69	1.69	
$\omega F_{ymax}A_{sc}$=	1353	902	902	564	338	Adjusted Brace Strength in Tension
$\beta\omega F_{ymax}A_{sc}$=	1827	1218	1218	761	457	Adjusted Brace Strength in Compression

BEAM DESIGN

Beam Demands

	B256	B256	B256	B256	B256	ETABS Beam ID
P_{Emh} (k) =	1123	757	757	473	284	Axial load due to sum of adj brace forces (tension & compression)
$M_{E,drift}$ (k-ft) =	0	0	0	0	0	Drift-induced ETABS seismic moment neglected
P_y (kip) =	419	259	259	162	97	Vertical unbalanced force due to adj. brace strength
M_{Emh} (k-ft) =	2201	1358	1358	849	509	$V_{Em,br}$*L/4
V_{Emh} (k) =	209.6	129.3	129.3	80.8	48.5	Seis. shear due to adj. brace strength
V_{ug} (k) =	0	15	15	15	15	Factored gravity shear (from ETABS Analysis)
V_u (k) =	210	144	144	96	64	V_{ug} + V_{Em}

Beam Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Beam Size =	W24x162	W18x143	W18x119	W18x97	W18x86
A_g (in^2) =	47.8	42.0	35.1	28.5	25.3
t_f (in) =	1.22	1.32	1.06	0.87	0.77
t_w (in) =	0.71	0.73	0.66	0.54	0.48
d (in) =	25.0	19.5	19.0	18.6	18.4
b_f (in) =	13.0	11.2	11.3	11.1	11.1
Z_x (in^3) =	468	322	262	211	186
r_x(in) =	10.40	8.09	7.90	7.82	7.77
r_y(in) =	3.05	2.72	2.69	2.65	2.63
r_{ts}(in) =	3.57	3.17	3.13	3.08	3.05
h_o(in) =	23.80	18.20	17.90	17.70	17.60
J (in^4) =	18.50	19.20	10.60	5.86	4.10

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SUBJECT Byers Hall Tier 1 - Quick Checks: BRBF Chevron Unbalanced Force

SHEET NO.

PROJECT NO. 197042.00-UCSF

DATE 03/16/2020

BY LZhou

CHECKED

Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL 3/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

BEAM DESIGN (CONT'D)

Seismic Compactness Per AISC 341-05 Section16.5a/8.2b

Beam Compact Flange $b_f/2t_f$ =	5.3	4.2	5.3	6.4	7.2	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.2	7.2	7.2	7.2	7.2	
$b_f/2t_f \le (b/2t)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	
Beam Compact Web $(d-2t_f)/t_w$ =	32.0	23.1	25.8	31.5	35.1	
$C_a = P_u/\phi P_y$ =	0.52	0.40	0.48	0.37	0.25	
2.45 $(E/F_y)^{0.5}$ (1 - 0.93C_a) =	30.4	37.0	32.7	38.7	45.3	if $C_a \le 0.125$
0.77 $(E/F_y)^{0.5}$ (2.93 - C_a) =	44.7	46.9	45.4	47.5	49.7	if $C_a > 0.125$
1.49 $(E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	44.7	46.9	45.4	47.5	49.7	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	2151	1890	1580	1283	1139	AISC 360-05 Equation D2-1
DCR =	0.52	0.40	0.48	0.37	0.25	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section E3 - Compression

L_x(ft) =	7.00	7.00	7.00	7.00	7.00	Strong axis unbraced length
L_y(ft) =	7.00	7.00	7.00	7.00	7.00	Weak axis unbraced length
k_x =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_x$ =	8.1	10.4	10.6	10.7	10.8	
k_y =	1.00	1.00	1.00	1.00	1.00	
$(kL/r)_y$ =	27.5	30.9	31.2	31.7	31.9	
F_e (ksi) =	377.35	300.11	293.52	284.86	280.58	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	47.3	46.6	46.6	46.5	46.4	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2035	1763	1471	1192	1057	AISC 360-05 Equation E3-1
DCR =	0.55	0.43	0.51	0.40	0.27	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

AISC 360-05 Section F2 - Flexure

L_p(ft) =	10.8	9.6	9.5	9.4	9.3	AISC 360-05 Equation F2-5
L_r (ft) =	35.8	39.6	34.3	30.4	28.6	AISC 360-05 Equation F2-6
C_b =	1.00	1.00	1.00	1.00	1.00	
S_x (in^3) =	414.0	282.0	231.0	188.0	166.0	
M_p (k-ft) =	1950	1342	1092	879	775	$Z_x F_y$
M_n (k-ft) =	1950	1342	1092	879	775	AISC 360-05 Equation F2-2
$\phi_b M_n$ (k-ft) =	1755	1208	983	791	698	
DCR =	1.25	1.12	1.38	1.07	0.73	
	Revise	Revise	Revise	Revise	Beam OK	

