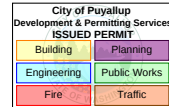


Calculations required to be provided by
the Permittee on site for all Inspections



PRCTI20250117

**City of Puyallup
Building
REVIEWED
FOR
COMPLIANCE**

BSnowden

02/21/2025

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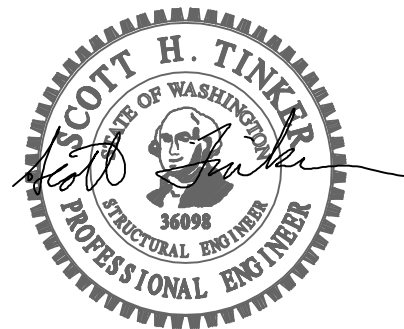
December 20, 2024

STRUCTURAL CALCULATIONS
(Permit Submittal)

**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

1015 39th Avenue SE
Puyallup, WA 98374

Quantum Job Number: 19305.04



Prepared for:
THE BENAROYA COMPANY | CENTERIS DATA CENTERS
18300 Cascade Avenue S, STE 200
Seattle, WA 98188

Prepared by:
QUANTUM CONSULTING ENGINEERS
1511 Third Avenue, Suite 323
Seattle, WA 98101
TEL 206.957.3900
FAX 206.957.3901



QUANTUM | CONSULTING ENGINEERS

**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

1015 39th AVENUE SE
PUYALLUP, WA 98374

QUANTUM JOB NUMBER: 19305.04

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QUANTUM | CONSULTING ENGINEERS

**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

1015 39th AVENUE SE
PUYALLUP, WA 98374

QUANTUM JOB NUMBER: 19305.04

DESIGN CRITERIA

Structural Design Criteria

Building Code: 2021 International Building Code
Building Department: City of Puyallup

Seismic Criteria

S_s: 1.26 I_e: 1.50
S₁: 0.43 Seismic Soil Site Class: C
S_{ds}: 1.01 Seismic Design Category: D
S_{d1}: 0.43 Cs: 0.13
R: 8.00 Steel Buckling Restrained Braced Frames

Wind Criteria

Wind Speed: 108 MPH
Risk Category: IV
Wind Exposure: B
Kzt: 1

Geotechnical Criteria

Allowable Bearing Pressure 5000 PSF
Frost Depth 18" min.
Soils Consultant Geoengineers, Inc.
Soils Report Number File No 4565-064-09
Soils Report Date October 14, 2024
Active Soil Pressure (Restrained/Unrestrained) 55 PCF / 35 PCF
Seismic Surcharge Pressure 8H PSF
Passive Soil Pressure 300 PCF
Coefficient of Friction 0.4

Materials Criteria

Concrete (28 Day Strength):

Foundation/Slab on Grade F'c= 3,000 PSI
Plinths / Jackets F'c= 5,000 PSI

Reinforcing Steel:

Grade 60 Fy= 60,000 PSI

Structural Steel:

Wide-Flange Sections: A-992 Fy= 50,000 PSI
Miscellaneous Sections: A-36 Fy= 36,000 PSI
Tube Sections: A-500 Fy= 46,000 PSI
Pipe Sections: A-53 Fy= 35,000 PSI
Welding Fy= 70,000 PSI

Light Gauge Steel:

43 AND 33 MIL.: ASTM A-653 SS GRADE 33 Fy= 33,000 PSI
118, 97, 68 AND 54 MIL.: ASTM A-653 SS GRADE 50 Fy= 50,000 PSI
WELDING Fy= 60,000 PSI



Quantum Consulting Engineers LLC
1511 Third Avenue, Suite 323
Seattle, WA 98101

Project: 1015 Building IV Upgrade
Client: Benaroya

Date: 12/6/24 Job No: 19305.04
Designer: TVM Sheet: 1
Checked By:

Building Loads

Snow Load	Roof	25 psf
Live Load	Exit Facilities	100 psf
	Office	100 psf

Assembly Loads

Roof	Gravity:	Seismic:	Comments
Roofing	2.0 psf	2.0 psf	
1 1/2" B Deck	2.3 psf	2.3 psf	
Rigid Insulation	4.0 psf	4.0 psf	
Steel Framing	5.0 psf	5.0 psf	
Mech./Elec.	3.0 psf	2.5 psf	
Acoustic Panels	1.0 psf	1.0 psf	
Sprinklers	3.0 psf	2.0 psf	
Partitions		5.0 psf	
Misc.	2.0 psf	2.5 psf	
Total:	22.3 psf	26.3 psf	SL=25 psf

Roof RAM Input	Gravity:	Seismic:	Comments
Roofing	2.0 psf	2.0 psf	
Rigid Insulation	4.0 psf	4.0 psf	
Roof Joists	2.0 psf	2.0 psf	
Mech./Elec.	4.0 psf	3.5 psf	
Acoustic Panels	1.0 psf	1.0 psf	
Sprinklers	3.0 psf	2.0 psf	
Partitions		5.0 psf	
Misc.	2.0 psf	2.5 psf	
Total:	18.0 psf	22.0 psf	SL=25 psf

Non Composite Joist Floors	Gravity:	Seismic:	Comments
Flooring	2.5 psf	2.5 psf	
Conc. + Shallow Verco	41.0 psf	41.0 psf	
Floor Joists	3.5 psf	3.5 psf	
Steel Framing	4.5 psf	4.5 psf	
Mech./Elec.	1.0 psf	1.0 psf	
Acoustic Panels	1.0 psf	1.0 psf	
Sprinklers	3.0 psf	2.0 psf	
Misc.	1.5 psf	1.5 psf	
Partitions		10.0 psf	
Total:	58.0 psf	67.0 psf	LL=100 psf

Non Composite Joist Floors RAM	Gravity:	Seismic:	Comments
Flooring	2.5 psf	2.5 psf	
Floor Joists	3.5 psf	3.5 psf	
Mech./Elec.	1.0 psf	1.0 psf	
Acoustic Panels	1.0 psf	1.0 psf	
Sprinklers	3.0 psf	2.0 psf	
Misc.	1.0 psf	2.0 psf	
Partitions		10.0 psf	
Total:	12.0 psf	22.0 psf	LL=100 psf

Composite Beam Floors	Gravity:	Seismic:	Comments
Flooring	2.5 psf	2.5 psf	
3 1/2" Over 3" Deck	62.5 psf	62.5 psf	
Steel Framing	14.0 psf	14.0 psf	
Mech./Elec.	1.0 psf	1.0 psf	
Acoustic Panels	1.0 psf	1.0 psf	
Sprinklers	3.0 psf	2.0 psf	
Misc.	1.5 psf	1.5 psf	
Partitions		10.0 psf	
Total:	85.5 psf	94.5 psf	LL=100 psf

Composite Beam Floors RAM Input	Gravity:	Seismic:	Comments
Flooring	2.5 psf	2.5 psf	
Mech./Elec.	1.0 psf	1.0 psf	
Acoustic Panels	1.0 psf	1.0 psf	
Sprinklers	3.0 psf	2.0 psf	
Misc.	2.5 psf	1.5 psf	
Partitions		10.0 psf	
Total:	10.0 psf	18.0 psf	LL=100 psf

Deflection Criteria

Roof	Walls	Floor
Live Load: L/240	L/120	Live Load: L/360
Total Load: L/180	L/240	Total Load: L/240



Quantum Consulting Engineers LLC
1511 Third Avenue, Suite 323
Seattle, WA 98101

Project: 1015 Building IV Upgrade

Date: 12/6/24 Job No: 19305.04

Client: Benaroya

Designer: TVM Sheet: 1

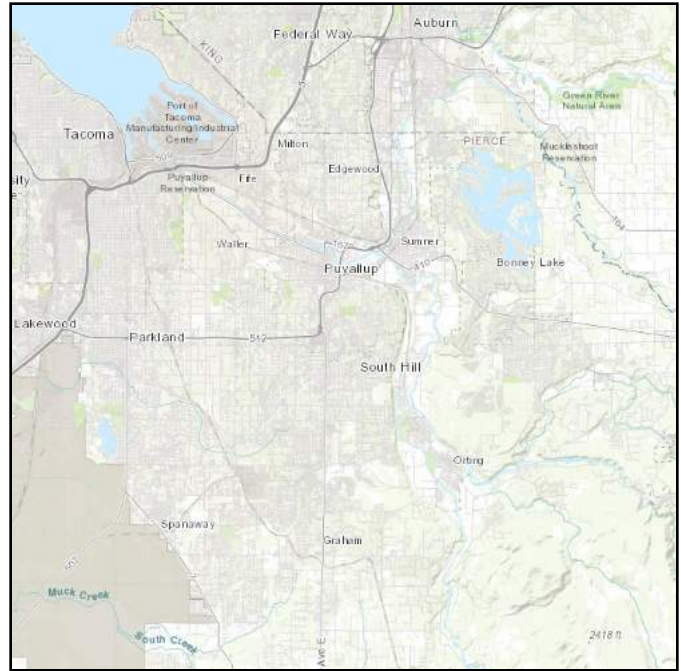
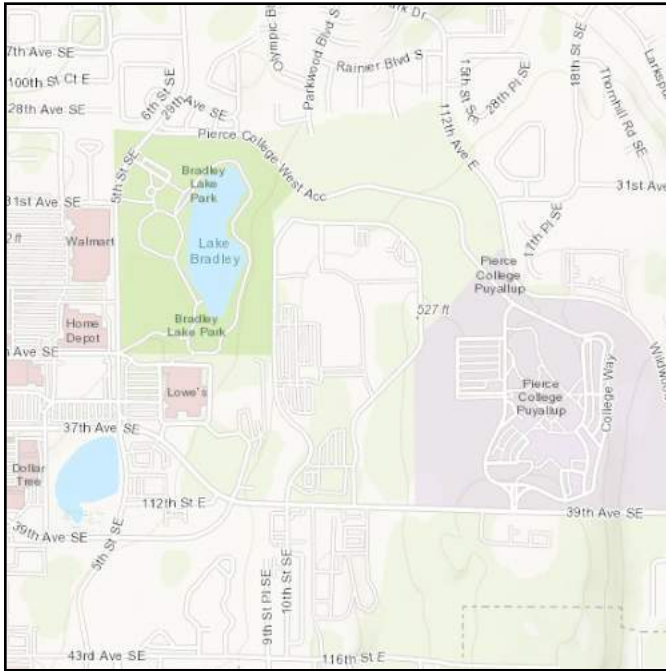
Checked By:

ASCE Hazards Report

Address:
1015 39th Ave SE
Puyallup, Washington
98374

Standard: ASCE/SEI 7-16
Risk Category: IV
Soil Class: C - Very Dense
Soil and Soft Rock

Latitude: 47.158856
Longitude: -122.279864
Elevation: 487.9235751673597 ft
(NAVD 88)



Wind

Results:

Wind Speed	108 Vmph
10-year MRI	67 Vmph
25-year MRI	73 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph

Data Source: ASCE/SEI 7-16, Fig. 26.5-1D and Figs. CC.2-1–CC.2-4, and Section 26.5.2

Date Accessed: Tue Oct 15 2024

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 1.6% probability of exceedance in 50 years (annual exceedance probability = 0.00033, MRI = 3,000 years).

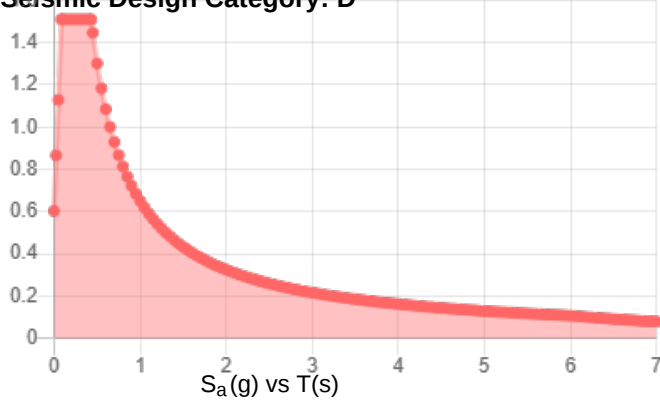
Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Site Soil Class: C - Very Dense Soil and Soft Rock

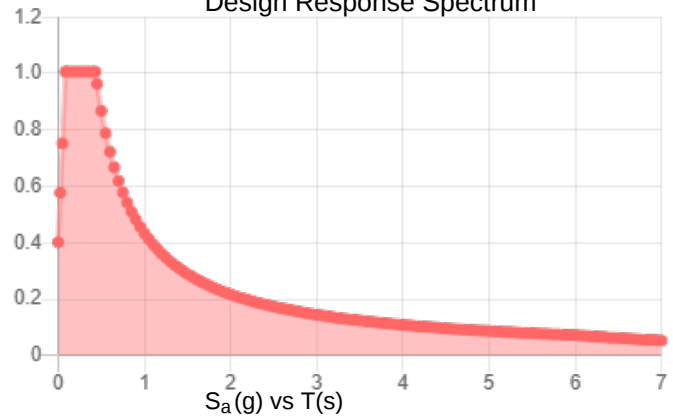
Results:

S_S :	1.257	S_{D1} :	0.433
S_1 :	0.433	T_L :	6
F_a :	1.2	PGA :	0.5
F_v :	1.5	PGA _M :	0.6
S_{MS} :	1.508	F_{PGA} :	1.2
S_{M1} :	0.65	I_e :	1.5
S_{DS} :	1.006	C_v :	1.151

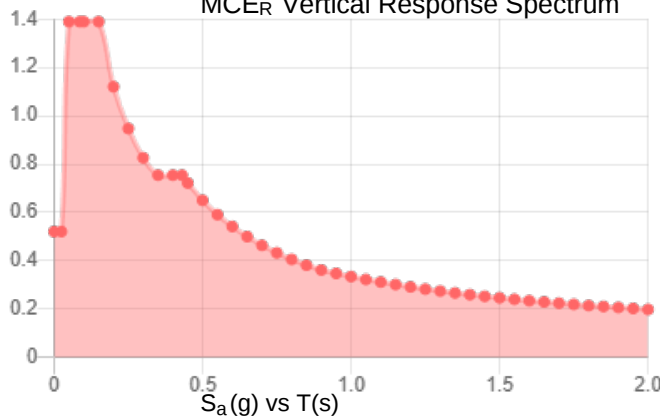
Seismic Design Category: D
MCE_R Response Spectrum



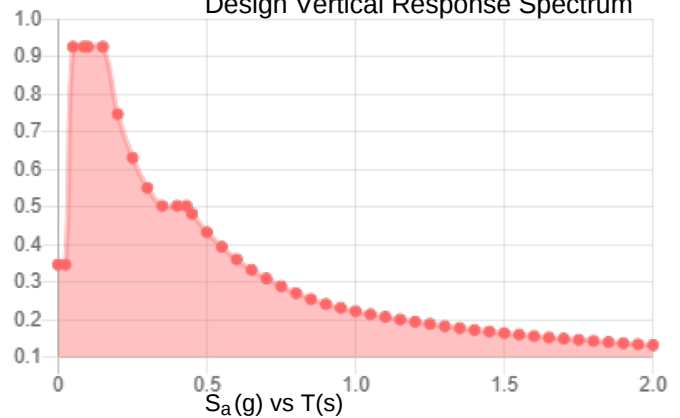
Design Response Spectrum



MCE_R Vertical Response Spectrum



Design Vertical Response Spectrum



Data Accessed: Tue Oct 15 2024

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

Results:

Mapped Elevation:

Data Source:

Date Accessed: Tue Oct 15 2024

In "Case Study" areas, site-specific case studies are required to establish ground snow loads. Extreme local variations in ground snow loads in these areas preclude mapping at this scale.

Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2 percent annual probability of being exceeded (50-year mean recurrence interval).

Statutory requirements of the Authority Having Jurisdiction are not included. Site is outside ASCE/SEI 7-16, Table 7.2-5 boundaries. For ground snow loads in this area, see SEAW Snow Load Analysis for Washington, 2nd Ed. (1995). [Structural Engineers Association of Washington, Seattle, WA](#). Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported 'elevation' and 'mapped elevation' differ significantly from each other.

Seismic Base Shear for the Equivalent Lateral Force Procedure

Per IBC 2018 & ASCE 7-16

Structure: **South Hill South**
Address: **1015 39th Ave SE, Puyallup, WA**
Latitude: **47.1590** Longitude: **-122.2800**

Structure Classification

Risk Category: **IV** per ASCE Table 1.5-1

Seismic Force-Resisting System: **Steel Buckling Restrained Braced Frames**

R: **8** per ASCE Table 12.2-1
W_o: **2 1/2** per ASCE Table 12.2-1
C_d: **5** per ASCE Table 12.2-1
h_n (ft): **37.67** height above the base to the highest level of the structure

Site Ground Motion

Reg. Structure/5 Stories Max: **Yes** Per ASCE 12.8.1.3
S₁ (g-sec): **0.43** S_s (g-sec): **1.26**
Site Class: **C** **Assumed Value** per ASCE 11.4.3

F_v **1.50**

F_a **1.20**

1.2 Min Value where SC D Assumed

S_{M1} (g-sec): **0.65**

S_{MS} (g-sec): **1.51**

per ASCE 11.4.4

S_{D1} (g-sec): **0.43**

S_{DS} (g-sec): **1.01**

per ASCE 11.4.5

SDC: **D** per ASCE 11.6

I_E: **1.50** per ASCE Table 1.5-2

Fundamental Period per ASCE 12.8.2

Period Method: **Properly Substantiated Analysis**
Structure Type: **Steel BRB Frame**
T_L (sec): **6.00** ASCE Figures 22-14 through 22-17
T_S: 0.43
Period (sec.): **0.65** Properly Substantiated Analysis per ASCE 12.8.2
T_a (sec): 0.46 C_t * h_{nx} per ASCE Eq. 12.8-7
T (sec.): 0.64 = T_a * C_u: Period Upper Limit
T_{use} (sec): **0.65** T <= T_L

Equivalent Lateral Force Procedure Design Base Shear per ASCE 12.8

C_s: 0.19 = S_{DS} / (R/I_E) per ASCE Eq. 12.8-2
C_{s-max}: 0.13 = S_{D1} / (T_a * R/I_E) for T <= T_L per ASCE Eq. 12.8-3
C_{s-max}: -- = S_{D1} * T_L / (T_a² * R/I_E) for T > T_L per ASCE Eq. 12.8-4
C_{s-min}: 0.07 per ASCE Eq. 12.8-5
C_{s-min}: -- = 0.5S₁ / (R/I_E) for S₁ => 0.6g per ASCE Eq. 12.8-6
C_{s-use}: 0.13

V : 0.126 W = C_{s-use} * W per ASCE Eq. 12.8-1



Quantum Consulting Engineers LLC
1511 Third Avenue, Suite 323
Seattle, WA 98101

Project: **1015 Buidling**
Client: **Benaroya**

Date: 12/6/24
Designer: **TVM**
Checked By:

Job No: **19305.04**
Sheet: **1**

Vert. Distribution of Seismic Forces for the Equiv. Lateral Force Procedure

Per IBC 2018 & ASCE 7-16

Structure: **South Hill South**

Seismic Parameters

I_E : 1.50 per ASCE Table 1.5-2
 S_{DS} (g-sec): 1.01 per ASCE 11.4.4
 Period (Sec): 0.46 per ASCE 12.8.2.1
 k : 1.00 per ASCE 12.8.3

Vertical Distribution of Seismic Forces per ASCE 12.8.3

$F_x = C_{vx}V$ per ASCE Eq. 12.8-11

$C_{vx} = (w_x h_x^k) / (S_w h_i^k)$ per ASCE Eq. 12.8-12

Level	h_x (ft)	w_x (k)	% of W_{total}	$w_x * h_x^k$	C_{vx} (%)	F_x (k)	V_x (k)
Roof	38.16	3160.00	31.0%	120585.6	45.2%	579.12	579.12
L2	20.83	7019.91	69.0%	146224.7	54.8%	702.25	1281.37

Total WT (k): 10179.91 Sum: 266810

C_{s-use} : 0.126

V (k): 1281.37 per ASCE 12.8.1

Vertical Distribution of Seismic Diaphragm Forces per ASCE 12.10.1.1

$F_{px} = (S_F / S_w I_i) * w_{px}$ per ASCE Eq 12.10-1

$F_{px-max} = 0.4 * S_{DS} * I_E * w_{px}$ per per ASCE 12.10.1.1

$F_{px-min} = 0.2 * S_{DS} * I_E * w_{px}$ per per ASCE 12.10.1.1

Level	w_{px} (k)	Σw_i (k)	F_x (k)	ΣF_i (k)	F_{px} (k)	F_{pxMIN} (k)	F_{pxMax} (k)
Roof	3160.00	3160.00	579.12	579.12	579.12	953.31	1906.618
L2	7019.91	10179.91	702.25	1281.37	883.61	2117.77	4235.533

Notes

Fp-min Controls

Fp-min Controls

Collector Forces per ASCE 12.10.1.2

Level	F_x (k) 12.8.3	ΩF_x (k)	F_{px} (k) 12.10-1	ΩF_{px} (k)	F_{pxMIN} (k)	$1.25 F_{pxMIN}$ (k)
Roof	579.12	1447.79	579.12	1447.79	953.31	1191.64
L2	702.25	1755.62	883.61	2209.03	2117.77	2647.21



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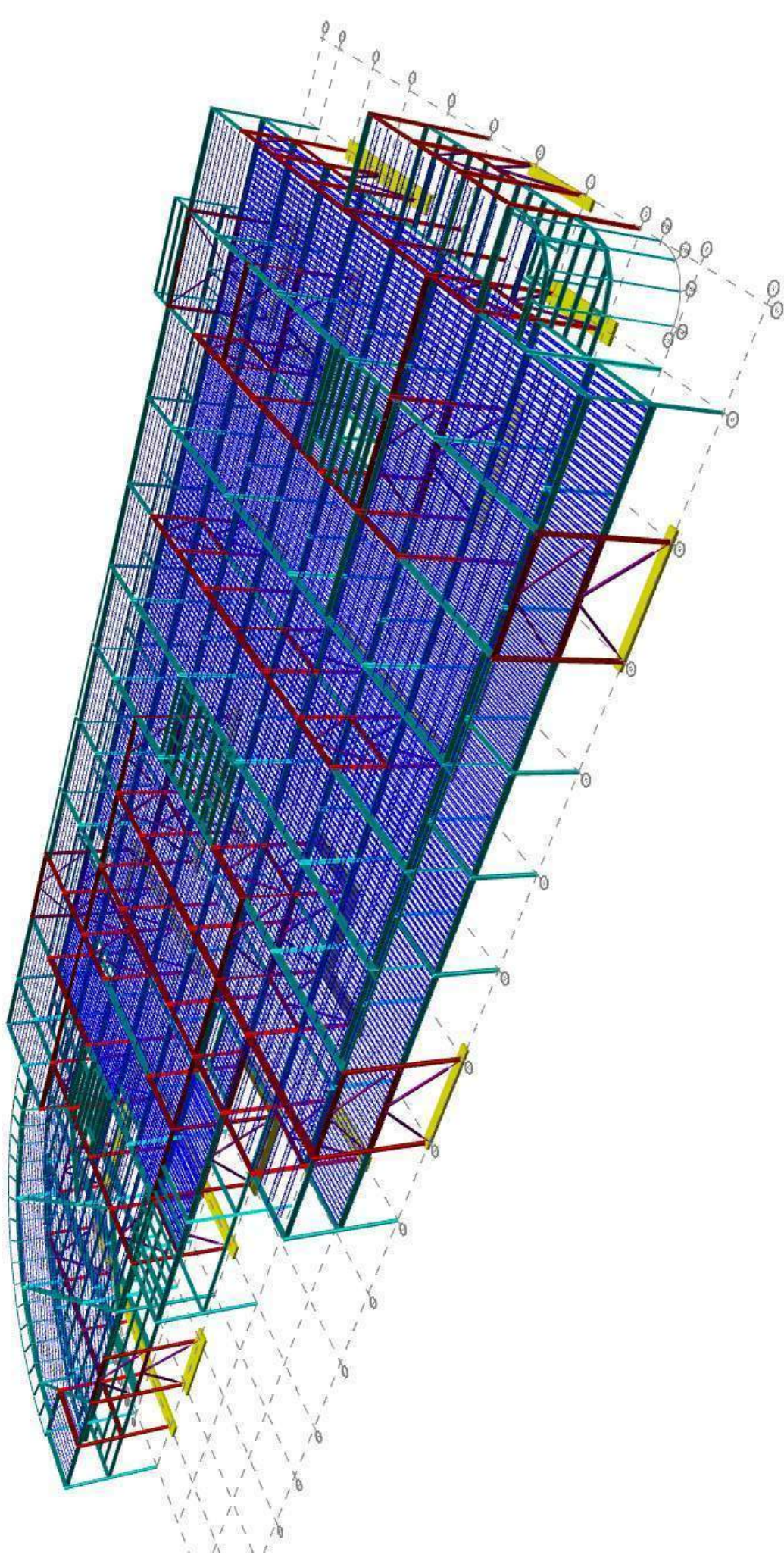
**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

1015 39th AVENUE SE
PUYALLUP, WA 98374

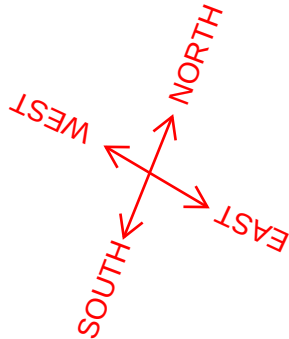
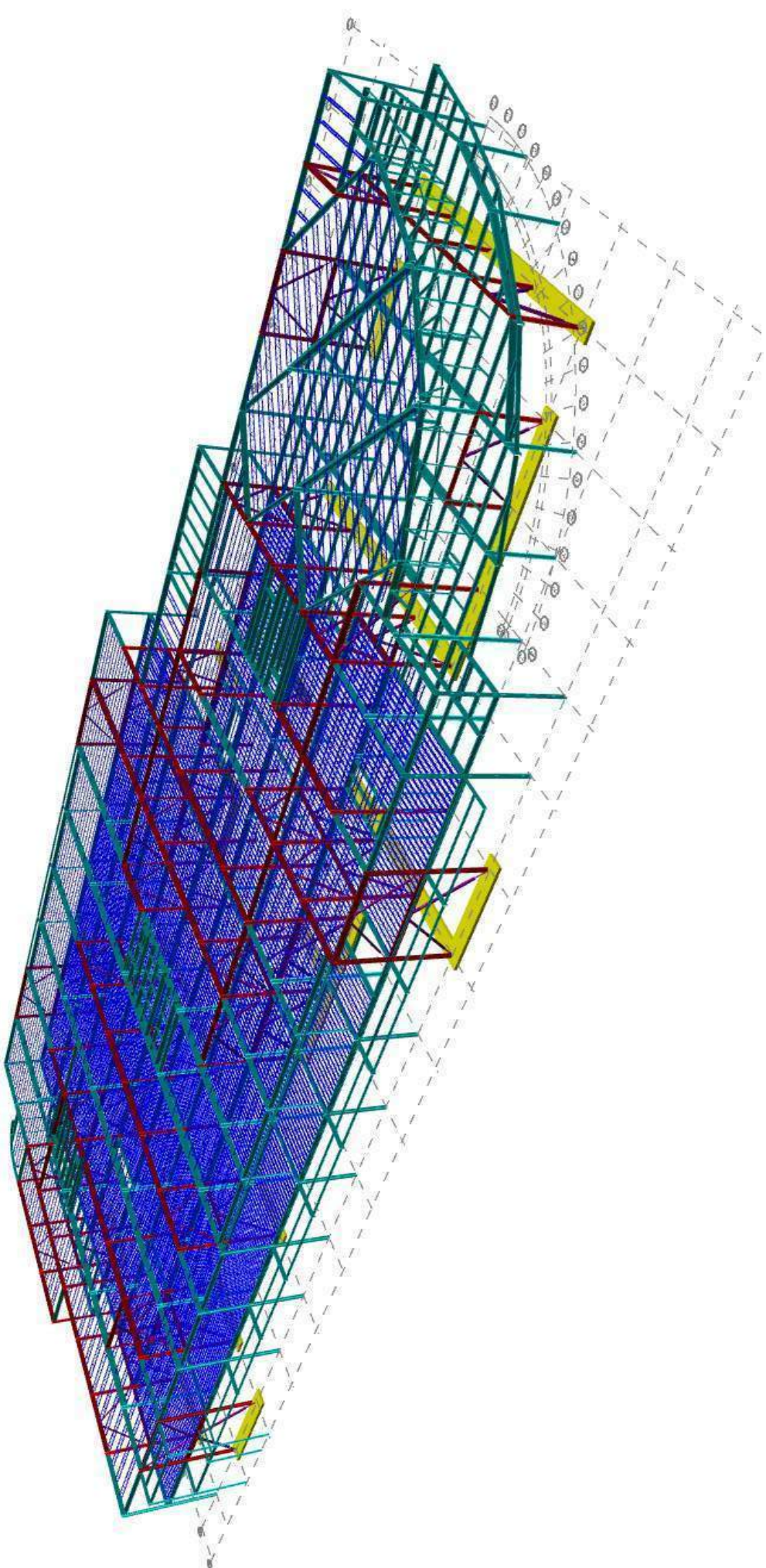
QUANTUM JOB NUMBER: 19305.04

RAM BUILDING ANALYSIS

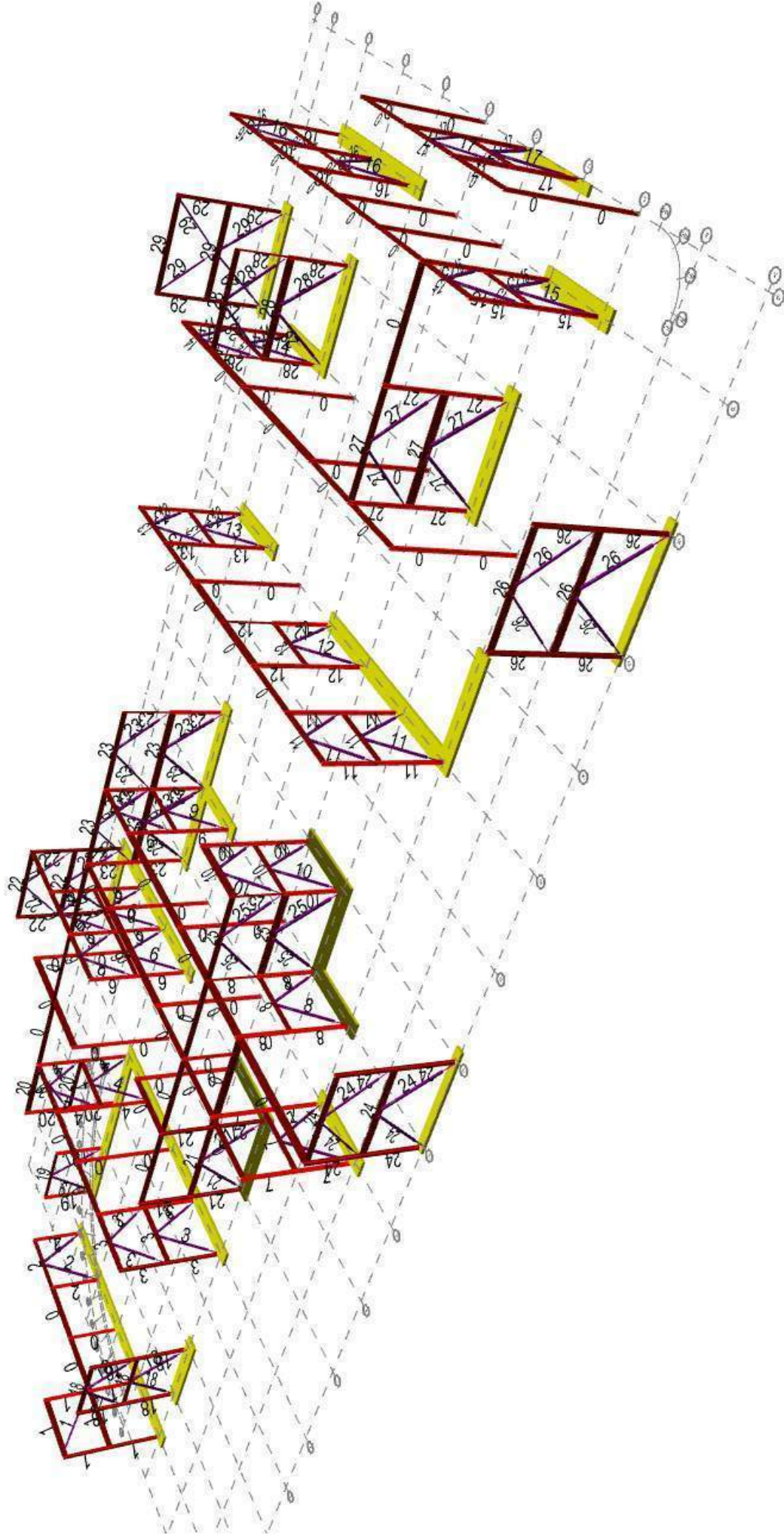
MODEL VIEWS AND FLOOR MAPS



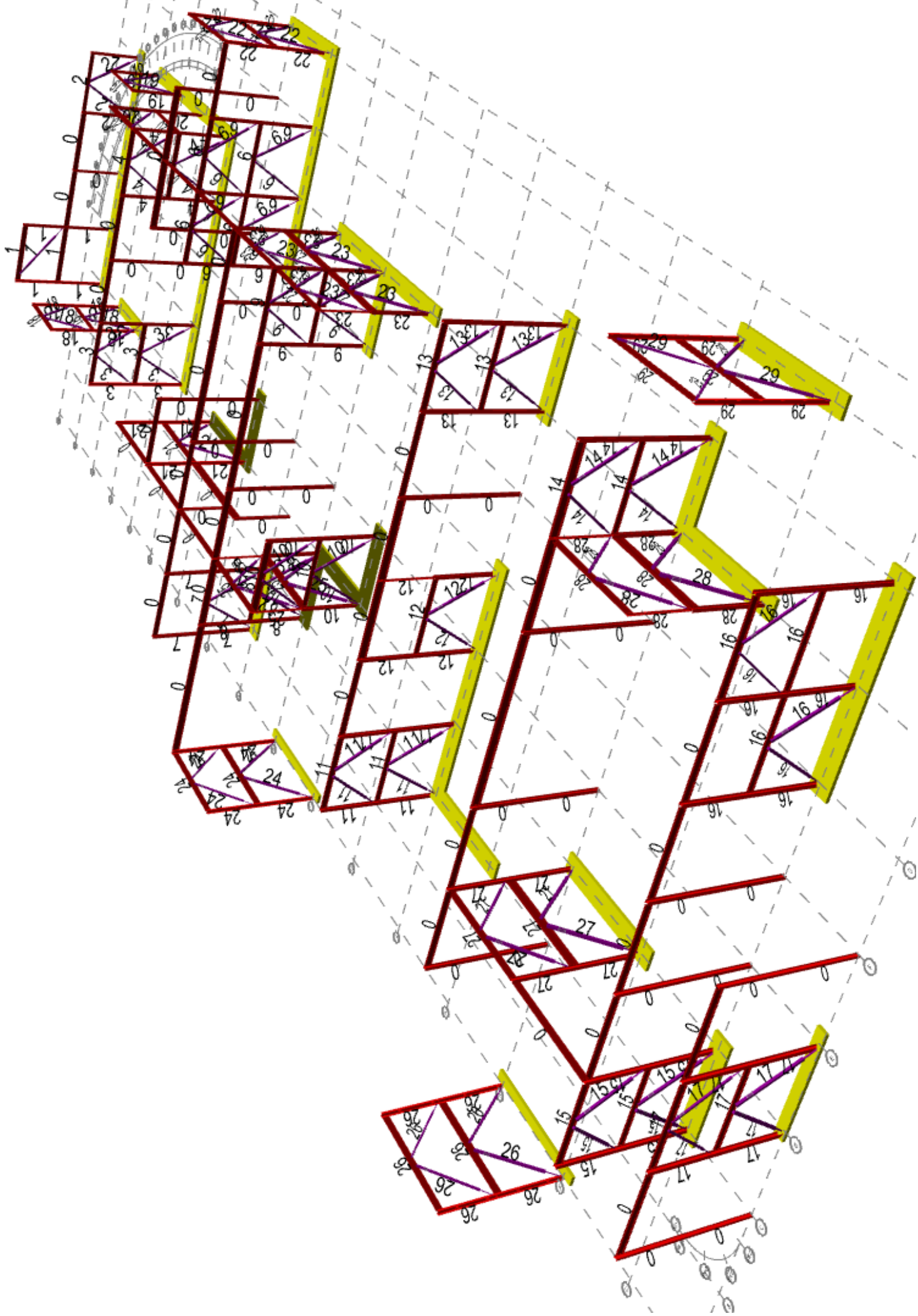
South Hill 1015 Building - West View



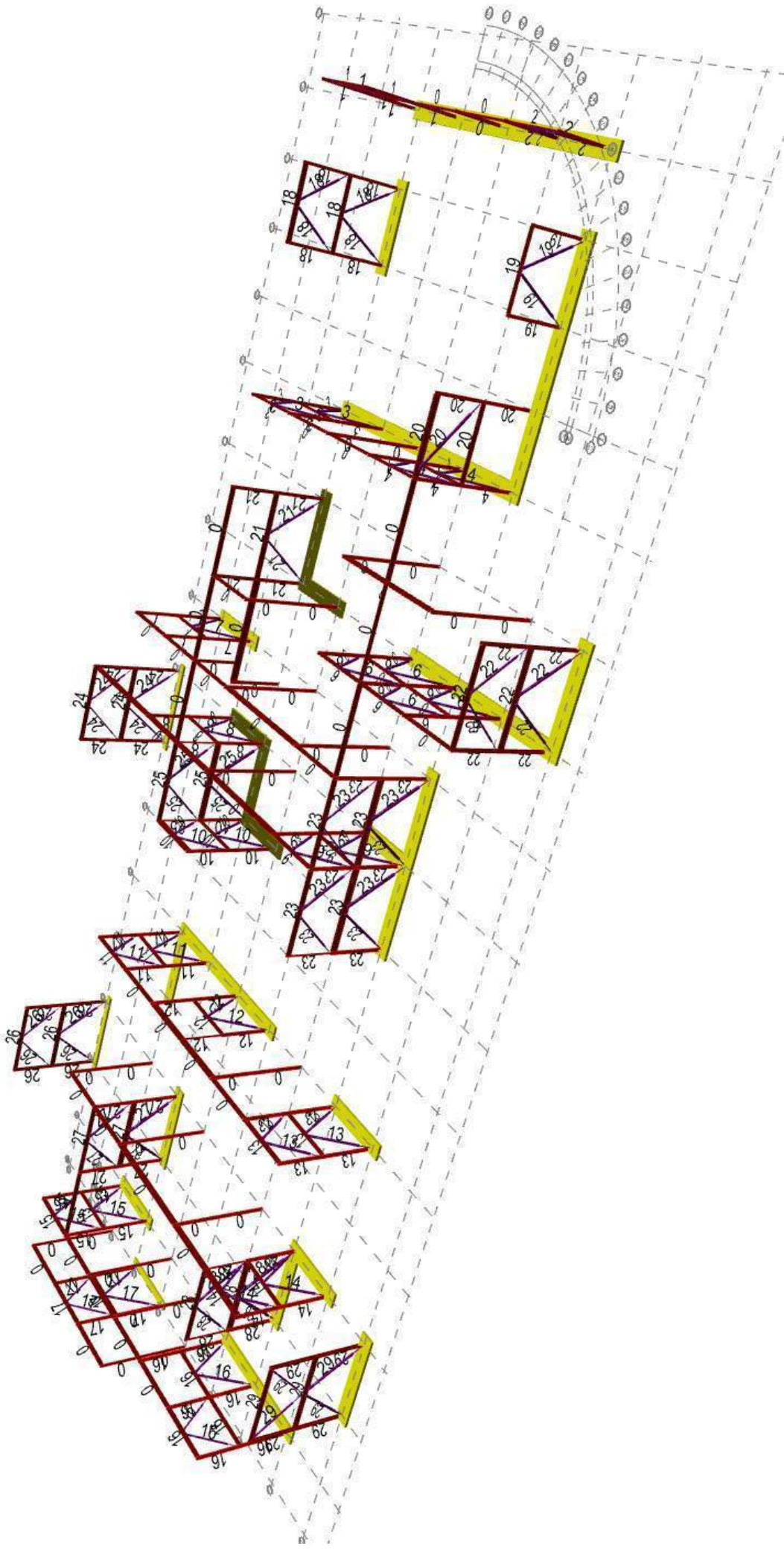
South Hill 1015 Building - East View



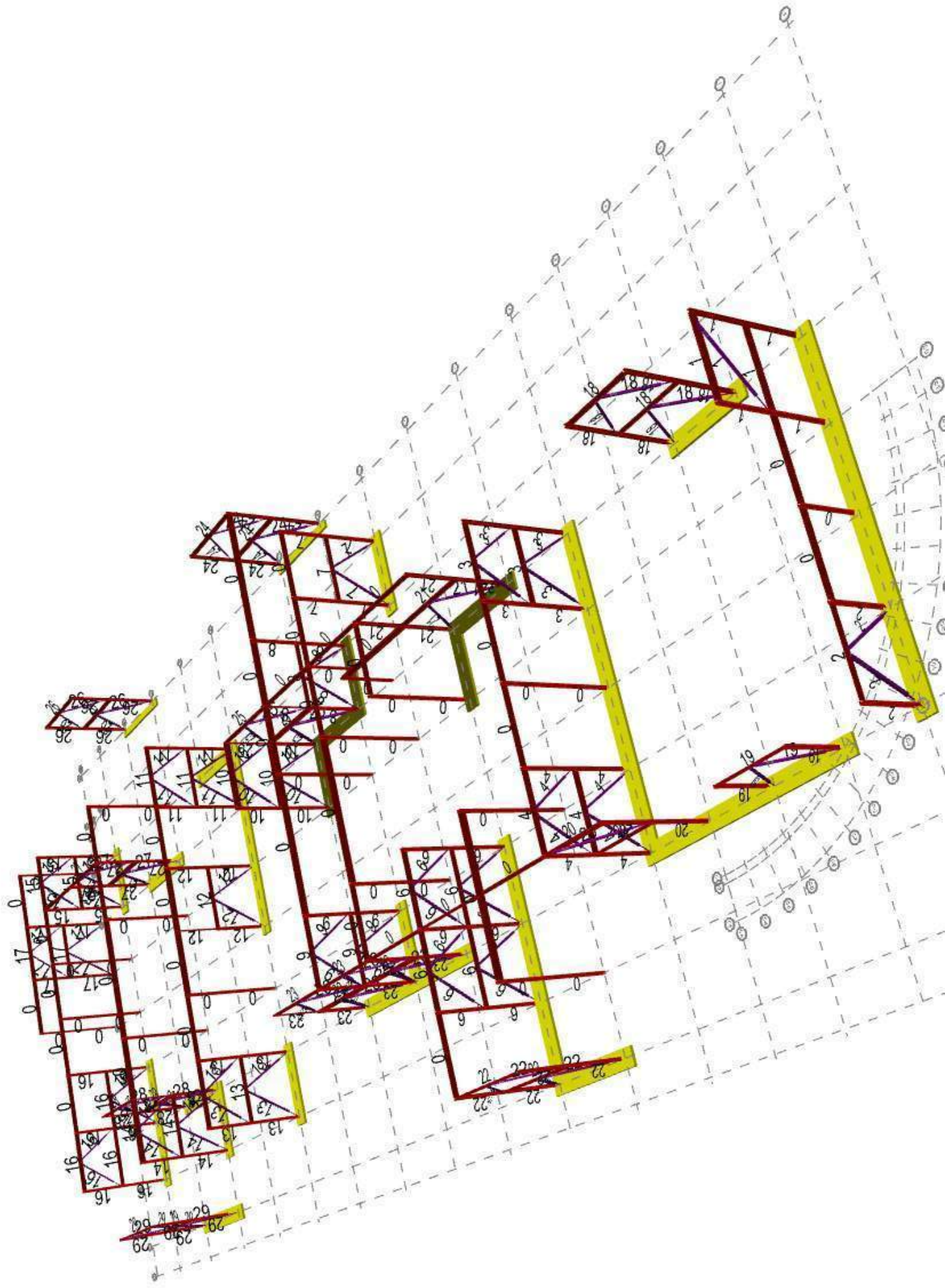
South Hill 1015 Building - Frames From West Side