AISC 360-05 Section H1 - Combined Compression & Flexure

P_u (k) =	1123	757	757	473	284	
M_u (k-ft) =	2201	1358	1358	849	509	
$P_u/\phi_c P_{nc}$ =	0.55	0.43	0.51	0.40	0.27	
combined equation =	1.67	1.43	1.74	1.35	0.92	AISC 360-05 Equation H1-1a or H1-1b
	Revise	Revise	Revise	Revise	Beam OK	

AISC 360-05 Section H2 - Combined Tension & Flexure

P_u (k) =	1123	757	757	473	284	
M_u (k-ft) =	2201	1358	1358	849	509	
$P_u/\phi_c P_{nc}$ =	0.52	0.40	0.48	0.37	0.25	
combined equation =	1.64	1.40	1.71	1.32	0.90	AISC 360-05 Equation H1-1a or H1-1b
	Revise	Revise	Revise	Revise	Beam OK	

AISC 360-05 Section G2 - Shear

$\phi_v V_n$ (k) =	429	332	299	244	219	AISC 360-05 Equation G2-1
DCR =	0.49	0.43	0.48	0.39	0.29	
	Beam OK	Beam OK	Beam OK	Beam OK	Beam OK	

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Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL 3/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (RIGHT)

Column Demands

	C65	C65	C65	C65	C65	ETABS Column ID
P_{DL} (k) =	299	249	180	116	55	(from ETABS gravity analysis)
P_{LL} (k) =	238	201	147	96	44	(from ETABS gravity analysis)
$1.2DL + f_1LL + E_v$ =	532	443	322	208	98	$E_V = (0.2)(S_{DS})(DL)$
$0.9DL - E_v$ =	215	179	129	84	40	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	

Brace in Compression - Column in Compression

$V_{c,br}$ (k) =	1078	998	623	374	0	Vert.component of the adj. brace force in compression
$V_{c,br,perp}$ (k) =	539	431	323	216	0	Vert. component of the adj. brace force from perp. frames
$V_{c,br,adj}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	3526	2286	1159	439	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	4058	2730	1481	647	98	

Brace in Tension - Column in Tension

$V_{t,br}$ (k) =	799	739	462	277	0	Vertical component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	399	319	240	160	0	Vert. component of the adj. brace force from perp. frames
$V_{t,br,adj}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	2612	1694	859	325	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	2397	1515	729	241	-40	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x257	W14x257	W14x132	W14x132	W14x132
A_g (in^2) =	75.6	75.6	38.8	38.8	38.8
t_f (in) =	1.89	1.89	1.03	1.03	1.03
t_w (in) =	1.18	1.18	0.65	0.65	0.65
d (in) =	16.4	16.4	14.7	14.7	14.7
b_f (in) =	16	16	14.7	14.7	14.7
Z_x (in^3) =	487	487	234	234	234
Z_y (in^3) =	246	246	113	113	113
r_x (in) =	6.71	6.71	6.28	6.28	6.28
r_y (in) =	4.13	4.13	3.76	3.76	3.76
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	32.0	23.9	25.6	25.7	25.7
$(kL/r)_y$ =	52.1	38.9	42.8	42.9	43.0

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column Compact Flange $b_f/2t_f$ =	4.23	4.23	7.14	7.14	7.14	
$(b/2t)_{max}$=0.30 $(E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	
Column Compact Web $(d-2t_f)/t_w$ =	10.7	10.7	19.6	19.6	19.6	
$C_a = P_u/\phi P_y$ =	1.19	0.80	0.85	0.37	0.06	
$2.45\ (E/F_y)^{0.5}\ (1 - 0.93C_a)$ =	-6.4	15.0	12.5	38.7	55.9	if $C_a \le 0.125$
$0.77\ (E/F_y)^{0.5}\ (2.93 - C_a)$ =	32.2	39.5	38.6	47.5	53.3	if $C_a > 0.125$
$1.49\ (E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	35.9	39.5	38.6	47.5	55.9	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	3402.0	3402.0	1746.0	1746.0	1746.0	AISC 360-05 Equation D2-1
DCR =	0.70	0.45	0.42	0.14	No Tension	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	105.61	189.52	156.11	155.33	154.95	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.0	44.8	43.7	43.7	43.7	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	2790	3046	1527	1526	1525	AISC 360-05 Equation E3-1
DCR =	1.45	0.90	0.97	0.42	0.06	
	Revise	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