South Hill 1015 Building - Frames From South Side



South Hill 1015 Building - Frames From East Side



South Hill 1015 Building - Frames From North Side



RAM Steel 24.00.01.18

DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy

Cat 4

Building Code: IBC

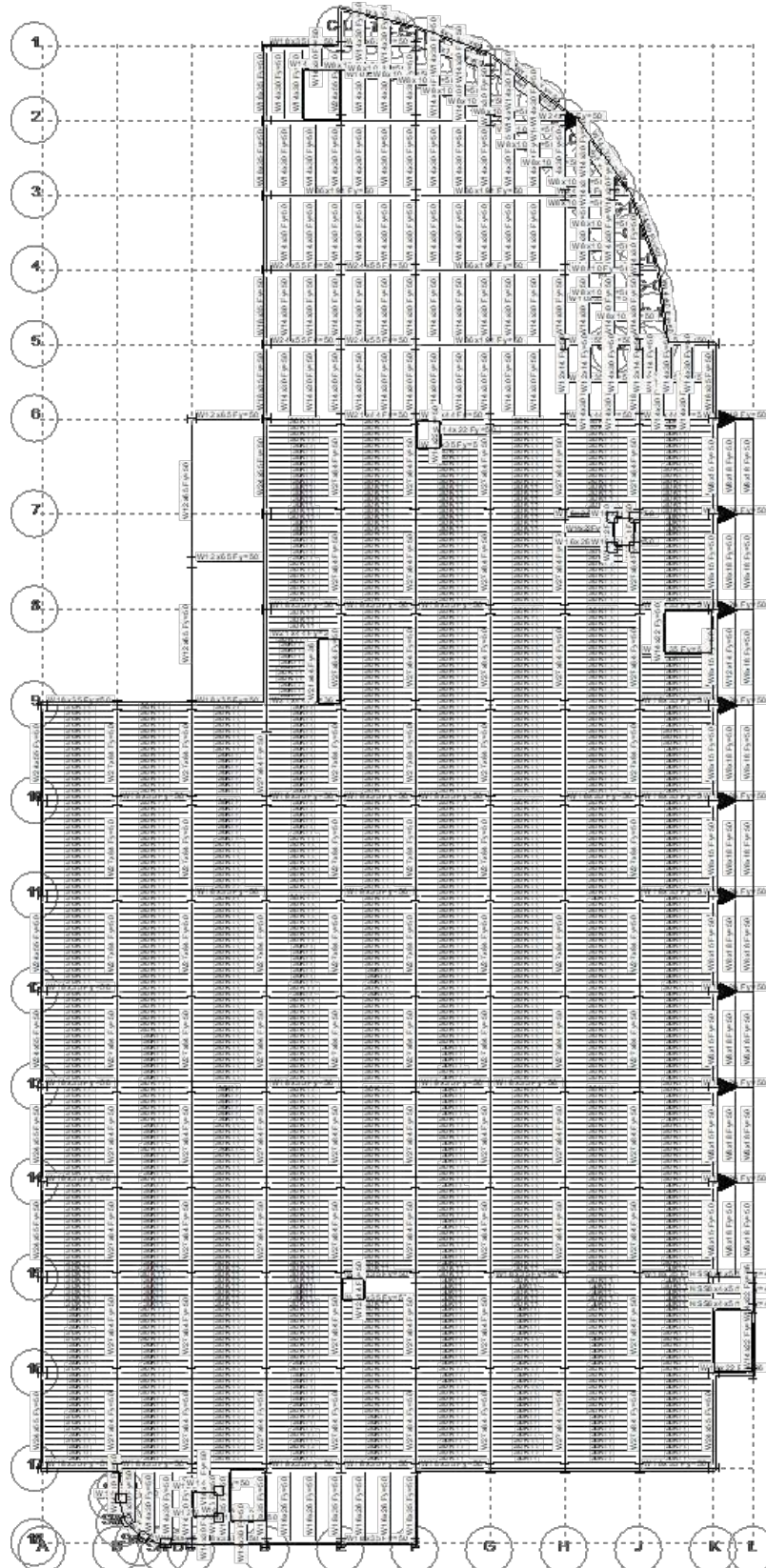
Page 1/1

12/09/24 12:48:10

Steel Code: AISC360-16 LRFD

Floor Type: 2nd Floor

Beam Designs



Floor Map



RAM Steel 24.00.01.18

DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy

Cat 4

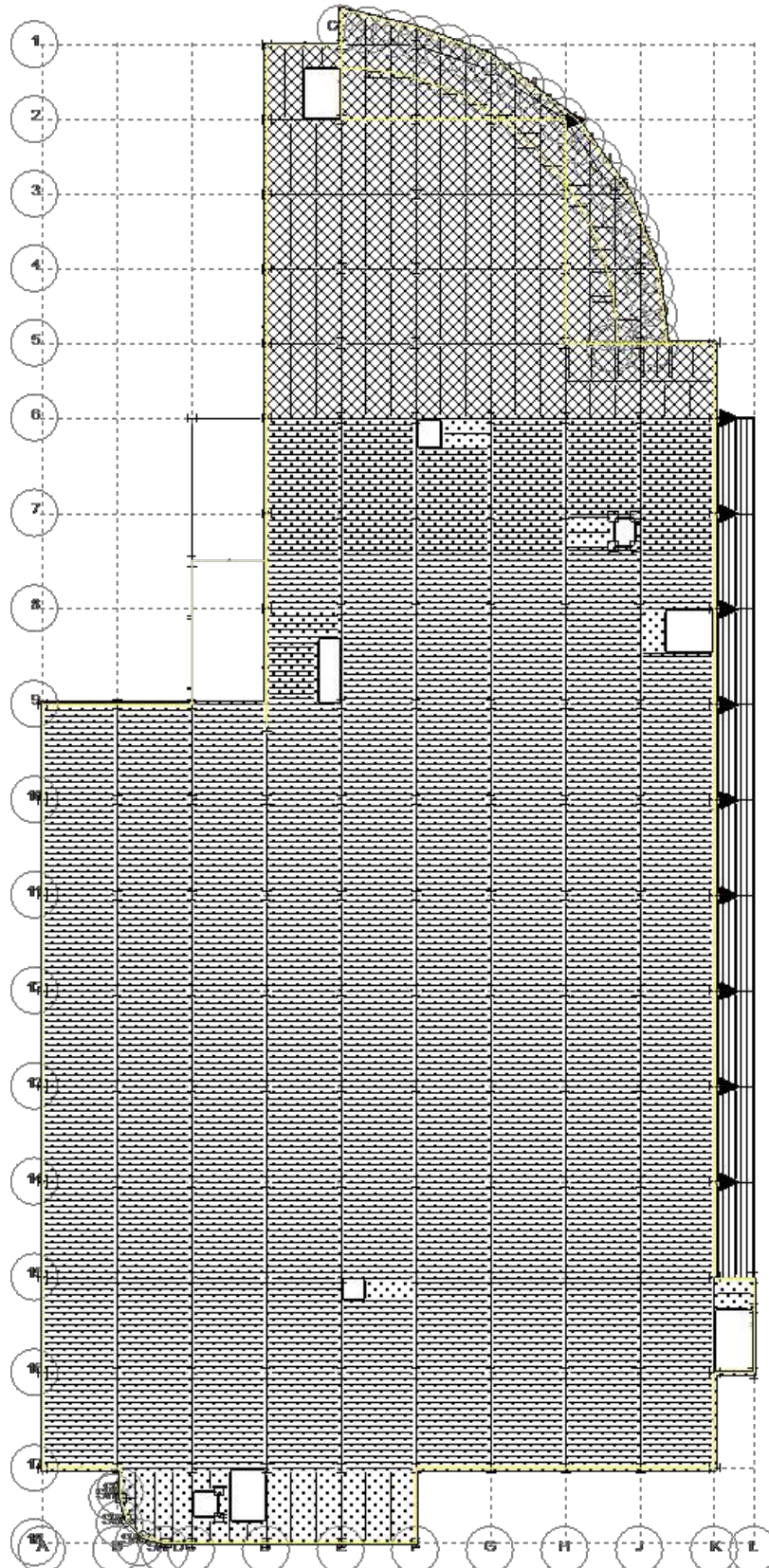
Page 1/2

12/09/24 12:48:10

Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

Floor Type: 2nd Floor



Floor Map



RAM Steel 24.00.01.18

DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy

Cat 4

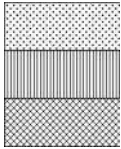
Page 2/2

12/09/24 12:48:10

Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

Surface Loads



Label	DL psf	CDL psf	LLReduction psfType	PLL psf	CLL psf	Mass DL psf
Typ Floor	12.0	0.0	100.0Unreducible	0.0	0.0	22.0
Typ Canopy	10.0	0.0	0.0Roof	0.0	0.0	10.0
Typ Floor - Comp	10.0	0.0	100.0Unreducible	0.0	0.0	18.0

Line Loads

	Label	DL k/ft	CDL k/ft	LLReduction k/ftType	PLL k/ft	CLL k/ft	Mass DL k/ft
L5	2nd Flr Ext Wall	0.000	0.000	0.000Reducible	0.000	0.000	0.318
L6	2nd Flr NE Balcony	0.180	0.000	0.000Reducible	0.000	0.000	0.180
L7	2nd Flr NE Window	0.204	0.000	0.000Reducible	0.000	0.000	0.087
L8	2nd Flr NE Wall Bel	0.000	0.000	0.000Reducible	0.000	0.000	0.164
L9	2nd Flr NW Entry	0.324	0.000	0.000Reducible	0.000	0.000	0.324



RAM Steel 24.00.01.18

DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy

Cat 4

Building Code: IBC

Page 1/1

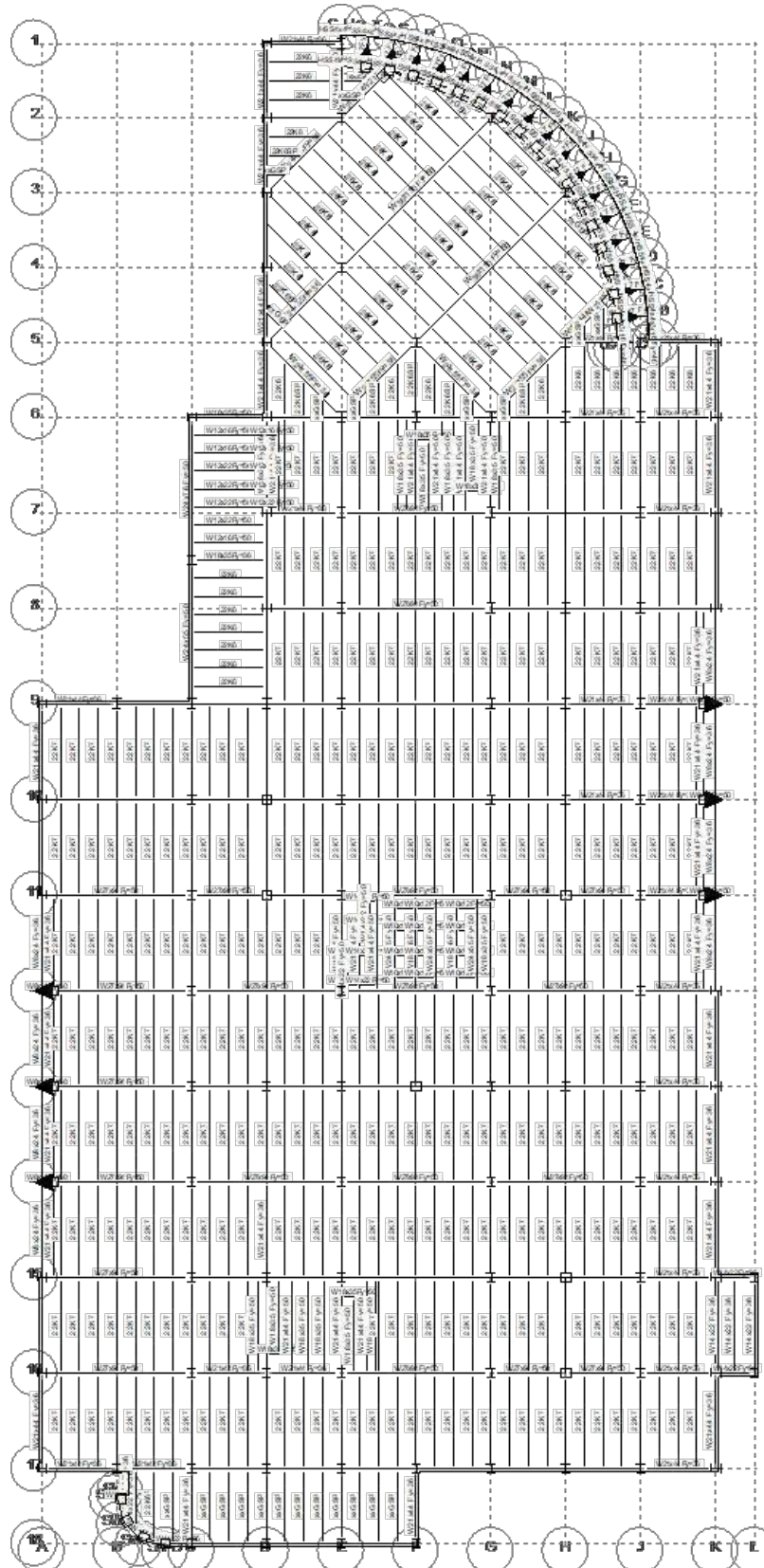
12/09/24 12:48:10

Steel Code: AISC360-16 LRFD

Floor Map

Floor Type: Roof

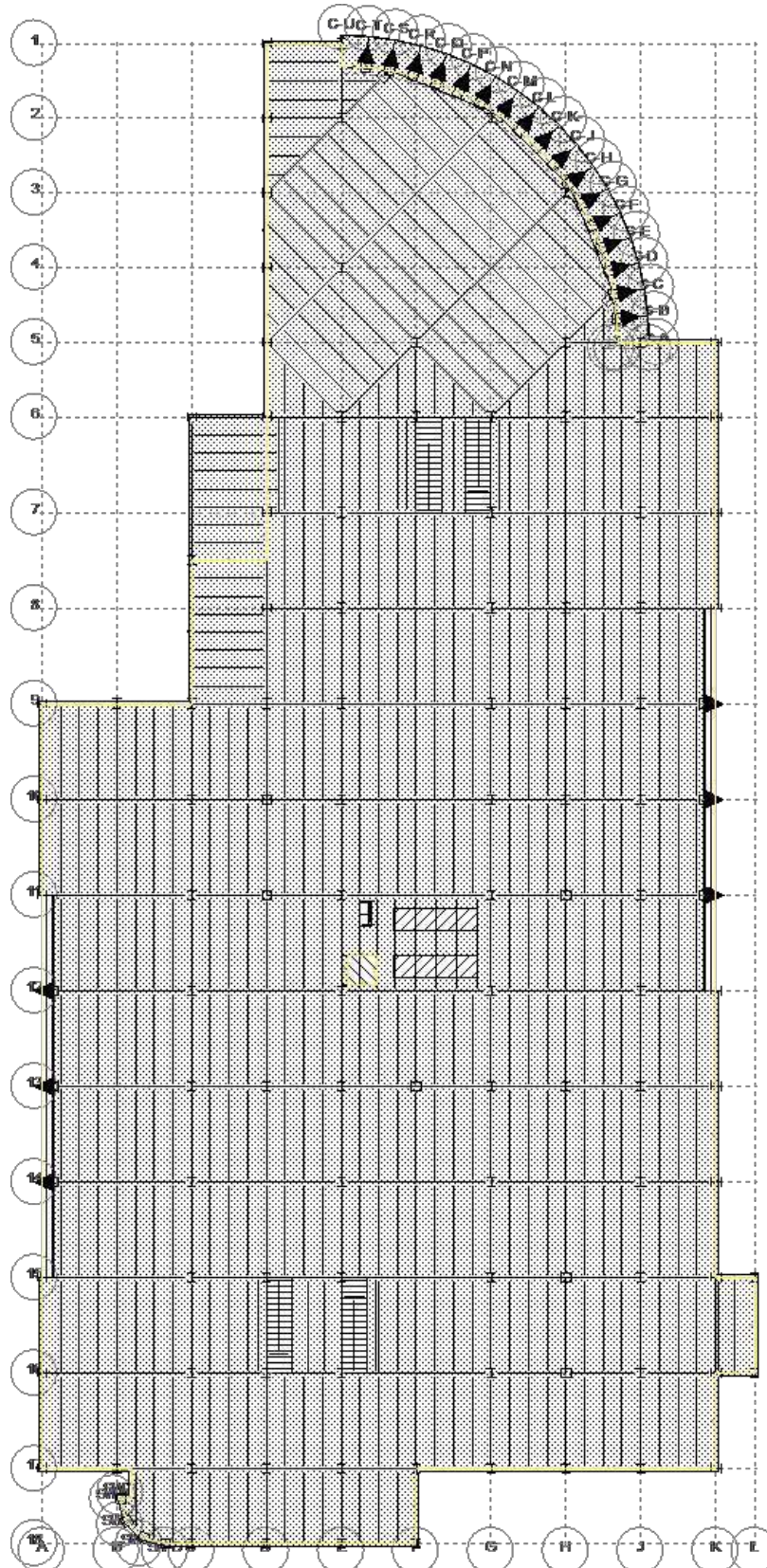
Beam Designs



Page 1/2

Steel Code: AISC360-16 LRFD

Floor Type: Roof



Floor Map



RAM Steel 24.00.01.18

DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy

Cat 4

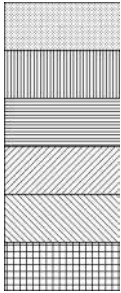
Page 2/2

12/09/24 12:48:10

Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

Surface Loads



Label	DL psf	CDL psf	LLReduction psfType	PLL psf	CLL psf	Mass DL psf
Typ Roof	18.0	0.0	0.0Roof	0.0	0.0	22.0
Typ Canopy	10.0	0.0	0.0Roof	0.0	0.0	10.0
Mech Units	73.5	0.0	0.0Reducible	0.0	0.0	78.5
Chillers	96.0	0.0	0.0Reducible	0.0	0.0	101.0
Pump	45.0	0.0	0.0Reducible	0.0	0.0	50.0
Buffer Tank	395.0	0.0	0.0Reducible	0.0	0.0	400.0

Line Loads

	Label	DL k/ft	CDL k/ft	LLReduction k/ftType	PLL k/ft	CLL k/ft	Mass DL k/ft
L1	Rf Ext Wall	0.057	0.000	0.000Reducible	0.000	0.000	0.213
L2	Rf Canopy Edge	0.030	0.000	0.000Reducible	0.000	0.000	0.030
L3	Rf Win. Wall @ Ent	0.000	0.000	0.000Reducible	0.000	0.000	0.087
L4	Rf NW Entry	0.150	0.000	0.000Reducible	0.000	0.000	0.150
L10	Pump Curb	0.100	0.000	0.000Reducible	0.000	0.000	0.100



QUANTUM | CONSULTING ENGINEERS

**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

1015 39th AVENUE SE
PUYALLUP, WA 98374

QUANTUM JOB NUMBER: 19305.04

RAM BUILDING ANALYSIS

ASCE 7-16 IRREGULARITY CHECKS, CRITERIA, AND FORCES

Seismic Force Resisting System Irregularities

Structure: **South Hill 1015 Building**
 Address: **1015 39th Ave SE Puyallup WA**

Horizontal Regularities per ASCE 7-16 Table 12.3-1

<u>Type</u>	<u>Description</u>	<u>Notes</u>
1a	Torsional Irregularity (See calc on following page)	$d_{max} / d_{avg} = 1.36$ at worst case. Torsional Irregularity occurs. Diaphragm forces must be increased by 25% per 12.3.3.4 $A_x = 1.14$, $e = 5.7\%$ East West Only
1b	Extreme Torsional Irregularity	$d_{max} / d_{avg} = 1.36$ at worst case. Extreme Torsional Irregularity does not occur. $\rho = 1.0$ per 12.3.4.2
2	Reentrant Corner Irregularity	Occurs at Grid 9 / D Diaphragm forces must be increased
3	Diaphragm Discontinuity Irregularity	Significant change in stiffness occurs from roof diaphragm to floor diaphragm. Diaphragm forces must be increased by 25% per 12.3.3.4
4	Out-of-plane Offset	Does not occur.
5	Nonparallel System Irregularity	Does not occur.

Vertical Regularities per ASCE 7-16 Table 12.3-2

<u>Type</u>	<u>Description</u>	<u>Notes</u>
1a	Stiffness-Soft Story Irregularity	Does not occur.
1b	Stiffness- Extreme Soft Story Irregularity	Does not occur.
2	Weight (Mass) Irregularity	Does not occur.
3	Vertical Geometric Irregularity	Does not occur.
4	In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity	Occurs, columns and connection designed for overstrength factor
5a	Discontinuity in Lateral Strength-Weak Story Irregularity	Does not occur.
5b	Discontinuity in Lateral Strength-Extreme Weak Story Irregularity	Does not occur.



Quantum Consulting Engineers LLC
 1511 Third Avenue, Suite 323
 Seattle, WA 98101

Project: South Hill 1015 Building

Client: Benaroya

Designer: TVM

Project: 19305.04

Sheet: 1

Chkd By:

12/6/24

Seismic Force Resisting System Irregularities

Torsional Irregularity Check

Roof		RAM Elastic			Ax	
Grid	Direction	Deflection	δ_{max}	δ_{avg}	EQ 12.8-14	
2	N/S +5%	1.095		1.10	0.811	1.27
17	N/S +5%	0.527	$\delta_{max} / \delta_{avg} =$		1.35	
2	N/S -5%	0.665		0.76	0.715	0.79
17	N/S -5%	0.764	$\delta_{max} / \delta_{avg} =$		1.07	
A	E/W +5%	0.582		0.61	0.597	0.73
K	E/W +5%	0.613	$\delta_{max} / \delta_{avg} =$		1.03	
A	E/W -5%	0.723		0.72	0.616	0.95
K	E/W -5%	0.510	$\delta_{max} / \delta_{avg} =$		1.17	

Level 2		RAM Elastic			Ax	
Grid	Direction	Deflection	δ_{max}	δ_{avg}	EQ 12.8-14	
2	N/S +5%	0.549		0.55	0.459	0.99
17	N/S +5%	0.370	$\delta_{max} / \delta_{avg} =$		1.19	
2	N/S -5%	0.364		0.52	0.444	0.97
17	N/S -5%	0.524	$\delta_{max} / \delta_{avg} =$		1.18	
A	E/W +5%	0.349		0.37	0.360	0.74
K	E/W +5%	0.372	$\delta_{max} / \delta_{avg} =$		1.03	
A	E/W -5%	0.394		0.39	0.366	0.80
K	E/W -5%	0.338	$\delta_{max} / \delta_{avg} =$		1.08	

Accidental Eccentricity Amplifications

Code Eccentricity = 5.00%

East / West

Ax (Controlling) = 1.27

Amplified Eccentricity = 6.33%

North / South

Ax (Controlling) = 1.00

Amplified Eccentricity = 5.00%



Quantum Consulting Engineers LLC

1511 Third Avenue, Suite 323
Seattle, WA 98101

Project: South Hill 1015 Building

Client: Benaroya

Designer: TVM

Job No: 19305.04

Sheet: 2

Chkd By:

12/6/24

Criteria, Mass and Exposure Data



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CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor (DL): 1.40 Scale Factor (LL): 0.50
Scale Factor (Roof): 0.50 Scale Factor (Snow): 0.20
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 8.00
Advanced Wall Mesh Criteria: Optimization Level : 3 Shape Quality Level : 0.60
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Use Reduced Stiffness for Steel Members (AISC 360): $\tau_b = 1.00$
Sign not considered for Dynamic Load Case Results.
Rigid Links Not Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

DIAPHRAGM DATA:

Story	Diaph #	Diaph Type
Roof	1	Semirigid
2nd Floor	1	Rigid

Disconnect Internal Nodes of Beams: Yes
Disconnect Nodes outside Slab Boundary: Yes

Semirigid Diaphragm Parameters:

Use Beams for Exterior Boundary
Calculate Diaphragm Mass
Hard Node Density Factor: 1.00

STORY MASS DATA:

Includes Self Mass of:

Beams
Columns (Half mass of columns above and below)
Walls (Half mass of walls above and below)
Slabs/Deck

Calculated Values:

Story	Diaph #	Weight kips	Mass k-s2/ft	MMI ft-k-s2	Xm ft	Ym ft	EccX ft	EccY ft
Roof	1	3160.51	98.15	2117348	119.87	223.97	12.00	32.37
2nd Floor	1	6985.62	216.94	5201035	123.53	236.65	11.97	32.99
	None	33.91	1.05	655	53.96	321.92	--	--



Criteria, Mass and Exposure Data

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Story	Diaph #	Combine
Roof	1	None
2nd Floor	1	None
	None	1-2nd Floor

Combined/Merged Values:

Story	Diaph #	Weight kips	Mass k-s2/ft	MMI ft-k-s2	Xm ft	Ym ft	EccX ft	EccY ft
Roof	1	3160.5	98.15	2117347	119.87	223.97	12.00	32.37
2nd Floor	1	7019.5	218.00	5214383	123.19	237.07	11.97	32.99

Distributed Mass Values for Pseudo-Flexible or Meshed Diaphragms:

Story	Diaph #	Total Weight kips	Total Mass k-s2/ft
Roof	1	3114.86	96.735

WIND EXPOSURE DATA:

Calculated Values:

Story	Diaph #	Building Extents (ft)				Expose	Parapet ft
		Min X	Max X	Min Y	Max Y		
Roof	1	-1.00	239.00	-1.00	504.84	Full	2.34
2nd Floor	1	-0.67	238.67	-0.67	514.85	Full	0.00

STORY GRAVITY LOADS DATA:

Includes Weight of:

Beams
Columns
Walls
Slabs/Deck

Live Load Reduction (Calculated)

Reducible : 0.00 %
Storage : 0.00 %

Calculated Values:

Story	Diaph #	Dead kips	Xc ft	Yc ft	Live kips	Xc ft	Yc ft
Roof	1	2692.58	119.58	224.35	0.00	0.00	0.00
2nd Floor	1	5732.89	124.31	241.26	8983.38	121.80	225.44
	None	31.19	54.31	325.31	0.00	0.00	0.00
Story	Diaph #	Snow kips	Xc ft	Yc ft	Combine		
Roof	1	2320.09	120.29	225.23	None		

Criteria, Mass and Exposure Data



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2nd Floor	1	94.83	231.58	233.04	None
	None	0.00	0.00	0.00	None

Some Gravity Load has been detected on one or more stories that is not associated with any diaphragms. It will be ignored in Notional Loads unless it is combined with one or more diaphragms. See the Loads - Gravity for Notional command.

Loads and Applied Forces



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LOAD CASE: Seismic

Seismic ASCE 7-16 Equivalent Lateral Force
Importance Factor: 1.50 TL: 6.00 s
Site Class C: Very Dense Soil and Soft Rock
Ss: 1.257 g S1: 0.434 g
Fa: 1.200 Fv: 1.500 SDs: 1.006 g SD1: 0.434 g
Risk Category: IV Seismic Design Category: D
Provisions for: Force
Ground Level: Base

Dir	Eccent	R	Ta Equation				Building Period-T		
X	+ And -	8.00	Std,Ct=0.030,x=0.75				Calculated		
Y	+ And -	8.00	Std,Ct=0.030,x=0.75				Calculated		
Dir	Ta	Cu	T	T-used	Cs	Eq12.8-2 Cs(max) Eq12.8-3	Cs(min) Eq12.8-5	Cs-used	k
X	0.461	1.400	0.657	0.645	0.189	0.126	0.066	0.126	1.072
Dir	Ta	Cu	T	T-used	Cs	Eq12.8-2 Cs(max) Eq12.8-3	Cs(min) Eq12.8-5	Cs-used	k
Y	0.461	1.400	0.618	0.618	0.189	0.132	0.066	0.132	1.059

Total Building Weight (kips) = 10180.04

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	594.65	0.00	119.87	256.34
2nd Floor	1	20.83	690.00	0.00	123.19	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	594.647	0.000
		594.65	0.00

APPLIED STORY FORCES

Type: EQ_ASCE716_X_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	594.65	0.00
2nd Floor	20.83	690.00	0.00

Loads and Applied Forces



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1284.65	0.00
---------	------

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	594.65	0.00	119.87	191.59
2nd Floor	1	20.83	690.00	0.00	123.19	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	594.647	0.000
		594.65	0.00

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	594.65	0.00
2nd Floor	20.83	690.00	0.00
		1284.65	0.00

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	0.00	618.00	131.87	223.97
2nd Floor	1	20.83	0.00	723.01	135.16	237.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	0.000	618.005
		0.00	618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_+E_F

Loads and Applied Forces



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Level	Ht ft	Fx kips	Fy kips
Roof	38.16	0.00	618.00
2nd Floor	20.83	0.00	723.01
		0.00	1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	0.00	618.00	107.87	223.97
2nd Floor	1	20.83	0.00	723.01	111.22	237.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	0.000	618.005
		0.00	618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	0.00	618.00
2nd Floor	20.83	0.00	723.01
		0.00	1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_+E_0.3Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	594.65	185.40	131.87	256.34
2nd Floor	1	20.83	690.00	216.90	135.16	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	594.647	185.401

Loads and Applied Forces



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DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy

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594.65

185.40

APPLIED STORY FORCES

Type: EQ_ASCE716_X_+E_0.3Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	594.65	185.40
2nd Floor	20.83	690.00	216.90
		1284.65	402.31

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_+E_-0.3Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	594.65	-185.40	131.87	256.34
2nd Floor	1	20.83	690.00	-216.90	135.16	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	594.647	-185.401
		594.65	-185.40

APPLIED STORY FORCES

Type: EQ_ASCE716_X_+E_-0.3Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	594.65	-185.40
2nd Floor	20.83	690.00	-216.90
		1284.65	-402.31

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-X_+E_0.3Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-594.65	185.40	131.87	256.34
2nd Floor	1	20.83	-690.00	216.90	135.16	270.06

Loads and Applied Forces



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Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx	Sum Fy
		kips	kips
Roof	1	-594.647	185.401
		<u>-594.65</u>	<u>185.40</u>

APPLIED STORY FORCES

Type: EQ_ASCE716_-X_+E_0.3Y_+E_F

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	38.16	-594.65	185.40
2nd Floor	20.83	-690.00	216.90
		<u>-1284.65</u>	<u>402.31</u>

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-X_+E_-0.3Y_+E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	38.16	-594.65	-185.40	131.87	256.34
2nd Floor	1	20.83	-690.00	-216.90	135.16	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx	Sum Fy
		kips	kips
Roof	1	-594.647	-185.401
		<u>-594.65</u>	<u>-185.40</u>

APPLIED STORY FORCES

Type: EQ_ASCE716_-X_+E_-0.3Y_+E_F

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	38.16	-594.65	-185.40
2nd Floor	20.83	-690.00	-216.90
		<u>-1284.65</u>	<u>-402.31</u>

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_0.3X_+E_Y_+E_F

Loads and Applied Forces



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Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	178.39	618.00	131.87	256.34
2nd Floor	1	20.83	207.00	723.01	135.16	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	178.394	618.005
		178.39	618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_0.3X_+E_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	178.39	618.00
2nd Floor	20.83	207.00	723.01
		385.40	1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_0.3X_+E_-Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	178.39	-618.00	131.87	256.34
2nd Floor	1	20.83	207.00	-723.01	135.16	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	178.394	-618.005
		178.39	-618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_0.3X_+E_-Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	178.39	-618.00
2nd Floor	20.83	207.00	-723.01

Loads and Applied Forces



RAM Frame 24.00.01.18

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385.40 -1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-0.3X_+E_Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-178.39	618.00	131.87	256.34
2nd Floor	1	20.83	-207.00	723.01	135.16	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-178.394	618.005
		-178.39	618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_-0.3X_+E_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-178.39	618.00
2nd Floor	20.83	-207.00	723.01
		-385.40	1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-0.3X_+E_-Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-178.39	-618.00	131.87	256.34
2nd Floor	1	20.83	-207.00	-723.01	135.16	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-178.394	-618.005
		-178.39	-618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_-0.3X_+E_-Y_+E_F

Level	Ht	Fx	Fy
-------	----	----	----

Loads and Applied Forces



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Cat 4

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	ft	kips	kips
Roof	38.16	-178.39	-618.00
2nd Floor	20.83	-207.00	-723.01
		-385.40	-1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_+E_0.3Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	594.65	185.40	107.87	256.34
2nd Floor	1	20.83	690.00	216.90	111.22	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	594.647	185.401
		594.65	185.40

APPLIED STORY FORCES

Type: EQ_ASCE716_X_+E_0.3Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	594.65	185.40
2nd Floor	20.83	690.00	216.90
		1284.65	402.31

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_+E_-0.3Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	594.65	-185.40	107.87	256.34
2nd Floor	1	20.83	690.00	-216.90	111.22	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	594.647	-185.401

Loads and Applied Forces



RAM Frame 24.00.01.18

DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy

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594.65

-185.40

APPLIED STORY FORCES

Type: EQ_ASCE716_X_+E_-0.3Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	594.65	-185.40
2nd Floor	20.83	690.00	-216.90
		<u>1284.65</u>	<u>-402.31</u>

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-X_+E_0.3Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-594.65	185.40	107.87	256.34
2nd Floor	1	20.83	-690.00	216.90	111.22	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-594.647	185.401
		<u>-594.65</u>	<u>185.40</u>

APPLIED STORY FORCES

Type: EQ_ASCE716_-X_+E_0.3Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-594.65	185.40
2nd Floor	20.83	-690.00	216.90
		<u>-1284.65</u>	<u>402.31</u>

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-X_+E_-0.3Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-594.65	-185.40	107.87	256.34
2nd Floor	1	20.83	-690.00	-216.90	111.22	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

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Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-594.647	-185.401
		-594.65	-185.40

APPLIED STORY FORCES

Type: EQ_ASCE716_-X_+E_-0.3Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-594.65	-185.40
2nd Floor	20.83	-690.00	-216.90
		-1284.65	-402.31

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_0.3X_+E_Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	178.39	618.00	107.87	256.34
2nd Floor	1	20.83	207.00	723.01	111.22	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	178.394	618.005
		178.39	618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_0.3X_+E_Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	178.39	618.00
2nd Floor	20.83	207.00	723.01
		385.40	1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_0.3X_+E_-Y_-E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y
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		ft	kips	kips	ft	ft
Roof	1	38.16	178.39	-618.00	107.87	256.34
2nd Floor	1	20.83	207.00	-723.01	111.22	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	178.394	-618.005
		178.39	-618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_0.3X_+E_-Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	178.39	-618.00
2nd Floor	20.83	207.00	-723.01
		385.40	-1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-0.3X_+E_Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-178.39	618.00	107.87	256.34
2nd Floor	1	20.83	-207.00	723.01	111.22	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-178.394	618.005
		-178.39	618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_-0.3X_+E_Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-178.39	618.00
2nd Floor	20.83	-207.00	723.01
		-385.40	1341.02

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APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-0.3X_+E_-Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-178.39	-618.00	107.87	256.34
2nd Floor	1	20.83	-207.00	-723.01	111.22	270.06

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-178.394	-618.005
		<u>-178.39</u>	<u>-618.00</u>

APPLIED STORY FORCES

Type: EQ_ASCE716_-0.3X_+E_-Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-178.39	-618.00
2nd Floor	20.83	-207.00	-723.01
		<u>-385.40</u>	<u>-1341.02</u>

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_-E_0.3Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	594.65	185.40	131.87	191.59
2nd Floor	1	20.83	690.00	216.90	135.16	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	594.647	185.401
		<u>594.65</u>	<u>185.40</u>

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_0.3Y_+E_F

Level	Ht ft	Fx kips	Fy kips
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Roof	38.16	594.65	185.40
2nd Floor	20.83	690.00	216.90
		1284.65	402.31

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_-E_-0.3Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	594.65	-185.40	131.87	191.59
2nd Floor	1	20.83	690.00	-216.90	135.16	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	594.647	-185.401
		594.65	-185.40

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_-0.3Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	594.65	-185.40
2nd Floor	20.83	690.00	-216.90
		1284.65	-402.31

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-X_-E_0.3Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-594.65	185.40	131.87	191.59
2nd Floor	1	20.83	-690.00	216.90	135.16	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-594.647	185.401
		-594.65	185.40

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APPLIED STORY FORCES

Type: EQ_ASCE716_-X_-E_0.3Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-594.65	185.40
2nd Floor	20.83	-690.00	216.90
		<u>-1284.65</u>	<u>402.31</u>

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-X_-E_-0.3Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-594.65	-185.40	131.87	191.59
2nd Floor	1	20.83	-690.00	-216.90	135.16	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-594.647	-185.401
		<u>-594.65</u>	<u>-185.40</u>

APPLIED STORY FORCES

Type: EQ_ASCE716_-X_-E_-0.3Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-594.65	-185.40
2nd Floor	20.83	-690.00	-216.90
		<u>-1284.65</u>	<u>-402.31</u>

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_0.3X_-E_Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	178.39	618.00	131.87	191.59
2nd Floor	1	20.83	207.00	723.01	135.16	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx	Sum Fy
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		kip	kip
Roof	1	178.394	618.005
		178.39	618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_0.3X_-E_Y_+E_F

Level	Ht ft	Fx kip	Fy kip
Roof	38.16	178.39	618.00
2nd Floor	20.83	207.00	723.01
		385.40	1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_0.3X_-E_-Y_+E_F

Level	Diaph.#	Ht ft	Fx kip	Fy kip	X ft	Y ft
Roof	1	38.16	178.39	-618.00	131.87	191.59
2nd Floor	1	20.83	207.00	-723.01	135.16	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kip	Sum Fy kip
Roof	1	178.394	-618.005
		178.39	-618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_0.3X_-E_-Y_+E_F

Level	Ht ft	Fx kip	Fy kip
Roof	38.16	178.39	-618.00
2nd Floor	20.83	207.00	-723.01
		385.40	-1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-0.3X_-E_Y_+E_F

Level	Diaph.#	Ht ft	Fx kip	Fy kip	X ft	Y ft
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Roof	1	38.16	-178.39	618.00	131.87	191.59
2nd Floor	1	20.83	-207.00	723.01	135.16	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-178.394	618.005
		<u>-178.39</u>	<u>618.00</u>

APPLIED STORY FORCES

Type: EQ_ASCE716_-0.3X_-E_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-178.39	618.00
2nd Floor	20.83	-207.00	723.01
		<u>-385.40</u>	<u>1341.02</u>

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-0.3X_-E_-Y_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-178.39	-618.00	131.87	191.59
2nd Floor	1	20.83	-207.00	-723.01	135.16	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-178.394	-618.005
		<u>-178.39</u>	<u>-618.00</u>

APPLIED STORY FORCES

Type: EQ_ASCE716_-0.3X_-E_-Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-178.39	-618.00
2nd Floor	20.83	-207.00	-723.01
		<u>-385.40</u>	<u>-1341.02</u>

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APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_-E_0.3Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	594.65	185.40	107.87	191.59
2nd Floor	1	20.83	690.00	216.90	111.22	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	594.647	185.401
		594.65	185.40

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_0.3Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	594.65	185.40
2nd Floor	20.83	690.00	216.90
		1284.65	402.31

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_-E_-0.3Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	594.65	-185.40	107.87	191.59
2nd Floor	1	20.83	690.00	-216.90	111.22	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	594.647	-185.401
		594.65	-185.40

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_-0.3Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	594.65	-185.40

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2nd Floor	20.83	690.00	-216.90
		1284.65	-402.31

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-X_-E_0.3Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-594.65	185.40	107.87	191.59
2nd Floor	1	20.83	-690.00	216.90	111.22	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-594.647	185.401
		-594.65	185.40

APPLIED STORY FORCES

Type: EQ_ASCE716_-X_-E_0.3Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-594.65	185.40
2nd Floor	20.83	-690.00	216.90
		-1284.65	402.31

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-X_-E_-0.3Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-594.65	-185.40	107.87	191.59
2nd Floor	1	20.83	-690.00	-216.90	111.22	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-594.647	-185.401
		-594.65	-185.40

APPLIED STORY FORCES

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Type: EQ_ASCE716_-X_-E_-0.3Y_-E_F

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	38.16	-594.65	-185.40
2nd Floor	20.83	-690.00	-216.90
		<u>-1284.65</u>	<u>-402.31</u>

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_0.3X_-E_Y_-E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	38.16	178.39	618.00	107.87	191.59
2nd Floor	1	20.83	207.00	723.01	111.22	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx	Sum Fy
		kips	kips
Roof	1	178.394	618.005
		<u>178.39</u>	<u>618.00</u>

APPLIED STORY FORCES

Type: EQ_ASCE716_0.3X_-E_Y_-E_F

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	38.16	178.39	618.00
2nd Floor	20.83	207.00	723.01
		<u>385.40</u>	<u>1341.02</u>

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_0.3X_-E_-Y_-E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	38.16	178.39	-618.00	107.87	191.59
2nd Floor	1	20.83	207.00	-723.01	111.22	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx	Sum Fy
		kips	kips

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Roof	1	178.394	-618.005
		178.39	-618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_0.3X_-E_-Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	178.39	-618.00
2nd Floor	20.83	207.00	-723.01
		385.40	-1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-0.3X_-E_Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-178.39	618.00	107.87	191.59
2nd Floor	1	20.83	-207.00	723.01	111.22	204.07

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-178.394	618.005
		-178.39	618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_-0.3X_-E_Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-178.39	618.00
2nd Floor	20.83	-207.00	723.01
		-385.40	1341.02

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_-0.3X_-E_-Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	-178.39	-618.00	107.87	191.59

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2nd Floor	1	20.83	-207.00	-723.01	111.22	204.07
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Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	-178.394	-618.005
		-178.39	-618.00

APPLIED STORY FORCES

Type: EQ_ASCE716_-0.3X_-E_-Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	-178.39	-618.00
2nd Floor	20.83	-207.00	-723.01
		-385.40	-1341.02

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LOAD CASE: Fpx_min X

Seismic User Defined Story Forces
Ground Level: Base

APPLIED DIAPHRAGM FORCES

Type: EQ_User

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	953.31	0.00	119.87	223.97
2nd Floor	1	20.83	2117.77	0.00	123.53	236.67

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	953.310	0.000
		953.31	0.00

APPLIED STORY FORCES

Type: EQ_User

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	953.31	0.00
2nd Floor	20.83	2117.77	0.00
		3071.08	0.00

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LOAD CASE: Fpx_min Y

Seismic User Defined Story Forces

Ground Level: Base

APPLIED DIAPHRAGM FORCES

Type: EQ_User

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	0.00	953.31	119.87	223.97
2nd Floor	1	20.83	0.00	2117.77	123.53	236.67

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	0.000	953.310
		0.00	953.31

APPLIED STORY FORCES

Type: EQ_User

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	0.00	953.31
2nd Floor	20.83	0.00	2117.77
		0.00	3071.08

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LOAD CASE: Fpx X

Seismic

User Defined Story Forces

Ground Level:

Base

APPLIED DIAPHRAGM FORCES

Type: EQ_User

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	579.12	0.00	119.87	223.97
2nd Floor	1	20.83	883.61	0.00	123.53	236.67

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	579.120	0.000
		579.12	0.00

APPLIED STORY FORCES

Type: EQ_User

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	579.12	0.00
2nd Floor	20.83	883.61	0.00
		1462.73	0.00

Loads and Applied Forces



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LOAD CASE: Fpx Y

Seismic

User Defined Story Forces

Ground Level:

Base

APPLIED DIAPHRAGM FORCES

Type: EQ_User

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	38.16	0.00	579.12	119.87	223.97
2nd Floor	1	20.83	0.00	883.61	123.53	236.67

Applied Loads for Pseudo-Flexible or Semirigid Diaphragms:

Story	Diaph #	Sum Fx kips	Sum Fy kips
Roof	1	0.000	579.120
		0.00	579.12

APPLIED STORY FORCES

Type: EQ_User

Level	Ht ft	Fx kips	Fy kips
Roof	38.16	0.00	579.12
2nd Floor	20.83	0.00	883.61
		0.00	1462.73

Loads and Applied Forces



RAM Frame 24.00.01.18

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LOAD MULTIPLIERS FOR ECCENTRIC LOADING ON SEMIRIGID DIAPHRAGM:SEISMIC AND RESPONSE SPECTRA LOAD CASES

Story : Roof Diaph. #: 1

Loading Direction	Top Region	Bottom Region	Left Region	Right Region
+X	1.31	0.73	---	---
-X	0.70	1.26	---	---
+Y	---	---	0.75	1.26
-Y	---	---	1.23	0.77

Loads and Applied Forces



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ECCENTRIC LOAD MULTIPLIERS FOR SEMIRIGID DIAPHRAGMS: USER DEFINED SEISMIC LOAD CASE

Story : Roof		Diaph. #: 1			
Loading Direction		Top Region	Bottom Region	Left Region	Right Region
+X		1.01	0.99	---	---
-X		1.01	0.99	---	---
+Y		---	---	0.99	1.01
-Y		---	---	0.99	1.01



Frame Reactions

RAM Frame 24.00.01.18

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Building Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects

Member Force Output: At Face of Joint

P-Delta: Yes Scale Factor (DL): 1.40 Scale Factor (LL): 0.50
Scale Factor (Roof): 0.50 Scale Factor (Snow): 0.20

Ground Level: Base

Mesh Criteria :

Max. Distance Between Nodes on Mesh Line (ft) : 8.00

Advanced Wall Mesh Criteria: Optimization Level : 3 Shape Quality Level : 0.60

Merge Node Tolerance (in) : 0.0100

Geometry Tolerance (in) : 0.0050

Walls Out-of-plane Stiffness Not Included in Analysis.

Use Reduced Stiffness for Steel Members (AISC 360): $\tau_b = 1.00$

Sign not considered for Dynamic Load Case Results.

Rigid Links Not Included at Fixed Beam-to-Wall Locations

Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
Sp	PosSnowLoad	RAMUSER
Sn	NegSnowLoad	RAMUSER
E1	Seismic	EQ_ASCE716_X_+E_F
E2	Seismic	EQ_ASCE716_X_-E_F
E3	Seismic	EQ_ASCE716_Y_+E_F
E4	Seismic	EQ_ASCE716_Y_-E_F
E5	Seismic	EQ_ASCE716_X_+E_0.3Y_+E_F
E6	Seismic	EQ_ASCE716_X_+E_-0.3Y_+E_F
E7	Seismic	EQ_ASCE716_-X_+E_0.3Y_+E_F
E8	Seismic	EQ_ASCE716_-X_+E_-0.3Y_+E_F
E9	Seismic	EQ_ASCE716_0.3X_+E_Y_+E_F
E10	Seismic	EQ_ASCE716_0.3X_+E_-Y_+E_F
E11	Seismic	EQ_ASCE716_-0.3X_+E_Y_+E_F
E12	Seismic	EQ_ASCE716_-0.3X_+E_-Y_+E_F
E13	Seismic	EQ_ASCE716_X_+E_0.3Y_-E_F
E14	Seismic	EQ_ASCE716_X_+E_-0.3Y_-E_F
E15	Seismic	EQ_ASCE716_-X_+E_0.3Y_-E_F
E16	Seismic	EQ_ASCE716_-X_+E_-0.3Y_-E_F
E17	Seismic	EQ_ASCE716_0.3X_+E_Y_-E_F

**Frame Reactions**

E18	Seismic	EQ_ASCE716_0.3X_+E_-Y_-E_F
E19	Seismic	EQ_ASCE716_-0.3X_+E_Y_-E_F
E20	Seismic	EQ_ASCE716_-0.3X_+E_-Y_-E_F
E21	Seismic	EQ_ASCE716_X_-E_0.3Y_+E_F
E22	Seismic	EQ_ASCE716_X_-E_-0.3Y_+E_F
E23	Seismic	EQ_ASCE716_-X_-E_0.3Y_+E_F
E24	Seismic	EQ_ASCE716_-X_-E_-0.3Y_+E_F
E25	Seismic	EQ_ASCE716_0.3X_-E_Y_+E_F
E26	Seismic	EQ_ASCE716_0.3X_-E_-Y_+E_F
E27	Seismic	EQ_ASCE716_-0.3X_-E_Y_+E_F
E28	Seismic	EQ_ASCE716_-0.3X_-E_-Y_+E_F
E29	Seismic	EQ_ASCE716_X_-E_0.3Y_-E_F
E30	Seismic	EQ_ASCE716_X_-E_-0.3Y_-E_F
E31	Seismic	EQ_ASCE716_-X_-E_0.3Y_-E_F
E32	Seismic	EQ_ASCE716_-X_-E_-0.3Y_-E_F
E33	Seismic	EQ_ASCE716_0.3X_-E_Y_-E_F
E34	Seismic	EQ_ASCE716_0.3X_-E_-Y_-E_F
E35	Seismic	EQ_ASCE716_-0.3X_-E_Y_-E_F
E36	Seismic	EQ_ASCE716_-0.3X_-E_-Y_-E_F
E37	Fpx_min X	EQ_User
E38	Fpx_min Y	EQ_User
E39	Fpx X	EQ_User
E40	Fpx Y	EQ_User

Frame #1:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
215	D	-0.00	-0.00	35.64	0.00	0.00	0.00
	Lp	-0.00	-0.00	29.69	0.00	0.00	0.00
	Ln	0.00	0.00	-0.05	0.00	0.00	0.00
	Sp	-0.00	-0.00	7.06	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-0.00	-0.00	-33.34	0.00	0.00	0.00
	E2	-0.00	0.00	-16.92	0.00	0.00	0.00
	E3	0.00	0.00	2.14	0.00	0.00	0.00
	E4	-0.00	-0.00	-3.20	0.00	0.00	0.00
	E5	-0.00	-0.00	-32.70	0.00	0.00	0.00
	E6	-0.00	-0.00	-33.98	0.00	0.00	0.00
	E7	0.00	0.00	33.98	0.00	0.00	0.00
	E8	0.00	0.00	32.70	0.00	0.00	0.00