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CLIENT UCSF
SUBJECT Byers Hall Tier 1 - Quick Checks: BRBF Chevron Unbalanced Force

SHEET NO.
PROJECT NO. 197042.00-UCSF
DATE 03/16/2020
BY LZhou
CHECKED

Unbalanced Force on Beams at Chevron Frames

BRBF LOCATION : GL 3/D-E

GENERAL DESIGN PARAMETERS :

ϕ_b (flexure) =	0.90	C_d =	5	ρ =	1.0
ϕ_v (shear) =	0.90	I =	1.00	Ω =	2.5
ϕ_c (compression) =	0.90	ϕ_w (weld) =	0.75	S_{DS} =	0.900
ϕ (brace) =	0.90	ϕ_t (tension) =	0.90	E =	29000 ksi

COLUMN DESIGN (LEFT)

Column Demands

	C48	C48	C48	C48	C48	ETABS Column ID
P_{DL} (k) =	379	302	219	148	81	
P_{LL} (k) =	317	256	190	130	72	
$1.2DL + f_1LL + E_v$ =	682	545	398	269	148	$E_V = (0.2)(S_{DS})(DL)$
$0.9DL - E_v$ =	273	218	158	107	58	
Column Orientation =	Strong	Strong	Strong	Strong	Strong	

Brace in Compression - Column in Compression

$V_{c,br}$ (k) =	998	998	623	374	0	Vertical component of the adj. brace force in comp.
$V_{c,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force in comp from perp. frames
$V_{c,br,adj}$ (k) =	-739	-739	-462	-277	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	776	517	259	97	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{uc} = \Sigma P_{em} + P_{u,grav}$ (k) =	1457	1063	656	366	148	

Brace in Tension - Column in Tension

$V_{t,br}$ (k) =	739	739	462	277	0	Vertical component of the adj. brace force in tension
$V_{t,br,perp}$ (k) =	0	0	0	0	0	Vert. component of the adj. brace force in comp from perp. frames
$V_{t,br,adj}$ (k) =	-998	-998	-623	-374	0	Vert. component of the adj. brace force from adjacent frames
$\Sigma P_{em} + \Sigma P_{em,adj} + 0.3{*}\Sigma P_{em,perp}$ (k) =	-776	-517	-259	-97	0	Sum of axial forces in column due to adj. brace force at all levels
$P_{ut} = \Sigma P_{em} - P_{u,grav}$ (k) =	-1049	-735	-417	-204	-58	

Column Geometric Properties

F_y =	50 ksi	50 ksi	50 ksi	50 ksi	50 ksi
Column Size =	W14x342	W14x342	W14x145	W14x145	W14x145
A_g (in^2) =	101.0	101.0	42.7	42.7	42.7
t_f (in) =	2.47	2.47	1.09	1.09	1.09
t_w (in) =	1.54	1.54	0.68	0.68	0.68
d (in) =	17.5	17.5	14.8	14.8	14.8
b_f (in) =	16.4	16.4	15.5	15.5	15.5
Z_x (in^3) =	672	672	260	260	260
Z_y (in^3) =	338	338	133	133	133
r_x (in) =	6.98	6.98	6.33	6.33	6.33
r_y (in) =	4.24	4.24	3.98	3.98	3.98
L (ft) = L_x (ft) = L_y (ft)=	17.9	13.4	13.4	13.5	13.5
k_x =	1.00	1.00	1.00	1.00	1.00
k_y =	1.00	1.00	1.00	1.00	1.00
$(kL/r)_x$ =	30.8	23.0	25.4	25.5	25.5
$(kL/r)_y$ =	50.7	37.9	40.5	40.6	40.6