Frame Reactions



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Bentley Building Code: IBC

	E9	-0.00	-0.00	-7.86	0.00	0.00	0.00
	E10	-0.00	-0.00	-12.14	0.00	0.00	0.00
	E11	0.00	0.00	12.14	0.00	0.00	0.00
	E12	0.00	0.00	7.86	0.00	0.00	0.00
	E13	-0.00	-0.00	-34.30	0.00	0.00	0.00
	E14	-0.00	-0.00	-32.38	0.00	0.00	0.00
	E15	0.00	0.00	32.38	0.00	0.00	0.00
	E16	0.00	0.00	34.30	0.00	0.00	0.00
	E17	-0.00	-0.00	-13.20	0.00	0.00	0.00
	E18	-0.00	0.00	-6.80	0.00	0.00	0.00
	E19	0.00	-0.00	6.80	0.00	0.00	0.00
	E20	0.00	0.00	13.20	0.00	0.00	0.00
	E21	-0.00	0.00	-16.27	0.00	0.00	0.00
	E22	-0.00	0.00	-17.56	0.00	0.00	0.00
	E23	0.00	-0.00	17.56	0.00	0.00	0.00
	E24	0.00	-0.00	16.27	0.00	0.00	0.00
	E25	-0.00	0.00	-2.93	0.00	0.00	0.00
	E26	-0.00	0.00	-7.22	0.00	0.00	0.00
	E27	0.00	-0.00	7.22	0.00	0.00	0.00
	E28	0.00	-0.00	2.93	0.00	0.00	0.00
	E29	-0.00	0.00	-17.88	0.00	0.00	0.00
	E30	-0.00	0.00	-15.96	0.00	0.00	0.00
	E31	0.00	-0.00	15.96	0.00	0.00	0.00
	E32	0.00	-0.00	17.88	0.00	0.00	0.00
	E33	-0.00	-0.00	-8.27	0.00	0.00	0.00
	E34	-0.00	0.00	-1.88	0.00	0.00	0.00
	E35	0.00	-0.00	1.88	0.00	0.00	0.00
	E36	0.00	0.00	8.27	0.00	0.00	0.00
	E37	-0.00	-0.00	-40.39	0.00	0.00	0.00
	E38	-0.00	-0.00	-0.82	0.00	0.00	0.00
	E39	-0.00	-0.00	-24.49	0.00	0.00	0.00
	E40	-0.00	-0.00	-0.50	0.00	0.00	0.00
225	D	-0.00	-0.00	63.60	0.00	0.00	0.00
	Lp	0.00	-0.00	52.89	0.00	0.00	0.00
	Ln	0.00	0.00	-3.51	0.00	0.00	0.00
	Sp	-0.00	-0.00	22.76	0.00	0.00	0.00
	Sn	0.00	0.00	-0.02	0.00	0.00	0.00
	E1	0.00	-0.00	33.34	0.00	0.00	0.00
	E2	0.00	0.00	16.92	0.00	0.00	0.00

Frame Reactions



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E3	0.00	0.00	-2.14	0.00	0.00	0.00
E4	-0.00	-0.00	3.20	0.00	0.00	0.00
E5	-0.00	-0.00	32.70	0.00	0.00	0.00
E6	-0.00	-0.00	33.98	0.00	0.00	0.00
E7	-0.00	0.00	-33.98	0.00	0.00	0.00
E8	-0.00	0.00	-32.70	0.00	0.00	0.00
E9	0.00	-0.00	7.86	0.00	0.00	0.00
E10	-0.00	-0.00	12.14	0.00	0.00	0.00
E11	-0.00	0.00	-12.14	0.00	0.00	0.00
E12	-0.00	0.00	-7.86	0.00	0.00	0.00
E13	-0.00	-0.00	34.30	0.00	0.00	0.00
E14	0.00	-0.00	32.38	0.00	0.00	0.00
E15	0.00	0.00	-32.38	0.00	0.00	0.00
E16	0.00	0.00	-34.30	0.00	0.00	0.00
E17	0.00	-0.00	13.20	0.00	0.00	0.00
E18	0.00	0.00	6.80	0.00	0.00	0.00
E19	-0.00	-0.00	-6.80	0.00	0.00	0.00
E20	0.00	0.00	-13.20	0.00	0.00	0.00
E21	0.00	0.00	16.27	0.00	0.00	0.00
E22	0.00	0.00	17.56	0.00	0.00	0.00
E23	-0.00	-0.00	-17.56	0.00	0.00	0.00
E24	-0.00	-0.00	-16.27	0.00	0.00	0.00
E25	0.00	0.00	2.93	0.00	0.00	0.00
E26	-0.00	0.00	7.22	0.00	0.00	0.00
E27	0.00	-0.00	-7.22	0.00	0.00	0.00
E28	-0.00	-0.00	-2.93	0.00	0.00	0.00
E29	-0.00	0.00	17.88	0.00	0.00	0.00
E30	-0.00	0.00	15.96	0.00	0.00	0.00
E31	-0.00	-0.00	-15.96	0.00	0.00	0.00
E32	0.00	-0.00	-17.88	0.00	0.00	0.00
E33	-0.00	-0.00	8.27	0.00	0.00	0.00
E34	0.00	0.00	1.88	0.00	0.00	0.00
E35	-0.00	-0.00	-1.88	0.00	0.00	0.00
E36	0.00	0.00	-8.27	0.00	0.00	0.00
E37	-0.00	-0.00	40.39	0.00	0.00	0.00
E38	-0.00	-0.00	0.82	0.00	0.00	0.00
E39	0.00	-0.00	24.49	0.00	0.00	0.00
E40	-0.00	-0.00	0.50	0.00	0.00	0.00

Frame #2:

Frame Reactions



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Node	LdC	Rx	Ry	Rz	Mxx	Myy	Tzz
		kips	kips	kips	kip-ft	kip-ft	kip-ft
239	D	6.91	0.00	80.49	0.00	0.00	0.00
	Lp	6.92	0.00	50.10	0.00	0.00	0.00
	Ln	-0.00	-0.00	-2.73	0.00	0.00	0.00
	Sp	0.40	0.00	36.66	0.00	0.00	0.00
	Sn	0.00	-0.00	0.00	0.00	0.00	0.00
	E1	-113.55	0.00	-189.23	0.00	0.00	0.00
	E2	-66.26	-0.00	-110.42	0.00	0.00	0.00
	E3	5.10	-0.00	8.50	0.00	0.00	0.00
	E4	-12.97	0.00	-21.62	0.00	0.00	0.00
	E5	-112.02	0.00	-186.67	0.00	0.00	0.00
	E6	-115.08	0.00	-191.78	0.00	0.00	0.00
	E7	115.08	-0.00	191.78	0.00	0.00	0.00
	E8	112.02	-0.00	186.67	0.00	0.00	0.00
	E9	-28.96	0.00	-48.27	0.00	0.00	0.00
	E10	-39.17	0.00	-65.27	0.00	0.00	0.00
	E11	39.17	-0.00	65.27	0.00	0.00	0.00
	E12	28.96	-0.00	48.27	0.00	0.00	0.00
	E13	-117.45	0.00	-195.71	0.00	0.00	0.00
	E14	-109.66	0.00	-182.74	0.00	0.00	0.00
	E15	109.66	-0.00	182.74	0.00	0.00	0.00
	E16	117.45	-0.00	195.71	0.00	0.00	0.00
	E17	-47.04	0.00	-78.39	0.00	0.00	0.00
	E18	-21.09	-0.00	-35.15	0.00	0.00	0.00
	E19	21.09	0.00	35.15	0.00	0.00	0.00
	E20	47.04	-0.00	78.39	0.00	0.00	0.00
	E21	-64.73	-0.00	-107.87	0.00	0.00	0.00
	E22	-67.79	-0.00	-112.97	0.00	0.00	0.00
	E23	67.79	0.00	112.97	0.00	0.00	0.00
	E24	64.73	0.00	107.87	0.00	0.00	0.00
	E25	-14.78	-0.00	-24.62	0.00	0.00	0.00
	E26	-24.98	-0.00	-41.63	0.00	0.00	0.00
	E27	24.98	0.00	41.63	0.00	0.00	0.00
	E28	14.78	0.00	24.62	0.00	0.00	0.00
	E29	-70.16	-0.00	-116.91	0.00	0.00	0.00
	E30	-62.37	-0.00	-103.94	0.00	0.00	0.00
	E31	62.37	0.00	103.94	0.00	0.00	0.00
	E32	70.16	0.00	116.91	0.00	0.00	0.00

Frame Reactions



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	E33	-32.85	0.00	-54.75	0.00	0.00	0.00
	E34	-6.90	-0.00	-11.51	0.00	0.00	0.00
	E35	6.90	0.00	11.51	0.00	0.00	0.00
	E36	32.85	-0.00	54.75	0.00	0.00	0.00
	E37	-217.68	0.00	-362.74	0.00	0.00	0.00
	E38	-7.56	0.00	-12.59	0.00	0.00	0.00
	E39	-102.85	0.00	-171.39	0.00	0.00	0.00
	E40	-3.91	0.00	-6.51	0.00	0.00	0.00
252	D	-8.01	0.00	50.17	0.00	0.00	0.00
	Lp	-6.95	-0.00	42.25	0.00	0.00	0.00
	Ln	0.00	-0.00	-0.00	0.00	0.00	-0.00
	Sp	-2.01	0.00	4.19	0.00	0.00	0.00
	Sn	0.00	-0.00	-0.00	0.00	0.00	-0.00
	E1	-113.55	0.00	189.23	0.00	0.00	0.00
	E2	-66.26	-0.00	110.42	0.00	0.00	-0.00
	E3	5.10	-0.00	-8.50	0.00	0.00	-0.00
	E4	-12.97	-0.00	21.62	0.00	0.00	0.00
	E5	-112.02	-0.00	186.67	0.00	0.00	0.00
	E6	-115.08	0.00	191.78	0.00	0.00	0.00
	E7	115.08	-0.00	-191.78	0.00	0.00	-0.00
	E8	112.02	0.00	-186.67	0.00	0.00	-0.00
	E9	-28.96	-0.00	48.27	0.00	0.00	0.00
	E10	-39.17	0.00	65.27	0.00	0.00	0.00
	E11	39.17	-0.00	-65.27	0.00	0.00	-0.00
	E12	28.96	0.00	-48.27	0.00	0.00	-0.00
	E13	-117.45	-0.00	195.71	0.00	0.00	0.00
	E14	-109.66	0.00	182.74	0.00	0.00	0.00
	E15	109.66	-0.00	-182.74	0.00	0.00	-0.00
	E16	117.45	0.00	-195.71	0.00	0.00	-0.00
	E17	-47.04	-0.00	78.39	0.00	0.00	0.00
	E18	-21.09	0.00	35.15	0.00	0.00	-0.00
	E19	21.09	-0.00	-35.15	0.00	0.00	0.00
	E20	47.04	0.00	-78.39	0.00	0.00	-0.00
	E21	-64.73	-0.00	107.87	0.00	0.00	-0.00
	E22	-67.79	0.00	112.97	0.00	0.00	-0.00
	E23	67.79	-0.00	-112.97	0.00	0.00	0.00
	E24	64.73	0.00	-107.87	0.00	0.00	0.00
	E25	-14.78	-0.00	24.62	0.00	0.00	-0.00
	E26	-24.98	0.00	41.63	0.00	0.00	-0.00

Frame Reactions



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E27	24.98	-0.00	-41.63	0.00	0.00	0.00
E28	14.78	0.00	-24.62	0.00	0.00	0.00
E29	-70.16	-0.00	116.91	0.00	0.00	-0.00
E30	-62.37	0.00	103.94	0.00	0.00	-0.00
E31	62.37	-0.00	-103.94	0.00	0.00	0.00
E32	70.16	0.00	-116.91	0.00	0.00	0.00
E33	-32.85	-0.00	54.75	0.00	0.00	0.00
E34	-6.90	0.00	11.51	0.00	0.00	-0.00
E35	6.90	-0.00	-11.51	0.00	0.00	0.00
E36	32.85	0.00	-54.75	0.00	0.00	-0.00
E37	-217.68	0.00	362.74	0.00	0.00	0.00
E38	-7.56	-0.00	12.59	0.00	0.00	0.00
E39	-102.85	0.00	171.39	0.00	0.00	0.00
E40	-3.91	-0.00	6.51	0.00	0.00	0.00

Frame #3:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
212	D	4.37	-0.00	44.37	0.00	0.00	0.00
	Lp	5.80	-0.00	38.73	0.00	0.00	0.00
	Ln	0.00	0.00	0.02	0.00	0.00	0.00
	Sp	-0.25	-0.00	13.08	0.00	0.00	0.00
	Sn	0.00	0.00	0.00	0.00	0.00	0.00
	E1	-55.37	-0.00	-134.77	0.00	0.00	0.00
	E2	-40.43	0.00	-92.53	0.00	0.00	0.00
	E3	1.61	0.00	2.18	0.00	0.00	0.00
	E4	-4.10	-0.00	-11.35	0.00	0.00	0.00
	E5	-54.89	-0.00	-134.12	0.00	0.00	0.00
	E6	-55.86	-0.00	-135.43	0.00	0.00	0.00
	E7	55.86	0.00	135.43	0.00	0.00	0.00
	E8	54.89	0.00	134.12	0.00	0.00	0.00
	E9	-15.00	-0.00	-38.25	0.00	0.00	0.00
	E10	-18.23	-0.00	-42.62	0.00	0.00	0.00
	E11	18.23	0.00	42.62	0.00	0.00	0.00
	E12	15.00	0.00	38.25	0.00	0.00	0.00
	E13	-56.60	-0.00	-138.18	0.00	0.00	0.00
	E14	-54.14	-0.00	-131.37	0.00	0.00	0.00
	E15	54.14	0.00	131.37	0.00	0.00	0.00
	E16	56.60	0.00	138.18	0.00	0.00	0.00
	E17	-20.71	-0.00	-51.78	0.00	0.00	0.00

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	E18	-12.51	0.00	-29.08	0.00	0.00	0.00
	E19	12.51	-0.00	29.08	0.00	0.00	0.00
	E20	20.71	0.00	51.78	0.00	0.00	0.00
	E21	-39.94	0.00	-91.87	0.00	0.00	0.00
	E22	-40.91	0.00	-93.18	0.00	0.00	0.00
	E23	40.91	-0.00	93.18	0.00	0.00	0.00
	E24	39.94	-0.00	91.87	0.00	0.00	0.00
	E25	-10.52	0.00	-25.57	0.00	0.00	0.00
	E26	-13.74	0.00	-29.94	0.00	0.00	0.00
	E27	13.74	-0.00	29.94	0.00	0.00	0.00
	E28	10.52	-0.00	25.57	0.00	0.00	0.00
	E29	-41.66	0.00	-95.93	0.00	0.00	0.00
	E30	-39.20	0.00	-89.12	0.00	0.00	0.00
	E31	39.20	-0.00	89.12	0.00	0.00	0.00
	E32	41.66	-0.00	95.93	0.00	0.00	0.00
	E33	-16.23	-0.00	-39.11	0.00	0.00	0.00
	E34	-8.03	0.00	-16.41	0.00	0.00	0.00
	E35	8.03	-0.00	16.41	0.00	0.00	0.00
	E36	16.23	0.00	39.11	0.00	0.00	0.00
	E37	-115.32	-0.00	-246.65	0.00	0.00	0.00
	E38	-2.39	-0.00	-7.94	0.00	0.00	0.00
	E39	-54.68	-0.00	-124.11	0.00	0.00	0.00
	E40	-1.23	-0.00	-4.43	0.00	0.00	0.00
224	D	-4.70	0.00	70.36	0.00	0.00	0.00
	Lp	-5.63	0.00	70.04	0.00	0.00	0.00
	Ln	0.00	-0.00	-0.02	0.00	0.00	0.00
	Sp	-0.45	0.00	23.17	0.00	0.00	0.00
	Sn	0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	-55.28	0.00	134.76	0.00	0.00	0.00
	E2	-40.36	-0.00	92.52	0.00	0.00	0.00
	E3	1.61	-0.00	-2.18	0.00	0.00	0.00
	E4	-4.09	0.00	11.35	0.00	0.00	0.00
	E5	-54.79	0.00	134.10	0.00	0.00	0.00
	E6	-55.76	0.00	135.41	0.00	0.00	0.00
	E7	55.76	-0.00	-135.41	0.00	0.00	0.00
	E8	54.79	-0.00	-134.10	0.00	0.00	0.00
	E9	-14.97	0.00	38.24	0.00	0.00	0.00
	E10	-18.19	0.00	42.61	0.00	0.00	0.00
	E11	18.19	-0.00	-42.61	0.00	0.00	0.00

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E12	14.97	-0.00	-38.24	0.00	0.00	0.00
E13	-56.50	0.00	138.16	0.00	0.00	0.00
E14	-54.05	0.00	131.35	0.00	0.00	0.00
E15	54.05	-0.00	-131.35	0.00	0.00	0.00
E16	56.50	-0.00	-138.16	0.00	0.00	0.00
E17	-20.67	0.00	51.78	0.00	0.00	0.00
E18	-12.49	-0.00	29.08	0.00	0.00	0.00
E19	12.49	0.00	-29.08	0.00	0.00	0.00
E20	20.67	-0.00	-51.78	0.00	0.00	0.00
E21	-39.87	-0.00	91.87	0.00	0.00	0.00
E22	-40.84	-0.00	93.18	0.00	0.00	0.00
E23	40.84	0.00	-93.18	0.00	0.00	0.00
E24	39.87	0.00	-91.87	0.00	0.00	0.00
E25	-10.50	-0.00	25.57	0.00	0.00	0.00
E26	-13.72	-0.00	29.94	0.00	0.00	0.00
E27	13.72	0.00	-29.94	0.00	0.00	0.00
E28	10.50	0.00	-25.57	0.00	0.00	0.00
E29	-41.58	-0.00	95.93	0.00	0.00	0.00
E30	-39.13	-0.00	89.12	0.00	0.00	0.00
E31	39.13	0.00	-89.12	0.00	0.00	0.00
E32	41.58	0.00	-95.93	0.00	0.00	0.00
E33	-16.20	0.00	39.11	0.00	0.00	0.00
E34	-8.01	-0.00	16.41	0.00	0.00	0.00
E35	8.01	0.00	-16.41	0.00	0.00	0.00
E36	16.20	-0.00	-39.11	0.00	0.00	0.00
E37	-115.12	0.00	246.63	0.00	0.00	0.00
E38	-2.38	0.00	7.94	0.00	0.00	0.00
E39	-54.58	0.00	124.10	0.00	0.00	0.00
E40	-1.23	0.00	4.43	0.00	0.00	0.00

Frame #4:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
238	D	4.40	0.00	90.42	0.00	0.00	0.00
	Lp	5.75	0.00	70.00	0.00	0.00	0.00
	Ln	0.00	0.00	0.02	0.00	0.00	0.00
	Sp	-0.20	-0.00	22.20	0.00	0.00	0.00
	Sn	0.00	0.00	0.00	0.00	0.00	0.00
	E1	-47.49	-0.00	-123.21	0.00	0.00	0.00
	E2	-34.68	0.00	-84.08	0.00	0.00	0.00

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E3	1.26	0.00	-0.22	0.00	0.00	0.00
E4	-3.63	-0.00	-12.27	0.00	0.00	0.00
E5	-47.11	-0.00	-123.28	0.00	0.00	0.00
E6	-47.87	-0.00	-123.14	0.00	0.00	0.00
E7	47.87	0.00	123.14	0.00	0.00	0.00
E8	47.11	0.00	123.28	0.00	0.00	0.00
E9	-12.99	0.00	-37.19	0.00	0.00	0.00
E10	-15.51	-0.00	-36.74	0.00	0.00	0.00
E11	15.51	0.00	36.74	0.00	0.00	0.00
E12	12.99	-0.00	37.19	0.00	0.00	0.00
E13	-48.58	-0.00	-126.89	0.00	0.00	0.00
E14	-46.40	-0.00	-119.53	0.00	0.00	0.00
E15	46.40	0.00	119.53	0.00	0.00	0.00
E16	48.58	0.00	126.89	0.00	0.00	0.00
E17	-17.87	-0.00	-49.23	0.00	0.00	0.00
E18	-10.62	0.00	-24.69	0.00	0.00	0.00
E19	10.62	-0.00	24.69	0.00	0.00	0.00
E20	17.87	0.00	49.23	0.00	0.00	0.00
E21	-34.30	0.00	-84.15	0.00	0.00	0.00
E22	-35.06	0.00	-84.01	0.00	0.00	0.00
E23	35.06	-0.00	84.01	0.00	0.00	0.00
E24	34.30	-0.00	84.15	0.00	0.00	0.00
E25	-9.15	0.00	-25.45	0.00	0.00	0.00
E26	-11.66	-0.00	-25.00	0.00	0.00	0.00
E27	11.66	0.00	25.00	0.00	0.00	0.00
E28	9.15	-0.00	25.45	0.00	0.00	0.00
E29	-35.77	0.00	-87.76	0.00	0.00	0.00
E30	-33.59	0.00	-80.40	0.00	0.00	0.00
E31	33.59	-0.00	80.40	0.00	0.00	0.00
E32	35.77	-0.00	87.76	0.00	0.00	0.00
E33	-14.03	-0.00	-37.49	0.00	0.00	0.00
E34	-6.78	0.00	-12.96	0.00	0.00	0.00
E35	6.78	-0.00	12.96	0.00	0.00	0.00
E36	14.03	0.00	37.49	0.00	0.00	0.00
E37	-98.92	-0.00	-221.55	0.00	0.00	0.00
E38	-2.23	-0.00	-10.44	0.00	0.00	0.00
E39	-46.90	-0.00	-112.48	0.00	0.00	0.00
E40	-1.17	-0.00	-5.98	0.00	0.00	0.00
D	-4.81	0.01	67.86	0.00	0.00	0.00

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Lp	-5.76	0.01	72.67	0.00	0.00	0.00
Ln	-0.00	0.00	-0.03	0.00	0.00	0.00
Sp	-0.41	-0.00	16.75	0.00	0.00	0.00
Sn	-0.00	0.00	-0.00	0.00	0.00	0.00
E1	-47.52	-0.01	124.31	0.00	0.00	0.00
E2	-34.71	-0.02	81.53	0.00	0.00	0.00
E3	1.52	0.01	-46.44	0.00	0.00	0.00
E4	-3.40	-0.02	-29.52	0.00	0.00	0.00
E5	-47.07	-0.00	110.38	0.00	0.00	0.00
E6	-47.98	-0.01	138.24	0.00	0.00	0.00
E7	47.98	0.01	-138.24	0.00	0.00	0.00
E8	47.07	0.00	-110.38	0.00	0.00	0.00
E9	-12.74	0.01	-9.14	0.00	0.00	0.00
E10	-15.77	-0.02	83.73	0.00	0.00	0.00
E11	15.77	0.02	-83.73	0.00	0.00	0.00
E12	12.74	-0.01	9.14	0.00	0.00	0.00
E13	-48.54	-0.01	115.46	0.00	0.00	0.00
E14	-46.50	-0.00	133.17	0.00	0.00	0.00
E15	46.50	0.00	-133.17	0.00	0.00	0.00
E16	48.54	0.01	-115.46	0.00	0.00	0.00
E17	-17.66	-0.02	7.77	0.00	0.00	0.00
E18	-10.86	0.02	66.81	0.00	0.00	0.00
E19	10.86	-0.02	-66.81	0.00	0.00	0.00
E20	17.66	0.02	-7.77	0.00	0.00	0.00
E21	-34.26	-0.01	67.60	0.00	0.00	0.00
E22	-35.17	-0.02	95.46	0.00	0.00	0.00
E23	35.17	0.02	-95.46	0.00	0.00	0.00
E24	34.26	0.01	-67.60	0.00	0.00	0.00
E25	-8.90	0.01	-21.98	0.00	0.00	0.00
E26	-11.93	-0.02	70.89	0.00	0.00	0.00
E27	11.93	0.02	-70.89	0.00	0.00	0.00
E28	8.90	-0.01	21.98	0.00	0.00	0.00
E29	-35.73	-0.02	72.67	0.00	0.00	0.00
E30	-33.69	-0.01	90.39	0.00	0.00	0.00
E31	33.69	0.01	-90.39	0.00	0.00	0.00
E32	35.73	0.02	-72.67	0.00	0.00	0.00
E33	-13.82	-0.02	-5.06	0.00	0.00	0.00
E34	-7.01	0.01	53.98	0.00	0.00	0.00
E35	7.01	-0.01	-53.98	0.00	0.00	0.00

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E36	13.82	0.02	5.06	0.00	0.00	0.00
E37	-99.12	-0.02	220.34	0.00	0.00	0.00
E38	-1.86	-0.36	-58.61	0.00	0.00	0.00
E39	-46.96	-0.01	111.77	0.00	0.00	0.00
E40	-0.94	-0.07	-35.63	0.00	0.00	0.00

Frame #6:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
236	D	0.68	0.01	73.37	0.00	0.00	0.00
	Lp	1.15	0.01	79.90	0.00	0.00	0.00
	Ln	0.00	0.00	0.01	0.00	0.00	0.00
	Sp	-0.14	0.00	29.24	0.00	0.00	0.00
	Sn	0.00	0.00	0.00	0.00	0.00	0.00
	E1	-38.27	0.00	-107.90	0.00	0.00	0.00
	E2	-32.12	-0.00	-85.05	0.00	0.00	0.00
	E3	0.66	0.06	2.95	0.00	0.00	0.00
	E4	-1.69	0.03	-3.55	0.00	0.00	0.00
	E5	-38.07	0.02	-107.01	0.00	0.00	0.00
	E6	-38.47	-0.02	-108.79	0.00	0.00	0.00
	E7	38.47	0.02	108.79	0.00	0.00	0.00
	E8	38.07	-0.02	107.01	0.00	0.00	0.00
	E9	-10.82	0.06	-29.42	0.00	0.00	0.00
	E10	-12.15	-0.06	-35.32	0.00	0.00	0.00
	E11	12.15	0.06	35.32	0.00	0.00	0.00
	E12	10.82	-0.06	29.42	0.00	0.00	0.00
	E13	-38.78	0.01	-108.97	0.00	0.00	0.00
	E14	-37.76	-0.01	-106.84	0.00	0.00	0.00
	E15	37.76	0.01	106.84	0.00	0.00	0.00
	E16	38.78	-0.01	108.97	0.00	0.00	0.00
	E17	-13.17	0.03	-35.92	0.00	0.00	0.00
	E18	-9.79	-0.03	-28.82	0.00	0.00	0.00
	E19	9.79	0.03	28.82	0.00	0.00	0.00
	E20	13.17	-0.03	35.92	0.00	0.00	0.00
	E21	-31.92	0.01	-84.17	0.00	0.00	0.00
	E22	-32.32	-0.02	-85.94	0.00	0.00	0.00
	E23	32.32	0.02	85.94	0.00	0.00	0.00
	E24	31.92	-0.01	84.17	0.00	0.00	0.00
	E25	-8.97	0.06	-22.56	0.00	0.00	0.00
	E26	-10.30	-0.06	-28.47	0.00	0.00	0.00

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	E27	10.30	0.06	28.47	0.00	0.00	0.00
	E28	8.97	-0.06	22.56	0.00	0.00	0.00
	E29	-32.63	0.01	-86.12	0.00	0.00	0.00
	E30	-31.61	-0.01	-83.99	0.00	0.00	0.00
	E31	31.61	0.01	83.99	0.00	0.00	0.00
	E32	32.63	-0.01	86.12	0.00	0.00	0.00
	E33	-11.33	0.03	-29.07	0.00	0.00	0.00
	E34	-7.94	-0.03	-21.97	0.00	0.00	0.00
	E35	7.94	0.03	21.97	0.00	0.00	0.00
	E36	11.33	-0.03	29.07	0.00	0.00	0.00
	E37	-84.49	0.00	-201.65	0.00	0.00	0.00
	E38	-0.99	-0.28	-0.87	0.00	0.00	0.00
	E39	-40.14	-0.00	-103.76	0.00	0.00	0.00
	E40	-0.51	-0.03	-0.34	0.00	0.00	0.00
247	D	-0.33	0.00	66.81	0.00	0.00	0.00
	Lp	-0.02	0.00	82.01	0.00	0.00	0.00
	Ln	-0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.46	-0.00	20.48	0.00	0.00	0.00
	Sn	-0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-76.26	-0.01	10.45	0.00	0.00	0.00
	E2	-64.01	-0.01	7.54	0.00	0.00	0.00
	E3	1.32	0.03	-0.17	0.00	0.00	0.00
	E4	-3.38	-0.00	0.41	0.00	0.00	0.00
	E5	-75.86	0.00	10.40	0.00	0.00	0.00
	E6	-76.65	-0.01	10.50	0.00	0.00	0.00
	E7	76.65	0.01	-10.50	0.00	0.00	0.00
	E8	75.86	-0.00	-10.40	0.00	0.00	0.00
	E9	-21.56	0.03	2.97	0.00	0.00	0.00
	E10	-24.19	-0.03	3.31	0.00	0.00	0.00
	E11	24.19	0.03	-3.31	0.00	0.00	0.00
	E12	21.56	-0.03	-2.97	0.00	0.00	0.00
	E13	-77.27	-0.01	10.58	0.00	0.00	0.00
	E14	-75.24	-0.00	10.33	0.00	0.00	0.00
	E15	75.24	0.00	-10.33	0.00	0.00	0.00
	E16	77.27	0.01	-10.58	0.00	0.00	0.00
	E17	-26.25	-0.01	3.54	0.00	0.00	0.00
	E18	-19.50	0.00	2.73	0.00	0.00	0.00
	E19	19.50	-0.00	-2.73	0.00	0.00	0.00
	E20	26.25	0.01	-3.54	0.00	0.00	0.00

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	E21	-63.61	-0.00	7.49	0.00	0.00	0.00
	E22	-64.40	-0.02	7.59	0.00	0.00	0.00
	E23	64.40	0.02	-7.59	0.00	0.00	0.00
	E24	63.61	0.00	-7.49	0.00	0.00	0.00
	E25	-17.88	0.02	2.09	0.00	0.00	0.00
	E26	-20.52	-0.03	2.43	0.00	0.00	0.00
	E27	20.52	0.03	-2.43	0.00	0.00	0.00
	E28	17.88	-0.02	-2.09	0.00	0.00	0.00
	E29	-65.02	-0.01	7.67	0.00	0.00	0.00
	E30	-63.00	-0.01	7.42	0.00	0.00	0.00
	E31	63.00	0.01	-7.42	0.00	0.00	0.00
	E32	65.02	0.01	-7.67	0.00	0.00	0.00
	E33	-22.58	-0.01	2.67	0.00	0.00	0.00
	E34	-15.83	0.00	1.86	0.00	0.00	0.00
	E35	15.83	-0.00	-1.86	0.00	0.00	0.00
	E36	22.58	0.01	-2.67	0.00	0.00	0.00
	E37	-168.43	-0.01	14.54	0.00	0.00	0.00
	E38	-1.97	-0.34	0.21	0.00	0.00	0.00
	E39	-79.99	-0.01	8.79	0.00	0.00	0.00
	E40	-1.02	-0.06	0.12	0.00	0.00	0.00
256	D	-0.97	0.01	60.90	0.00	0.00	0.00
	Lp	-1.15	0.00	72.33	0.00	0.00	0.00
	Ln	0.00	0.00	-0.01	0.00	0.00	0.00
	Sp	-0.29	-0.00	20.28	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-38.29	-0.01	97.45	0.00	0.00	0.00
	E2	-32.13	-0.02	77.51	0.00	0.00	0.00
	E3	0.66	0.02	-2.78	0.00	0.00	0.00
	E4	-1.69	-0.02	3.14	0.00	0.00	0.00
	E5	-38.09	-0.01	96.61	0.00	0.00	0.00
	E6	-38.49	-0.02	98.28	0.00	0.00	0.00
	E7	38.49	0.02	-98.28	0.00	0.00	0.00
	E8	38.09	0.01	-96.61	0.00	0.00	0.00
	E9	-10.82	0.02	26.45	0.00	0.00	0.00
	E10	-12.15	-0.02	32.02	0.00	0.00	0.00
	E11	12.15	0.02	-32.02	0.00	0.00	0.00
	E12	10.82	-0.02	-26.45	0.00	0.00	0.00
	E13	-38.80	-0.02	98.39	0.00	0.00	0.00
	E14	-37.78	-0.01	96.50	0.00	0.00	0.00

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E15	37.78	0.01	-96.50	0.00	0.00	0.00
E16	38.80	0.02	-98.39	0.00	0.00	0.00
E17	-13.18	-0.02	32.38	0.00	0.00	0.00
E18	-9.79	0.01	26.09	0.00	0.00	0.00
E19	9.79	-0.01	-26.09	0.00	0.00	0.00
E20	13.18	0.02	-32.38	0.00	0.00	0.00
E21	-31.93	-0.01	76.67	0.00	0.00	0.00
E22	-32.33	-0.02	78.34	0.00	0.00	0.00
E23	32.33	0.02	-78.34	0.00	0.00	0.00
E24	31.93	0.01	-76.67	0.00	0.00	0.00
E25	-8.98	0.01	20.47	0.00	0.00	0.00
E26	-10.30	-0.02	26.04	0.00	0.00	0.00
E27	10.30	0.02	-26.04	0.00	0.00	0.00
E28	8.98	-0.01	-20.47	0.00	0.00	0.00
E29	-32.64	-0.02	78.45	0.00	0.00	0.00
E30	-31.62	-0.01	76.57	0.00	0.00	0.00
E31	31.62	0.01	-76.57	0.00	0.00	0.00
E32	32.64	0.02	-78.45	0.00	0.00	0.00
E33	-11.33	-0.02	26.39	0.00	0.00	0.00
E34	-7.95	0.01	20.11	0.00	0.00	0.00
E35	7.95	-0.01	-20.11	0.00	0.00	0.00
E36	11.33	0.02	-26.39	0.00	0.00	0.00
E37	-84.52	-0.02	187.11	0.00	0.00	0.00
E38	-0.99	-0.35	0.66	0.00	0.00	0.00
E39	-40.15	-0.01	94.98	0.00	0.00	0.00
E40	-0.51	-0.07	0.23	0.00	0.00	0.00

Frame #7:

Node	LdC	Rx	Ry	Rz	Mxx	Myy	Tzz
		kips	kips	kips	kip-ft	kip-ft	kip-ft
195	D	0.60	0.00	37.17	0.00	0.00	0.00
	Lp	0.87	0.00	41.66	0.00	0.00	0.00
	Ln	0.00	-0.00	0.00	0.00	0.00	0.00
	Sp	-0.16	-0.00	10.29	0.00	0.00	0.00
	Sn	0.00	-0.00	0.00	0.00	0.00	0.00
	E1	-36.53	0.02	-60.86	0.00	0.00	0.00
	E2	-32.85	0.03	-54.69	0.00	0.00	0.00
	E3	0.39	0.01	0.67	0.00	0.00	0.00
	E4	-1.02	0.05	-1.69	0.00	0.00	0.00
	E5	-36.42	0.02	-60.66	0.00	0.00	0.00

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	E6	-36.65	0.02	-61.06	0.00	0.00	0.00
	E7	36.65	-0.02	61.06	0.00	0.00	0.00
	E8	36.42	-0.02	60.66	0.00	0.00	0.00
	E9	-10.57	0.01	-17.59	0.00	0.00	0.00
	E10	-11.35	-0.00	-18.92	0.00	0.00	0.00
	E11	11.35	0.00	18.92	0.00	0.00	0.00
	E12	10.57	-0.01	17.59	0.00	0.00	0.00
	E13	-36.84	0.03	-61.36	0.00	0.00	0.00
	E14	-36.23	0.00	-60.35	0.00	0.00	0.00
	E15	36.23	-0.00	60.35	0.00	0.00	0.00
	E16	36.84	-0.03	61.36	0.00	0.00	0.00
	E17	-11.98	0.05	-19.95	0.00	0.00	0.00
	E18	-9.94	-0.04	-16.57	0.00	0.00	0.00
	E19	9.94	0.04	16.57	0.00	0.00	0.00
	E20	11.98	-0.05	19.95	0.00	0.00	0.00
	E21	-32.73	0.03	-54.49	0.00	0.00	0.00
	E22	-32.96	0.02	-54.89	0.00	0.00	0.00
	E23	32.96	-0.02	54.89	0.00	0.00	0.00
	E24	32.73	-0.03	54.49	0.00	0.00	0.00
	E25	-9.46	0.01	-15.74	0.00	0.00	0.00
	E26	-10.24	0.00	-17.07	0.00	0.00	0.00
	E27	10.24	-0.00	17.07	0.00	0.00	0.00
	E28	9.46	-0.01	15.74	0.00	0.00	0.00
	E29	-33.15	0.04	-55.20	0.00	0.00	0.00
	E30	-32.54	0.01	-54.18	0.00	0.00	0.00
	E31	32.54	-0.01	54.18	0.00	0.00	0.00
	E32	33.15	-0.04	55.20	0.00	0.00	0.00
	E33	-10.87	0.05	-18.10	0.00	0.00	0.00
	E34	-8.83	-0.04	-14.72	0.00	0.00	0.00
	E35	8.83	0.04	14.72	0.00	0.00	0.00
	E36	10.87	-0.05	18.10	0.00	0.00	0.00
	E37	-83.20	0.03	-138.34	0.00	0.00	0.00
	E38	-0.60	-0.32	-0.98	0.00	0.00	0.00
	E39	-39.55	0.02	-65.82	0.00	0.00	0.00
	E40	-0.31	-0.05	-0.51	0.00	0.00	0.00
201	D	-0.88	0.00	54.94	0.00	0.00	0.00
	Lp	-0.88	0.01	42.12	0.00	0.00	0.00
	Ln	0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	-0.25	0.00	18.73	0.00	0.00	0.00

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Sn	0.00	-0.00	-0.00	0.00	0.00	0.00
E1	-36.54	0.01	60.86	0.00	0.00	0.00
E2	-32.85	0.01	54.69	0.00	0.00	0.00
E3	0.39	0.04	-0.67	0.00	0.00	0.00
E4	-1.02	0.06	1.69	0.00	0.00	0.00
E5	-36.42	0.02	60.66	0.00	0.00	0.00
E6	-36.65	0.00	61.06	0.00	0.00	0.00
E7	36.65	-0.00	-61.06	0.00	0.00	0.00
E8	36.42	-0.02	-60.66	0.00	0.00	0.00
E9	-10.57	0.04	17.59	0.00	0.00	0.00
E10	-11.35	-0.03	18.92	0.00	0.00	0.00
E11	11.35	0.03	-18.92	0.00	0.00	0.00
E12	10.57	-0.04	-17.59	0.00	0.00	0.00
E13	-36.84	0.03	61.36	0.00	0.00	0.00
E14	-36.23	-0.01	60.35	0.00	0.00	0.00
E15	36.23	0.01	-60.35	0.00	0.00	0.00
E16	36.84	-0.03	-61.36	0.00	0.00	0.00
E17	-11.98	0.07	19.95	0.00	0.00	0.00
E18	-9.94	-0.06	16.57	0.00	0.00	0.00
E19	9.94	0.06	-16.57	0.00	0.00	0.00
E20	11.98	-0.07	-19.95	0.00	0.00	0.00
E21	-32.73	0.03	54.49	0.00	0.00	0.00
E22	-32.97	0.00	54.89	0.00	0.00	0.00
E23	32.97	-0.00	-54.89	0.00	0.00	0.00
E24	32.73	-0.03	-54.49	0.00	0.00	0.00
E25	-9.46	0.04	15.74	0.00	0.00	0.00
E26	-10.25	-0.03	17.07	0.00	0.00	0.00
E27	10.25	0.03	-17.07	0.00	0.00	0.00
E28	9.46	-0.04	-15.74	0.00	0.00	0.00
E29	-33.15	0.03	55.20	0.00	0.00	0.00
E30	-32.54	-0.00	54.18	0.00	0.00	0.00
E31	32.54	0.00	-54.18	0.00	0.00	0.00
E32	33.15	-0.03	-55.20	0.00	0.00	0.00
E33	-10.87	0.07	18.10	0.00	0.00	0.00
E34	-8.84	-0.06	14.72	0.00	0.00	0.00
E35	8.84	0.06	-14.72	0.00	0.00	0.00
E36	10.87	-0.07	-18.10	0.00	0.00	0.00
E37	-83.20	0.02	138.33	0.00	0.00	0.00
E38	-0.60	-0.28	0.98	0.00	0.00	0.00

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E39	-39.55	0.01	65.81	0.00	0.00	0.00
E40	-0.31	-0.03	0.51	0.00	0.00	0.00

Frame #8:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
200	D	0.68	0.00	74.46	0.00	0.00	0.00
	Lp	1.09	0.00	79.99	0.00	0.00	0.00
	Ln	-0.00	-0.00	0.00	0.00	0.00	0.00
	Sp	-0.10	-0.00	29.69	0.00	0.00	0.00
	Sn	-0.00	-0.00	0.00	0.00	0.00	0.00
	E1	-35.36	0.01	-58.89	0.00	0.00	0.00
	E2	-34.04	0.02	-56.67	0.00	0.00	0.00
	E3	0.21	0.02	0.24	0.00	0.00	0.00
	E4	-0.29	0.05	-0.61	0.00	0.00	0.00
	E5	-35.29	0.02	-58.82	0.00	0.00	0.00
	E6	-35.42	0.01	-58.96	0.00	0.00	0.00
	E7	35.42	-0.01	58.96	0.00	0.00	0.00
	E8	35.29	-0.02	58.82	0.00	0.00	0.00
	E9	-10.40	0.02	-17.43	0.00	0.00	0.00
	E10	-10.82	-0.02	-17.91	0.00	0.00	0.00
	E11	10.82	0.02	17.91	0.00	0.00	0.00
	E12	10.40	-0.02	17.43	0.00	0.00	0.00
	E13	-35.44	0.03	-59.07	0.00	0.00	0.00
	E14	-35.27	-0.00	-58.71	0.00	0.00	0.00
	E15	35.27	0.00	58.71	0.00	0.00	0.00
	E16	35.44	-0.03	59.07	0.00	0.00	0.00
	E17	-10.89	0.05	-18.27	0.00	0.00	0.00
	E18	-10.32	-0.05	-17.06	0.00	0.00	0.00
	E19	10.32	0.05	17.06	0.00	0.00	0.00
	E20	10.89	-0.05	18.27	0.00	0.00	0.00
	E21	-33.98	0.03	-56.60	0.00	0.00	0.00
	E22	-34.10	0.01	-56.74	0.00	0.00	0.00
	E23	34.10	-0.01	56.74	0.00	0.00	0.00
	E24	33.98	-0.03	56.60	0.00	0.00	0.00
	E25	-10.00	0.03	-16.76	0.00	0.00	0.00
	E26	-10.42	-0.01	-17.24	0.00	0.00	0.00
	E27	10.42	0.01	17.24	0.00	0.00	0.00
	E28	10.00	-0.03	16.76	0.00	0.00	0.00
	E29	-34.13	0.04	-56.85	0.00	0.00	0.00

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	E30	-33.96	0.00	-56.49	0.00	0.00	0.00
	E31	33.96	-0.00	56.49	0.00	0.00	0.00
	E32	34.13	-0.04	56.85	0.00	0.00	0.00
	E33	-10.50	0.06	-17.61	0.00	0.00	0.00
	E34	-9.93	-0.05	-16.40	0.00	0.00	0.00
	E35	9.93	0.05	16.40	0.00	0.00	0.00
	E36	10.50	-0.06	17.61	0.00	0.00	0.00
	E37	-83.06	0.02	-138.10	0.00	0.00	0.00
	E38	-0.10	-0.30	-0.35	0.00	0.00	0.00
	E39	-39.53	0.01	-65.78	0.00	0.00	0.00
	E40	-0.04	-0.04	-0.18	0.00	0.00	0.00
208	D	-0.94	-10.27	62.06	0.00	0.00	0.00
	Lp	-1.10	-18.35	71.52	0.00	0.00	0.00
	Ln	0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.27	-0.30	20.49	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-35.34	-5.06	67.12	0.00	0.00	0.00
	E2	-34.00	5.53	46.23	0.00	0.00	0.00
	E3	0.07	-80.99	149.08	0.00	0.00	0.00
	E4	-0.44	-85.06	159.17	0.00	0.00	0.00
	E5	-35.32	-29.36	111.85	0.00	0.00	0.00
	E6	-35.36	19.23	22.40	0.00	0.00	0.00
	E7	35.36	-19.23	-22.40	0.00	0.00	0.00
	E8	35.32	29.36	-111.85	0.00	0.00	0.00
	E9	-10.53	-82.50	169.21	0.00	0.00	0.00
	E10	-10.67	79.47	-128.94	0.00	0.00	0.00
	E11	10.67	-79.47	128.94	0.00	0.00	0.00
	E12	10.53	82.50	-169.21	0.00	0.00	0.00
	E13	-35.47	-30.58	114.88	0.00	0.00	0.00
	E14	-35.21	20.46	19.37	0.00	0.00	0.00
	E15	35.21	-20.46	-19.37	0.00	0.00	0.00
	E16	35.47	30.58	-114.88	0.00	0.00	0.00
	E17	-11.04	-86.58	179.31	0.00	0.00	0.00
	E18	-10.16	83.54	-139.04	0.00	0.00	0.00
	E19	10.16	-83.54	139.04	0.00	0.00	0.00
	E20	11.04	86.58	-179.31	0.00	0.00	0.00
	E21	-33.98	-18.76	90.95	0.00	0.00	0.00
	E22	-34.02	29.83	1.51	0.00	0.00	0.00
	E23	34.02	-29.83	-1.51	0.00	0.00	0.00

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E24	33.98	18.76	-90.95	0.00	0.00	0.00
E25	-10.13	-79.33	162.94	0.00	0.00	0.00
E26	-10.27	82.65	-135.21	0.00	0.00	0.00
E27	10.27	-82.65	135.21	0.00	0.00	0.00
E28	10.13	79.33	-162.94	0.00	0.00	0.00
E29	-34.14	-19.99	93.98	0.00	0.00	0.00
E30	-33.87	31.05	-1.52	0.00	0.00	0.00
E31	33.87	-31.05	1.52	0.00	0.00	0.00
E32	34.14	19.99	-93.98	0.00	0.00	0.00
E33	-10.64	-83.40	173.04	0.00	0.00	0.00
E34	-9.76	86.72	-145.31	0.00	0.00	0.00
E35	9.76	-86.72	145.31	0.00	0.00	0.00
E36	10.64	83.40	-173.04	0.00	0.00	0.00
E37	-82.87	-0.28	137.17	0.00	0.00	0.00
E38	-0.33	-189.22	318.27	0.00	0.00	0.00
E39	-39.47	0.11	64.85	0.00	0.00	0.00
E40	-0.18	-90.35	160.95	0.00	0.00	0.00