Seismic Compactness Per AISC 341-05 Section 16.5a/8.2b

Column shall satisfy highly ductile requirements

Column Compact Flange $b_f/2t_f$ =	3.32	3.32	7.11	7.11	7.11	
$(b/2t)_{max}=0.30\ (E/F_y)^{0.5}$=	7.22	7.22	7.22	7.22	7.22	
$b_f/2t_f \le (b/2t)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	
Column Compact Web $(d-2t_f)/t_w$ =	8.2	8.2	18.6	18.6	18.6	
$C_a = P_u/\phi P_y$ =	0.32	0.23	0.34	0.19	0.08	
$2.45\ (E/F_y)^{0.5}\ (1 - 0.93C_a)$ =	41.4	46.2	40.3	48.5	54.8	if $C_a \le 0.125$
$0.77\ (E/F_y)^{0.5}\ (2.93 - C_a)$ =	48.4	50.0	48.0	50.8	52.9	if $C_a > 0.125$
$1.49\ (E/F_y)^{0.5}$=	35.9	35.9	35.9	35.9	35.9	if $C_a > 0.125$ (min. limit)
$(h/t_w)_{max}$=	48.4	50.0	48.0	50.8	54.8	
$(d-2t_f)/t_w \le (h/t_w)_{max}$=	*Column OK*	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section D2 - Tension

$\phi_t P_{nt}$ (k) =	4545.0	4545.0	1921.5	1921.5	1921.5	AISC 360-05 Equation D2-1
DCR =	No Tension	No Tension	No Tension	No Tension	No Tension	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

AISC 360-05 Section E3 - Compression

F_e (ksi) =	111.31	199.75	174.91	174.04	173.61	AISC 360-05 Equation E3-4
F_{cr} (ksi) =	41.4	45.0	44.4	44.3	44.3	AISC 360-05 Equation E3-2 or E3-3
$\phi_c P_{nc}$ (k) =	3766	4093	1705	1704	1703	AISC 360-05 Equation E3-1
DCR =	0.39	0.26	0.38	0.22	0.09	
	Column OK	*Column OK*	*Column OK*	*Column OK*	*Column OK*	

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Appendix E

Comparison of IBC 2006/CBC 2007 & ASCE 41-17 Seismic Hazards

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CLIENT UCSF

SUBJECT Byers Hall: Comparison of Seismic Hazards

SHEET NO.

PROJECT NO. 197042.00-UCSF

DATE 18 March 2020

BY LZhou

CHECKED

		SPL-III			SPL-IV		SPL-V	
		CBC 2007 Benchmark	ASCE 41-17					
			BSE-1N	BSE-2N	BSE-R	BSE-C	2/3 BSE-R	2/3 BSE-C
MCE_R Spectral Resp. (T=0.2s)	S_S =	1.500	-	1.5	0.741	1.378	-	-
MCE_R Spectral Resp. (T=1.0s)	S_1 =	0.634	-	0.6	0.266	0.532	-	-
Site Amp. Factor (T=0.2s)	F_a =	0.9	-	1.3	1.315	1.3	-	-
Site Amp. Factor (T=1.0s)	F_v =	2.4	-	4.2	4.2	4.2	-	-
Site Mod. Spectral Resp. (T=0.2s)	S_{XS} =	1.350	1.300	1.950	0.974	1.791	0.650	1.194
Site Mod. Spectral Resp. (T=1.0s)	S_{X1} =	1.522	1.680	2.520	1.117	2.234	0.745	1.490
Design Spectral Resp. (T=0.2s)	S_{DS} =	0.900	-	-	-	-	-	-
Design Spectral Resp. (T=1.0s)	S_{D1} =	1.014	-	-	-	-	-	-

Seismic Mass	W =	20550	20550	20550	20550	20550	20550	20550

X-Direction

Bldg Period	T =	0.895	0.895	0.895	0.895	0.895	0.895	0.895
Spectral Response Acceleration	S_a =	0.900	1.300	1.950	0.974	1.791	0.650	1.194
Redundancy Factor	ρ =	1.0	-	-	-	-	-	-
Modification Factors	C_1C_2 =	-	1.1	1.2	1.1	1.2	1.1	1.2
Effective Mass Mod. Factor	C_m =	-	1.0	1.0	1.0	1.0	1.0	1.0
Importance Factor	I =	1	-	-	-	-	-	-
Response Modification Coeff.	R =	8	-	-	-	-	-	-
Tier 1 System Modification Factor	Ms =	-	4.5	7.0	4.5	7.0	4.5	7.0
Tier 2/3 Component Demand Mod. Factor	m =	-	5.6	7.5	5.6	7.5	5.6	7.5
Base Shear (Tier 1 Pseudo)	V_{Psuedo} =	-	26715	40073	20024	36813	13349	24542
Base Shear (CBC or Tier 2/3 ELF)	V_{ELF} =	2312	29387	48087	22027	44176	14684	29451
Base Shear (CBC RSA)	V_{DYN} =	1965	-	-	-	-	-	-
Base Shear (Tier 2/3 RSA)	$C_1C_2V_{DYN}$ =	-	20376	33342	15272	30630	10182	20420
Modified Base Shear (CBC RSA)	ρV_{DYN} =	1965	-	-	-	-	-	-
Modified Base Shear (Tier 1 Pseudo)	V_{Psuedo}/M_s =	-	5937	5725	4450	5259	2967	3506
Modified Base Shear (Tier 2/3 RSA)	$C_1C_2V_{DYN}/m$ =		3638	4446	2727	4084	1818	2723
Max. Brace Demand-Capacity Ratio (CBC)	DCR =	0.90	-	-	-	-	-	-
Est. Brace Acceptance Criteria Ratio (Tier 2/3)	ACR =	-	1.244	1.520	0.932	1.396	0.621	0.931