Frame #9:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
234	D	0.67	0.00	72.46	0.00	0.00	0.00
	Lp	1.07	0.00	78.24	0.00	0.00	0.00
	Ln	0.00	0.00	0.00	0.00	0.00	0.00
	Sp	-0.09	0.00	29.41	0.00	0.00	0.00
	Sn	0.00	0.00	0.00	0.00	0.00	0.00
	E1	-32.96	-0.01	-86.68	0.00	0.00	0.00
	E2	-31.73	-0.01	-81.99	0.00	0.00	0.00
	E3	0.13	0.04	1.55	0.00	0.00	0.00
	E4	-0.34	0.02	-0.39	0.00	0.00	0.00
	E5	-32.92	0.00	-86.21	0.00	0.00	0.00
	E6	-33.00	-0.02	-87.14	0.00	0.00	0.00
	E7	33.00	0.02	87.14	0.00	0.00	0.00
	E8	32.92	-0.00	86.21	0.00	0.00	0.00
	E9	-9.76	0.04	-24.45	0.00	0.00	0.00
	E10	-10.02	-0.04	-27.56	0.00	0.00	0.00
	E11	10.02	0.04	27.56	0.00	0.00	0.00
	E12	9.76	-0.04	24.45	0.00	0.00	0.00
	E13	-33.06	-0.00	-86.80	0.00	0.00	0.00
	E14	-32.86	-0.02	-86.56	0.00	0.00	0.00

Frame Reactions



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	E15	32.86	0.02	86.56	0.00	0.00	0.00
	E16	33.06	0.00	86.80	0.00	0.00	0.00
	E17	-10.23	0.02	-26.39	0.00	0.00	0.00
	E18	-9.55	-0.02	-25.61	0.00	0.00	0.00
	E19	9.55	0.02	25.61	0.00	0.00	0.00
	E20	10.23	-0.02	26.39	0.00	0.00	0.00
	E21	-31.69	0.00	-81.52	0.00	0.00	0.00
	E22	-31.77	-0.02	-82.46	0.00	0.00	0.00
	E23	31.77	0.02	82.46	0.00	0.00	0.00
	E24	31.69	-0.00	81.52	0.00	0.00	0.00
	E25	-9.39	0.04	-23.04	0.00	0.00	0.00
	E26	-9.65	-0.04	-26.15	0.00	0.00	0.00
	E27	9.65	0.04	26.15	0.00	0.00	0.00
	E28	9.39	-0.04	23.04	0.00	0.00	0.00
	E29	-31.83	-0.00	-82.11	0.00	0.00	0.00
	E30	-31.63	-0.02	-81.87	0.00	0.00	0.00
	E31	31.63	0.02	81.87	0.00	0.00	0.00
	E32	31.83	0.00	82.11	0.00	0.00	0.00
	E33	-9.86	0.02	-24.99	0.00	0.00	0.00
	E34	-9.18	-0.02	-24.21	0.00	0.00	0.00
	E35	9.18	0.02	24.21	0.00	0.00	0.00
	E36	9.86	-0.02	24.99	0.00	0.00	0.00
	E37	-77.43	-0.01	-177.90	0.00	0.00	0.00
	E38	-0.20	-0.31	0.77	0.00	0.00	0.00
	E39	-36.85	-0.01	-91.06	0.00	0.00	0.00
	E40	-0.10	-0.04	0.52	0.00	0.00	0.00
245	D	-0.91	0.06	67.80	0.00	0.00	0.00
	Lp	-1.08	-0.05	83.09	0.00	0.00	0.00
	Ln	0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.25	0.14	20.60	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-32.96	9.78	84.90	0.00	0.00	0.00
	E2	-31.72	-9.94	80.50	0.00	0.00	0.00
	E3	0.13	-166.02	0.07	0.00	0.00	0.00
	E4	-0.34	-158.60	1.88	0.00	0.00	0.00
	E5	-32.92	-40.03	84.92	0.00	0.00	0.00
	E6	-33.00	59.58	84.88	0.00	0.00	0.00
	E7	33.00	-59.58	-84.88	0.00	0.00	0.00
	E8	32.92	40.03	-84.92	0.00	0.00	0.00

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E9	-9.76	-163.08	25.54	0.00	0.00	0.00
E10	-10.01	168.95	25.40	0.00	0.00	0.00
E11	10.01	-168.95	-25.40	0.00	0.00	0.00
E12	9.76	163.08	-25.54	0.00	0.00	0.00
E13	-33.06	-37.80	85.47	0.00	0.00	0.00
E14	-32.86	57.36	84.34	0.00	0.00	0.00
E15	32.86	-57.36	-84.34	0.00	0.00	0.00
E16	33.06	37.80	-85.47	0.00	0.00	0.00
E17	-10.23	-155.66	27.35	0.00	0.00	0.00
E18	-9.54	161.53	23.59	0.00	0.00	0.00
E19	9.54	-161.53	-23.59	0.00	0.00	0.00
E20	10.23	155.66	-27.35	0.00	0.00	0.00
E21	-31.69	-59.75	80.52	0.00	0.00	0.00
E22	-31.76	39.86	80.48	0.00	0.00	0.00
E23	31.76	-39.86	-80.48	0.00	0.00	0.00
E24	31.69	59.75	-80.52	0.00	0.00	0.00
E25	-9.39	-169.00	24.22	0.00	0.00	0.00
E26	-9.64	163.03	24.08	0.00	0.00	0.00
E27	9.64	-163.03	-24.08	0.00	0.00	0.00
E28	9.39	169.00	-24.22	0.00	0.00	0.00
E29	-31.83	-57.52	81.06	0.00	0.00	0.00
E30	-31.62	37.64	79.94	0.00	0.00	0.00
E31	31.62	-37.64	-79.94	0.00	0.00	0.00
E32	31.83	57.52	-81.06	0.00	0.00	0.00
E33	-9.86	-161.58	26.03	0.00	0.00	0.00
E34	-9.17	155.62	22.27	0.00	0.00	0.00
E35	9.17	-155.62	-22.27	0.00	0.00	0.00
E36	9.86	161.58	-26.03	0.00	0.00	0.00
E37	-77.42	1.10	175.26	0.00	0.00	0.00
E38	-0.21	-371.73	1.62	0.00	0.00	0.00
E39	-36.84	0.14	89.47	0.00	0.00	0.00
E40	-0.11	-177.08	0.93	0.00	0.00	0.00

Frame #10:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
207	D	0.76	10.12	67.34	0.00	0.00	0.00
	Lp	1.10	18.29	80.29	0.00	0.00	0.00
	Ln	0.00	0.00	0.01	0.00	0.00	0.00
	Sp	-0.08	0.14	20.10	0.00	0.00	0.00

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Sn	0.00	0.00	0.00	0.00	0.00	0.00
E1	-34.16	-5.51	-106.61	0.00	0.00	0.00
E2	-35.19	5.05	-92.60	0.00	0.00	0.00
E3	-0.18	-80.95	-146.23	0.00	0.00	0.00
E4	0.21	-85.04	-155.65	0.00	0.00	0.00
E5	-34.21	-29.79	-150.48	0.00	0.00	0.00
E6	-34.10	18.78	-62.74	0.00	0.00	0.00
E7	34.10	-18.78	62.74	0.00	0.00	0.00
E8	34.21	29.79	150.48	0.00	0.00	0.00
E9	-10.43	-82.60	-178.21	0.00	0.00	0.00
E10	-10.06	79.30	114.24	0.00	0.00	0.00
E11	10.06	-79.30	-114.24	0.00	0.00	0.00
E12	10.43	82.60	178.21	0.00	0.00	0.00
E13	-34.10	-31.02	-153.31	0.00	0.00	0.00
E14	-34.22	20.00	-59.92	0.00	0.00	0.00
E15	34.22	-20.00	59.92	0.00	0.00	0.00
E16	34.10	31.02	153.31	0.00	0.00	0.00
E17	-10.04	-86.69	-187.63	0.00	0.00	0.00
E18	-10.46	83.38	123.66	0.00	0.00	0.00
E19	10.46	-83.38	-123.66	0.00	0.00	0.00
E20	10.04	86.69	187.63	0.00	0.00	0.00
E21	-35.25	-19.23	-136.47	0.00	0.00	0.00
E22	-35.14	29.34	-48.73	0.00	0.00	0.00
E23	35.14	-29.34	48.73	0.00	0.00	0.00
E24	35.25	19.23	136.47	0.00	0.00	0.00
E25	-10.74	-79.43	-174.01	0.00	0.00	0.00
E26	-10.37	82.47	118.45	0.00	0.00	0.00
E27	10.37	-82.47	-118.45	0.00	0.00	0.00
E28	10.74	79.43	174.01	0.00	0.00	0.00
E29	-35.13	-20.46	-139.29	0.00	0.00	0.00
E30	-35.25	30.56	-45.90	0.00	0.00	0.00
E31	35.25	-30.56	45.90	0.00	0.00	0.00
E32	35.13	20.46	139.29	0.00	0.00	0.00
E33	-10.35	-83.52	-183.43	0.00	0.00	0.00
E34	-10.77	86.55	127.87	0.00	0.00	0.00
E35	10.77	-86.55	-127.87	0.00	0.00	0.00
E36	10.35	83.52	183.43	0.00	0.00	0.00
E37	-82.73	-1.02	-206.14	0.00	0.00	0.00
E38	0.05	-189.17	-313.29	0.00	0.00	0.00

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	E39	-39.45	-0.34	-106.70	0.00	0.00	0.00
	E40	0.02	-90.32	-157.96	0.00	0.00	0.00
220	D	-0.99	0.00	109.31	0.00	0.00	0.00
	Lp	-1.12	0.00	78.85	0.00	0.00	0.00
	Ln	0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	-0.25	0.00	29.72	0.00	0.00	0.00
	Sn	0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	-34.16	0.00	98.38	0.00	0.00	0.00
	E2	-35.20	-0.00	103.04	0.00	0.00	0.00
	E3	-0.04	-0.00	-3.09	0.00	0.00	0.00
	E4	0.37	0.00	-2.92	0.00	0.00	0.00
	E5	-34.17	0.00	97.45	0.00	0.00	0.00
	E6	-34.14	0.00	99.30	0.00	0.00	0.00
	E7	34.14	-0.00	-99.30	0.00	0.00	0.00
	E8	34.17	-0.00	-97.45	0.00	0.00	0.00
	E9	-10.29	0.00	26.42	0.00	0.00	0.00
	E10	-10.21	0.00	32.60	0.00	0.00	0.00
	E11	10.21	-0.00	-32.60	0.00	0.00	0.00
	E12	10.29	-0.00	-26.42	0.00	0.00	0.00
	E13	-34.05	0.00	97.50	0.00	0.00	0.00
	E14	-34.27	0.00	99.26	0.00	0.00	0.00
	E15	34.27	-0.00	-99.26	0.00	0.00	0.00
	E16	34.05	-0.00	-97.50	0.00	0.00	0.00
	E17	-9.88	0.00	26.59	0.00	0.00	0.00
	E18	-10.61	-0.00	32.44	0.00	0.00	0.00
	E19	10.61	0.00	-32.44	0.00	0.00	0.00
	E20	9.88	-0.00	-26.59	0.00	0.00	0.00
	E21	-35.22	-0.00	102.11	0.00	0.00	0.00
	E22	-35.19	-0.00	103.97	0.00	0.00	0.00
	E23	35.19	0.00	-103.97	0.00	0.00	0.00
	E24	35.22	0.00	-102.11	0.00	0.00	0.00
	E25	-10.60	-0.00	27.82	0.00	0.00	0.00
	E26	-10.52	-0.00	34.00	0.00	0.00	0.00
	E27	10.52	0.00	-34.00	0.00	0.00	0.00
	E28	10.60	0.00	-27.82	0.00	0.00	0.00
	E29	-35.09	-0.00	102.16	0.00	0.00	0.00
	E30	-35.31	-0.00	103.92	0.00	0.00	0.00
	E31	35.31	0.00	-103.92	0.00	0.00	0.00
	E32	35.09	0.00	-102.16	0.00	0.00	0.00

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E33	-10.19	0.00	27.99	0.00	0.00	0.00
E34	-10.93	-0.00	33.84	0.00	0.00	0.00
E35	10.93	0.00	-33.84	0.00	0.00	0.00
E36	10.19	-0.00	-27.99	0.00	0.00	0.00
E37	-82.74	0.00	207.07	0.00	0.00	0.00
E38	0.29	0.00	-4.63	0.00	0.00	0.00
E39	-39.45	0.00	107.63	0.00	0.00	0.00
E40	0.16	0.00	-2.81	0.00	0.00	0.00

Frame #11:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
199	D	0.75	-0.00	71.39	0.00	0.00	0.00
	Lp	1.12	-0.00	79.81	0.00	0.00	0.00
	Ln	0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.01	-0.00	27.94	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-21.82	-0.00	-69.13	0.00	0.00	0.00
	E2	-25.78	0.00	-86.01	0.00	0.00	0.00
	E3	-0.43	0.00	-1.03	0.00	0.00	0.00
	E4	1.09	-0.00	3.01	0.00	0.00	0.00
	E5	-21.94	-0.00	-69.44	0.00	0.00	0.00
	E6	-21.69	-0.00	-68.82	0.00	0.00	0.00
	E7	21.69	0.00	68.82	0.00	0.00	0.00
	E8	21.94	0.00	69.44	0.00	0.00	0.00
	E9	-6.97	-0.00	-21.77	0.00	0.00	0.00
	E10	-6.12	-0.00	-19.71	0.00	0.00	0.00
	E11	6.12	0.00	19.71	0.00	0.00	0.00
	E12	6.97	0.00	21.77	0.00	0.00	0.00
	E13	-21.49	-0.00	-68.23	0.00	0.00	0.00
	E14	-22.14	-0.00	-70.03	0.00	0.00	0.00
	E15	22.14	0.00	70.03	0.00	0.00	0.00
	E16	21.49	0.00	68.23	0.00	0.00	0.00
	E17	-5.45	-0.00	-17.73	0.00	0.00	0.00
	E18	-7.63	0.00	-23.75	0.00	0.00	0.00
	E19	7.63	-0.00	23.75	0.00	0.00	0.00
	E20	5.45	0.00	17.73	0.00	0.00	0.00
	E21	-25.91	0.00	-86.32	0.00	0.00	0.00
	E22	-25.65	0.00	-85.70	0.00	0.00	0.00
	E23	25.65	-0.00	85.70	0.00	0.00	0.00

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	E24	25.91	-0.00	86.32	0.00	0.00	0.00
	E25	-8.16	0.00	-26.83	0.00	0.00	0.00
	E26	-7.31	0.00	-24.77	0.00	0.00	0.00
	E27	7.31	-0.00	24.77	0.00	0.00	0.00
	E28	8.16	-0.00	26.83	0.00	0.00	0.00
	E29	-25.45	0.00	-85.10	0.00	0.00	0.00
	E30	-26.11	0.00	-86.91	0.00	0.00	0.00
	E31	26.11	-0.00	86.91	0.00	0.00	0.00
	E32	25.45	-0.00	85.10	0.00	0.00	0.00
	E33	-6.64	-0.00	-22.79	0.00	0.00	0.00
	E34	-8.82	0.00	-28.81	0.00	0.00	0.00
	E35	8.82	-0.00	28.81	0.00	0.00	0.00
	E36	6.64	0.00	22.79	0.00	0.00	0.00
	E37	-56.57	-0.00	-155.36	0.00	0.00	0.00
	E38	0.64	-0.00	1.70	0.00	0.00	0.00
	E39	-27.04	-0.00	-82.04	0.00	0.00	0.00
	E40	0.33	-0.00	0.95	0.00	0.00	0.00
206	D	-0.88	0.00	64.85	0.00	0.00	0.00
	Lp	-1.13	0.00	80.18	0.00	0.00	0.00
	Ln	0.00	-0.00	0.00	0.00	0.00	0.00
	Sp	-0.17	0.00	20.46	0.00	0.00	0.00
	Sn	0.00	-0.00	0.00	0.00	0.00	0.00
	E1	-21.82	0.00	69.13	0.00	0.00	0.00
	E2	-25.78	-0.00	86.01	0.00	0.00	0.00
	E3	-0.43	-0.00	1.03	0.00	0.00	0.00
	E4	1.09	0.00	-3.01	0.00	0.00	0.00
	E5	-21.94	0.00	69.44	0.00	0.00	0.00
	E6	-21.69	0.00	68.82	0.00	0.00	0.00
	E7	21.69	-0.00	-68.82	0.00	0.00	0.00
	E8	21.94	-0.00	-69.44	0.00	0.00	0.00
	E9	-6.97	0.00	21.77	0.00	0.00	0.00
	E10	-6.12	0.00	19.71	0.00	0.00	0.00
	E11	6.12	-0.00	-19.71	0.00	0.00	0.00
	E12	6.97	-0.00	-21.77	0.00	0.00	0.00
	E13	-21.49	0.00	68.23	0.00	0.00	0.00
	E14	-22.14	0.00	70.03	0.00	0.00	0.00
	E15	22.14	-0.00	-70.03	0.00	0.00	0.00
	E16	21.49	-0.00	-68.23	0.00	0.00	0.00
	E17	-5.45	0.00	17.73	0.00	0.00	0.00

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E18	-7.63	-0.00	23.75	0.00	0.00	0.00
E19	7.63	0.00	-23.75	0.00	0.00	0.00
E20	5.45	-0.00	-17.73	0.00	0.00	0.00
E21	-25.91	-0.00	86.32	0.00	0.00	0.00
E22	-25.65	-0.00	85.70	0.00	0.00	0.00
E23	25.65	0.00	-85.70	0.00	0.00	0.00
E24	25.91	0.00	-86.32	0.00	0.00	0.00
E25	-8.16	-0.00	26.83	0.00	0.00	0.00
E26	-7.31	-0.00	24.77	0.00	0.00	0.00
E27	7.31	0.00	-24.77	0.00	0.00	0.00
E28	8.16	0.00	-26.83	0.00	0.00	0.00
E29	-25.45	-0.00	85.10	0.00	0.00	0.00
E30	-26.11	-0.00	86.91	0.00	0.00	0.00
E31	26.11	0.00	-86.91	0.00	0.00	0.00
E32	25.45	0.00	-85.10	0.00	0.00	0.00
E33	-6.64	0.00	22.79	0.00	0.00	0.00
E34	-8.82	-0.00	28.81	0.00	0.00	0.00
E35	8.82	0.00	-28.81	0.00	0.00	0.00
E36	6.64	-0.00	-22.79	0.00	0.00	0.00
E37	-56.57	0.00	155.36	0.00	0.00	0.00
E38	0.64	0.00	-1.70	0.00	0.00	0.00
E39	-27.04	0.00	82.04	0.00	0.00	0.00
E40	0.33	0.00	-0.95	0.00	0.00	0.00

Frame #12:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
219	D	0.73	-0.00	64.60	0.00	0.00	0.00
	Lp	1.15	-0.00	79.99	0.00	0.00	0.00
	Ln	0.00	0.00	0.00	0.00	0.00	0.00
	Sp	-0.06	-0.00	19.78	0.00	0.00	0.00
	Sn	0.00	0.00	0.00	0.00	0.00	0.00
	E1	-31.80	-0.00	-52.98	0.00	0.00	0.00
	E2	-37.58	0.00	-62.62	0.00	0.00	0.00
	E3	-0.62	0.00	-1.04	0.00	0.00	0.00
	E4	1.59	-0.00	2.65	0.00	0.00	0.00
	E5	-31.98	-0.00	-53.29	0.00	0.00	0.00
	E6	-31.61	-0.00	-52.67	0.00	0.00	0.00
	E7	31.61	0.00	52.67	0.00	0.00	0.00
	E8	31.98	0.00	53.29	0.00	0.00	0.00

Frame Reactions



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	E9	-10.16	-0.00	-16.93	0.00	0.00	0.00
	E10	-8.92	-0.00	-14.86	0.00	0.00	0.00
	E11	8.92	0.00	14.86	0.00	0.00	0.00
	E12	10.16	0.00	16.93	0.00	0.00	0.00
	E13	-31.32	-0.00	-52.19	0.00	0.00	0.00
	E14	-32.27	-0.00	-53.78	0.00	0.00	0.00
	E15	32.27	0.00	53.78	0.00	0.00	0.00
	E16	31.32	0.00	52.19	0.00	0.00	0.00
	E17	-7.95	-0.00	-13.25	0.00	0.00	0.00
	E18	-11.13	0.00	-18.54	0.00	0.00	0.00
	E19	11.13	-0.00	18.54	0.00	0.00	0.00
	E20	7.95	0.00	13.25	0.00	0.00	0.00
	E21	-37.76	0.00	-62.93	0.00	0.00	0.00
	E22	-37.39	0.00	-62.31	0.00	0.00	0.00
	E23	37.39	-0.00	62.31	0.00	0.00	0.00
	E24	37.76	-0.00	62.93	0.00	0.00	0.00
	E25	-11.90	0.00	-19.82	0.00	0.00	0.00
	E26	-10.65	0.00	-17.75	0.00	0.00	0.00
	E27	10.65	-0.00	17.75	0.00	0.00	0.00
	E28	11.90	-0.00	19.82	0.00	0.00	0.00
	E29	-37.10	0.00	-61.82	0.00	0.00	0.00
	E30	-38.05	0.00	-63.41	0.00	0.00	0.00
	E31	38.05	-0.00	63.41	0.00	0.00	0.00
	E32	37.10	-0.00	61.82	0.00	0.00	0.00
	E33	-9.68	-0.00	-16.14	0.00	0.00	0.00
	E34	-12.86	0.00	-21.43	0.00	0.00	0.00
	E35	12.86	-0.00	21.43	0.00	0.00	0.00
	E36	9.68	0.00	16.14	0.00	0.00	0.00
	E37	-82.45	-0.00	-137.40	0.00	0.00	0.00
	E38	0.93	-0.00	1.54	0.00	0.00	0.00
	E39	-39.41	-0.00	-65.67	0.00	0.00	0.00
	E40	0.48	-0.00	0.80	0.00	0.00	0.00
228	D	-0.91	0.00	64.78	0.00	0.00	0.00
	Lp	-1.16	0.00	80.01	0.00	0.00	0.00
	Ln	0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	-0.20	0.00	20.22	0.00	0.00	0.00
	Sn	0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	-31.80	0.00	52.98	0.00	0.00	0.00
	E2	-37.58	-0.00	62.62	0.00	0.00	0.00