$ACR_{41-17} = DCR_{CBC07} * (C_1C_2V_{DYN}/m)_{41-17} / (\rho V_{DYN})_{CBC07} * (\phi F_{yLB} / F_y)$

Y-Direction

Bldg Period	T =	0.806	0.806	0.806	0.806	0.806	0.806	0.806
Spectral Response Acceleration	S_a =	0.900	1.300	1.950	0.974	1.791	0.650	1.194
Redundancy Factor	ρ =	1.0	-	-	-	-	-	-
Modification Factors	C_1C_2 =	-	1.1	1.2	1.1	1.2	1.1	1.2
Effective Mass Mod. Factor	C_m =	-	1.0	1.0	1.0	1.0	1.0	1.0
Importance Factor	I =	1	-	-	-	-	-	-
Response Modification Coeff.	R =	8	-	-	-	-	-	-
Tier 1 System Modification Factor	Ms =	-	4.5	7.0	4.5	7.0	4.5	7.0
Tier 2/3 Component Demand Mod. Factor	m =	-	5.6	7.5	5.6	7.5	5.6	7.5
Base Shear (Tier 1 Pseudo)	V_{Psuedo} =	-	26715	40073	20024	36813	13349	24542
Base Shear (CBC or Tier 2/3 ELF)	V_{ELF} =	2312	29387	48087	22027	44176	14684	29451
Base Shear (CBC RSA)	V_{DYN} =	1965	-	-	-	-	-	-
Base Shear (Tier 2/3 RSA)	$C_1C_2V_{DYN}$ =	-	22888	37454	17156	34408	11437	22938
Modified Base Shear (CBC RSA)	ρV_{DYN} =	1965	-	-	-	-	-	-
Modified Base Shear (Tier 1 Pseudo)	V_{Psuedo}/M_s =	-	5937	5725	4450	5259	2967	3506
Modified Base Shear (Tier 2/3 RSA)	V_{DYN}/m =	-	4087	4994	3064	4588	2042	3058
Max. Brace Demand-Capacity Ratio	DCR =	0.89	-	-	-	-	-	-
Estimated Brace Acceptance Criteria Ratio	ACR =	-	1.382	1.688	1.036	1.551	0.690	1.034

$ACR_{41-17} = DCR_{CBC07} * (C_1C_2V_{DYN}/m)_{41-17} / (\rho V_{DYN})_{CBC07} * (\phi F_{yLB} / F_y)$

Under CBC 2007 loading, when neglecting gravity loads, all braces pass strength checks. The maximum DCR is 0.90.

When performing an ASCE 41-17 Tier 1, the ASCE 41-17 Tier 1 BSE-C modified base shear (V/Ms) is 168% higher than the CBC 2007 base shear. Since Tier 1 quick checks conservatively allow only 50% of the brace capacity to be used, the braces typically fail the Tier 1 stress check.

When performing an ASCE 41-17 Tier 1, the ASCE 41-17 Tier 2/3 BSE-C modified base shear (V/m) is 108% to 133% higher than the CBC 2007 base shear. An estimated ASCE 41-17 Tier 2/3 brace stress can be determined by scaling up the CBC 2007 demands by the ratio of the two base shears $(C_1C_2V_{DYN}/m)_{41-17}$ / $(\rho V_{DYN})_{CBC07}$. Then by scaling the resulting DCR down to use the specified average yield strength (41 ksi) in lieu of the lower bound yield strength (ϕ*34ksi), an acceptance criteria ratio per ASCE 41-17 Sec 7.5.2.2.1 can be determined. This results in a maximum estimated ACR of 1.551, with the 65 (out of 110) braces failing.