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E3	-0.62	-0.00	1.04	0.00	0.00	0.00
E4	1.59	0.00	-2.65	0.00	0.00	0.00
E5	-31.98	0.00	53.29	0.00	0.00	0.00
E6	-31.61	0.00	52.67	0.00	0.00	0.00
E7	31.61	-0.00	-52.67	0.00	0.00	0.00
E8	31.98	-0.00	-53.29	0.00	0.00	0.00
E9	-10.16	0.00	16.93	0.00	0.00	0.00
E10	-8.92	0.00	14.86	0.00	0.00	0.00
E11	8.92	-0.00	-14.86	0.00	0.00	0.00
E12	10.16	-0.00	-16.93	0.00	0.00	0.00
E13	-31.32	0.00	52.19	0.00	0.00	0.00
E14	-32.27	0.00	53.78	0.00	0.00	0.00
E15	32.27	-0.00	-53.78	0.00	0.00	0.00
E16	31.32	-0.00	-52.19	0.00	0.00	0.00
E17	-7.95	0.00	13.25	0.00	0.00	0.00
E18	-11.13	-0.00	18.54	0.00	0.00	0.00
E19	11.13	0.00	-18.54	0.00	0.00	0.00
E20	7.95	-0.00	-13.25	0.00	0.00	0.00
E21	-37.76	-0.00	62.93	0.00	0.00	0.00
E22	-37.39	-0.00	62.31	0.00	0.00	0.00
E23	37.39	0.00	-62.31	0.00	0.00	0.00
E24	37.76	0.00	-62.93	0.00	0.00	0.00
E25	-11.90	-0.00	19.82	0.00	0.00	0.00
E26	-10.65	-0.00	17.75	0.00	0.00	0.00
E27	10.65	0.00	-17.75	0.00	0.00	0.00
E28	11.90	0.00	-19.82	0.00	0.00	0.00
E29	-37.10	-0.00	61.82	0.00	0.00	0.00
E30	-38.05	-0.00	63.41	0.00	0.00	0.00
E31	38.05	0.00	-63.41	0.00	0.00	0.00
E32	37.10	0.00	-61.82	0.00	0.00	0.00
E33	-9.68	0.00	16.14	0.00	0.00	0.00
E34	-12.86	-0.00	21.43	0.00	0.00	0.00
E35	12.86	0.00	-21.43	0.00	0.00	0.00
E36	9.68	-0.00	-16.14	0.00	0.00	0.00
E37	-82.45	0.00	137.40	0.00	0.00	0.00
E38	0.93	0.00	-1.54	0.00	0.00	0.00
E39	-39.41	0.00	65.67	0.00	0.00	0.00
E40	0.48	0.00	-0.80	0.00	0.00	0.00

Frame #13:

Frame Reactions



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Node	LdC	Rx	Ry	Rz	Mxx	Myy	Tzz
		kips	kips	kips	kip-ft	kip-ft	kip-ft
243	D	0.74	-0.00	63.78	0.00	0.00	0.00
	Lp	1.12	-0.00	79.84	0.00	0.00	0.00
	Ln	0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.03	-0.00	19.93	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-21.82	-0.00	-65.76	0.00	0.00	0.00
	E2	-25.78	0.00	-81.69	0.00	0.00	0.00
	E3	-0.43	0.00	-0.91	0.00	0.00	0.00
	E4	1.09	-0.00	2.80	0.00	0.00	0.00
	E5	-21.94	-0.00	-66.04	0.00	0.00	0.00
	E6	-21.69	-0.00	-65.49	0.00	0.00	0.00
	E7	21.69	0.00	65.49	0.00	0.00	0.00
	E8	21.94	0.00	66.04	0.00	0.00	0.00
	E9	-6.97	-0.00	-20.64	0.00	0.00	0.00
	E10	-6.12	-0.00	-18.82	0.00	0.00	0.00
	E11	6.12	0.00	18.82	0.00	0.00	0.00
	E12	6.97	0.00	20.64	0.00	0.00	0.00
	E13	-21.49	-0.00	-64.92	0.00	0.00	0.00
	E14	-22.14	-0.00	-66.60	0.00	0.00	0.00
	E15	22.14	0.00	66.60	0.00	0.00	0.00
	E16	21.49	0.00	64.92	0.00	0.00	0.00
	E17	-5.45	-0.00	-16.93	0.00	0.00	0.00
	E18	-7.63	0.00	-22.53	0.00	0.00	0.00
	E19	7.63	-0.00	22.53	0.00	0.00	0.00
	E20	5.45	0.00	16.93	0.00	0.00	0.00
	E21	-25.91	0.00	-81.96	0.00	0.00	0.00
	E22	-25.65	0.00	-81.42	0.00	0.00	0.00
	E23	25.65	-0.00	81.42	0.00	0.00	0.00
	E24	25.91	-0.00	81.96	0.00	0.00	0.00
	E25	-8.16	0.00	-25.42	0.00	0.00	0.00
	E26	-7.31	0.00	-23.60	0.00	0.00	0.00
	E27	7.31	-0.00	23.60	0.00	0.00	0.00
	E28	8.16	-0.00	25.42	0.00	0.00	0.00
	E29	-25.45	0.00	-80.85	0.00	0.00	0.00
	E30	-26.11	0.00	-82.53	0.00	0.00	0.00
	E31	26.11	-0.00	82.53	0.00	0.00	0.00
	E32	25.45	-0.00	80.85	0.00	0.00	0.00

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	E33	-6.64	-0.00	-21.71	0.00	0.00	0.00
	E34	-8.82	0.00	-27.30	0.00	0.00	0.00
	E35	8.82	-0.00	27.30	0.00	0.00	0.00
	E36	6.64	0.00	21.71	0.00	0.00	0.00
	E37	-56.57	-0.00	-149.18	0.00	0.00	0.00
	E38	0.64	-0.00	1.60	0.00	0.00	0.00
	E39	-27.04	-0.00	-78.30	0.00	0.00	0.00
	E40	0.33	-0.00	0.90	0.00	0.00	0.00
255	D	-0.86	0.00	64.02	0.00	0.00	0.00
	Lp	-1.13	0.00	80.16	0.00	0.00	0.00
	Ln	0.00	-0.00	0.00	0.00	0.00	0.00
	Sp	-0.15	0.00	20.07	0.00	0.00	0.00
	Sn	0.00	-0.00	0.00	0.00	0.00	0.00
	E1	-21.82	0.00	65.76	0.00	0.00	0.00
	E2	-25.78	-0.00	81.69	0.00	0.00	0.00
	E3	-0.43	-0.00	0.91	0.00	0.00	0.00
	E4	1.09	0.00	-2.80	0.00	0.00	0.00
	E5	-21.94	0.00	66.04	0.00	0.00	0.00
	E6	-21.69	0.00	65.49	0.00	0.00	0.00
	E7	21.69	-0.00	-65.49	0.00	0.00	0.00
	E8	21.94	-0.00	-66.04	0.00	0.00	0.00
	E9	-6.97	0.00	20.64	0.00	0.00	0.00
	E10	-6.12	0.00	18.82	0.00	0.00	0.00
	E11	6.12	-0.00	-18.82	0.00	0.00	0.00
	E12	6.97	-0.00	-20.64	0.00	0.00	0.00
	E13	-21.49	0.00	64.92	0.00	0.00	0.00
	E14	-22.14	0.00	66.60	0.00	0.00	0.00
	E15	22.14	-0.00	-66.60	0.00	0.00	0.00
	E16	21.49	-0.00	-64.92	0.00	0.00	0.00
	E17	-5.45	0.00	16.93	0.00	0.00	0.00
	E18	-7.63	-0.00	22.53	0.00	0.00	0.00
	E19	7.63	0.00	-22.53	0.00	0.00	0.00
	E20	5.45	-0.00	-16.93	0.00	0.00	0.00
	E21	-25.91	-0.00	81.96	0.00	0.00	0.00
	E22	-25.65	-0.00	81.42	0.00	0.00	0.00
	E23	25.65	0.00	-81.42	0.00	0.00	0.00
	E24	25.91	0.00	-81.96	0.00	0.00	0.00
	E25	-8.16	-0.00	25.42	0.00	0.00	0.00
	E26	-7.31	-0.00	23.60	0.00	0.00	0.00

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E27	7.31	0.00	-23.60	0.00	0.00	0.00
E28	8.16	0.00	-25.42	0.00	0.00	0.00
E29	-25.45	-0.00	80.85	0.00	0.00	0.00
E30	-26.11	-0.00	82.53	0.00	0.00	0.00
E31	26.11	0.00	-82.53	0.00	0.00	0.00
E32	25.45	0.00	-80.85	0.00	0.00	0.00
E33	-6.64	0.00	21.71	0.00	0.00	0.00
E34	-8.82	-0.00	27.30	0.00	0.00	0.00
E35	8.82	0.00	-27.30	0.00	0.00	0.00
E36	6.64	-0.00	-21.71	0.00	0.00	0.00
E37	-56.57	0.00	149.18	0.00	0.00	0.00
E38	0.64	0.00	-1.60	0.00	0.00	0.00
E39	-27.04	0.00	78.30	0.00	0.00	0.00
E40	0.33	0.00	-0.90	0.00	0.00	0.00

Frame #14:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
242	D	0.73	-10.18	65.89	0.00	0.00	0.00
	Lp	1.11	-18.40	81.20	0.00	0.00	0.00
	Ln	-0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	-0.04	-0.14	19.82	0.00	0.00	0.00
	Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	-33.32	5.07	-94.47	0.00	0.00	0.00
	E2	-45.23	-4.71	-115.93	0.00	0.00	0.00
	E3	-1.23	-83.16	135.47	0.00	0.00	0.00
	E4	3.32	-79.44	138.80	0.00	0.00	0.00
	E5	-33.69	-19.88	-53.83	0.00	0.00	0.00
	E6	-32.95	30.02	-135.11	0.00	0.00	0.00
	E7	32.95	-30.02	135.11	0.00	0.00	0.00
	E8	33.69	19.88	53.83	0.00	0.00	0.00
	E9	-11.23	-81.64	107.12	0.00	0.00	0.00
	E10	-8.76	84.68	-163.81	0.00	0.00	0.00
	E11	8.76	-84.68	163.81	0.00	0.00	0.00
	E12	11.23	81.64	-107.12	0.00	0.00	0.00
	E13	-32.32	-18.76	-52.83	0.00	0.00	0.00
	E14	-34.31	28.90	-136.11	0.00	0.00	0.00
	E15	34.31	-28.90	136.11	0.00	0.00	0.00
	E16	32.32	18.76	52.83	0.00	0.00	0.00
	E17	-6.68	-77.92	110.46	0.00	0.00	0.00

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	E18	-13.31	80.96	-167.14	0.00	0.00	0.00
	E19	13.31	-80.96	167.14	0.00	0.00	0.00
	E20	6.68	77.92	-110.46	0.00	0.00	0.00
	E21	-45.60	-29.66	-75.29	0.00	0.00	0.00
	E22	-44.86	20.23	-156.57	0.00	0.00	0.00
	E23	44.86	-20.23	156.57	0.00	0.00	0.00
	E24	45.60	29.66	75.29	0.00	0.00	0.00
	E25	-14.80	-84.58	100.68	0.00	0.00	0.00
	E26	-12.34	81.75	-170.25	0.00	0.00	0.00
	E27	12.34	-81.75	170.25	0.00	0.00	0.00
	E28	14.80	84.58	-100.68	0.00	0.00	0.00
	E29	-44.23	-28.55	-74.29	0.00	0.00	0.00
	E30	-46.22	19.12	-157.58	0.00	0.00	0.00
	E31	46.22	-19.12	157.58	0.00	0.00	0.00
	E32	44.23	28.55	74.29	0.00	0.00	0.00
	E33	-10.25	-80.86	104.02	0.00	0.00	0.00
	E34	-16.89	78.03	-173.58	0.00	0.00	0.00
	E35	16.89	-78.03	173.58	0.00	0.00	0.00
	E36	10.25	80.86	-104.02	0.00	0.00	0.00
	E37	-93.01	0.91	-219.90	0.00	0.00	0.00
	E38	1.99	-185.92	291.30	0.00	0.00	0.00
	E39	-44.56	0.28	-113.19	0.00	0.00	0.00
	E40	1.03	-88.64	144.89	0.00	0.00	0.00
254	D	-0.88	0.00	64.46	0.00	0.00	0.00
	Lp	-1.12	0.00	79.04	0.00	0.00	0.00
	Ln	0.00	0.00	0.00	0.00	0.00	0.00
	Sp	-0.17	0.00	20.11	0.00	0.00	0.00
	Sn	0.00	0.00	0.00	0.00	0.00	0.00
	E1	-33.37	-0.01	86.35	0.00	0.00	0.00
	E2	-45.28	-0.05	122.92	0.00	0.00	0.00
	E3	-1.34	-0.00	4.37	0.00	0.00	0.00
	E4	3.23	-0.03	-6.79	0.00	0.00	0.00
	E5	-33.77	-0.01	87.66	0.00	0.00	0.00
	E6	-32.96	-0.01	85.04	0.00	0.00	0.00
	E7	32.96	0.01	-85.04	0.00	0.00	0.00
	E8	33.77	0.01	-87.66	0.00	0.00	0.00
	E9	-11.35	-0.00	30.27	0.00	0.00	0.00
	E10	-8.67	-0.00	21.54	0.00	0.00	0.00
	E11	8.67	0.00	-21.54	0.00	0.00	0.00

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E12	11.35	0.00	-30.27	0.00	0.00	0.00
E13	-32.40	-0.02	84.31	0.00	0.00	0.00
E14	-34.34	-0.00	88.39	0.00	0.00	0.00
E15	34.34	0.00	-88.39	0.00	0.00	0.00
E16	32.40	0.02	-84.31	0.00	0.00	0.00
E17	-6.78	-0.04	19.11	0.00	0.00	0.00
E18	-13.24	0.03	32.70	0.00	0.00	0.00
E19	13.24	-0.03	-32.70	0.00	0.00	0.00
E20	6.78	0.04	-19.11	0.00	0.00	0.00
E21	-45.68	-0.05	124.23	0.00	0.00	0.00
E22	-44.88	-0.05	121.61	0.00	0.00	0.00
E23	44.88	0.05	-121.61	0.00	0.00	0.00
E24	45.68	0.05	-124.23	0.00	0.00	0.00
E25	-14.92	-0.02	41.24	0.00	0.00	0.00
E26	-12.24	-0.01	32.51	0.00	0.00	0.00
E27	12.24	0.01	-32.51	0.00	0.00	0.00
E28	14.92	0.02	-41.24	0.00	0.00	0.00
E29	-44.31	-0.06	120.88	0.00	0.00	0.00
E30	-46.25	-0.04	124.96	0.00	0.00	0.00
E31	46.25	0.04	-124.96	0.00	0.00	0.00
E32	44.31	0.06	-120.88	0.00	0.00	0.00
E33	-10.35	-0.05	30.08	0.00	0.00	0.00
E34	-16.81	0.02	43.67	0.00	0.00	0.00
E35	16.81	-0.02	-43.67	0.00	0.00	0.00
E36	10.35	0.05	-30.08	0.00	0.00	0.00
E37	-93.24	-0.05	218.20	0.00	0.00	0.00
E38	1.83	-0.38	-2.52	0.00	0.00	0.00
E39	-44.64	-0.03	112.50	0.00	0.00	0.00
E40	0.94	-0.09	-1.24	0.00	0.00	0.00

Frame #15:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
197	D	2.13	0.00	59.39	0.00	0.00	0.00
	Lp	2.53	0.00	62.43	0.00	0.00	0.00
	Ln	-0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	-0.03	-0.00	17.51	0.00	0.00	0.00
	Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	-39.47	0.00	-91.51	0.00	0.00	0.00
	E2	-61.70	0.05	-147.16	0.00	0.00	0.00

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E3	-2.39	-0.01	-6.78	0.00	0.00	0.00
E4	6.11	0.04	13.77	0.00	0.00	0.00
E5	-40.18	0.00	-93.54	0.00	0.00	0.00
E6	-38.75	0.01	-89.47	0.00	0.00	0.00
E7	38.75	-0.01	89.47	0.00	0.00	0.00
E8	40.18	-0.00	93.54	0.00	0.00	0.00
E9	-14.23	-0.01	-34.23	0.00	0.00	0.00
E10	-9.44	0.01	-20.67	0.00	0.00	0.00
E11	9.44	-0.01	20.67	0.00	0.00	0.00
E12	14.23	0.01	34.23	0.00	0.00	0.00
E13	-37.63	0.02	-87.37	0.00	0.00	0.00
E14	-41.30	-0.01	-95.64	0.00	0.00	0.00
E15	41.30	0.01	95.64	0.00	0.00	0.00
E16	37.63	-0.02	87.37	0.00	0.00	0.00
E17	-5.73	0.04	-13.68	0.00	0.00	0.00
E18	-17.95	-0.04	-41.22	0.00	0.00	0.00
E19	17.95	0.04	41.22	0.00	0.00	0.00
E20	5.73	-0.04	13.68	0.00	0.00	0.00
E21	-62.42	0.05	-149.19	0.00	0.00	0.00
E22	-60.99	0.06	-145.13	0.00	0.00	0.00
E23	60.99	-0.06	145.13	0.00	0.00	0.00
E24	62.42	-0.05	149.19	0.00	0.00	0.00
E25	-20.91	0.01	-50.93	0.00	0.00	0.00
E26	-16.12	0.03	-37.37	0.00	0.00	0.00
E27	16.12	-0.03	37.37	0.00	0.00	0.00
E28	20.91	-0.01	50.93	0.00	0.00	0.00
E29	-59.87	0.07	-143.03	0.00	0.00	0.00
E30	-63.54	0.04	-151.29	0.00	0.00	0.00
E31	63.54	-0.04	151.29	0.00	0.00	0.00
E32	59.87	-0.07	143.03	0.00	0.00	0.00
E33	-12.40	0.06	-30.38	0.00	0.00	0.00
E34	-24.62	-0.02	-57.92	0.00	0.00	0.00
E35	24.62	0.02	57.92	0.00	0.00	0.00
E36	12.40	-0.06	30.38	0.00	0.00	0.00
E37	-119.45	0.04	-255.41	0.00	0.00	0.00
E38	3.57	-0.33	6.77	0.00	0.00	0.00
E39	-57.33	0.03	-129.70	0.00	0.00	0.00
E40	1.85	-0.06	3.49	0.00	0.00	0.00
203	D	-2.26	0.00	58.03	0.00	0.00

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Lp	-2.55	0.00	60.61	0.00	0.00	0.00
Ln	-0.00	-0.00	0.00	0.00	0.00	0.00
Sp	-0.15	-0.00	17.91	0.00	0.00	0.00
Sn	-0.00	-0.00	0.00	0.00	0.00	0.00
E1	-39.46	-0.01	91.51	0.00	0.00	0.00
E2	-61.70	0.03	147.16	0.00	0.00	0.00
E3	-2.39	-0.02	6.78	0.00	0.00	0.00
E4	6.11	0.02	-13.77	0.00	0.00	0.00
E5	-40.18	-0.01	93.54	0.00	0.00	0.00
E6	-38.75	-0.00	89.47	0.00	0.00	0.00
E7	38.75	0.00	-89.47	0.00	0.00	0.00
E8	40.18	0.01	-93.54	0.00	0.00	0.00
E9	-14.23	-0.02	34.23	0.00	0.00	0.00
E10	-9.44	0.01	20.67	0.00	0.00	0.00
E11	9.44	-0.01	-20.67	0.00	0.00	0.00
E12	14.23	0.02	-34.23	0.00	0.00	0.00
E13	-37.63	0.00	87.37	0.00	0.00	0.00
E14	-41.30	-0.01	95.64	0.00	0.00	0.00
E15	41.30	0.01	-95.64	0.00	0.00	0.00
E16	37.63	-0.00	-87.37	0.00	0.00	0.00
E17	-5.73	0.02	13.68	0.00	0.00	0.00
E18	-17.95	-0.02	41.22	0.00	0.00	0.00
E19	17.95	0.02	-41.22	0.00	0.00	0.00
E20	5.73	-0.02	-13.68	0.00	0.00	0.00
E21	-62.42	0.02	149.19	0.00	0.00	0.00
E22	-60.98	0.03	145.13	0.00	0.00	0.00
E23	60.98	-0.03	-145.13	0.00	0.00	0.00
E24	62.42	-0.02	-149.19	0.00	0.00	0.00
E25	-20.91	-0.01	50.93	0.00	0.00	0.00
E26	-16.12	0.02	37.37	0.00	0.00	0.00
E27	16.12	-0.02	-37.37	0.00	0.00	0.00
E28	20.91	0.01	-50.93	0.00	0.00	0.00
E29	-59.87	0.03	143.03	0.00	0.00	0.00
E30	-63.54	0.02	151.29	0.00	0.00	0.00
E31	63.54	-0.02	-151.29	0.00	0.00	0.00
E32	59.87	-0.03	-143.03	0.00	0.00	0.00
E33	-12.40	0.03	30.38	0.00	0.00	0.00
E34	-24.62	-0.02	57.92	0.00	0.00	0.00
E35	24.62	0.02	-57.92	0.00	0.00	0.00

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E36	12.40	-0.03	-30.38	0.00	0.00	0.00
E37	-119.45	0.01	255.41	0.00	0.00	0.00
E38	3.57	-0.35	-6.77	0.00	0.00	0.00
E39	-57.33	0.01	129.70	0.00	0.00	0.00
E40	1.85	-0.07	-3.49	0.00	0.00	0.00

Frame #16:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
231	D	0.60	-0.00	36.98	0.00	0.00	0.00
	Lp	0.88	-0.00	41.65	0.00	0.00	0.00
	Ln	-0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.05	-0.00	10.49	0.00	0.00	0.00
	Sn	-0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-39.43	-0.00	-65.67	0.00	0.00	0.00
	E2	-61.67	0.00	-102.69	0.00	0.00	0.00
	E3	-2.40	0.00	-3.99	0.00	0.00	0.00
	E4	6.11	-0.00	10.17	0.00	0.00	0.00
	E5	-40.15	-0.00	-66.86	0.00	0.00	0.00
	E6	-38.71	-0.00	-64.47	0.00	0.00	0.00
	E7	38.71	0.00	64.47	0.00	0.00	0.00
	E8	40.15	0.00	66.86	0.00	0.00	0.00
	E9	-14.22	-0.00	-23.69	0.00	0.00	0.00
	E10	-9.43	-0.00	-15.71	0.00	0.00	0.00
	E11	9.43	0.00	15.71	0.00	0.00	0.00
	E12	14.22	0.00	23.69	0.00	0.00	0.00
	E13	-37.60	-0.00	-62.62	0.00	0.00	0.00
	E14	-41.26	-0.00	-68.72	0.00	0.00	0.00
	E15	41.26	0.00	68.72	0.00	0.00	0.00
	E16	37.60	0.00	62.62	0.00	0.00	0.00
	E17	-5.72	-0.00	-9.53	0.00	0.00	0.00
	E18	-17.94	0.00	-29.87	0.00	0.00	0.00
	E19	17.94	-0.00	29.87	0.00	0.00	0.00
	E20	5.72	0.00	9.53	0.00	0.00	0.00
	E21	-62.39	0.00	-103.89	0.00	0.00	0.00
	E22	-60.95	0.00	-101.49	0.00	0.00	0.00
	E23	60.95	-0.00	101.49	0.00	0.00	0.00
	E24	62.39	-0.00	103.89	0.00	0.00	0.00
	E25	-20.90	0.00	-34.80	0.00	0.00	0.00
	E26	-16.10	0.00	-26.82	0.00	0.00	0.00

Frame Reactions



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	E27	16.10	-0.00	26.82	0.00	0.00	0.00
	E28	20.90	-0.00	34.80	0.00	0.00	0.00
	E29	-59.83	0.00	-99.64	0.00	0.00	0.00
	E30	-63.50	0.00	-105.74	0.00	0.00	0.00
	E31	63.50	-0.00	105.74	0.00	0.00	0.00
	E32	59.83	-0.00	99.64	0.00	0.00	0.00
	E33	-12.39	-0.00	-20.64	0.00	0.00	0.00
	E34	-24.61	0.00	-40.98	0.00	0.00	0.00
	E35	24.61	-0.00	40.98	0.00	0.00	0.00
	E36	12.39	0.00	20.64	0.00	0.00	0.00
	E37	-119.25	-0.00	-198.64	0.00	0.00	0.00
	E38	3.56	-0.00	5.93	0.00	0.00	0.00
	E39	-57.27	-0.00	-95.38	0.00	0.00	0.00
	E40	1.84	-0.00	3.07	0.00	0.00	0.00
240	D	-0.73	0.00	37.10	0.00	0.00	0.00
	Lp	-0.90	-0.00	41.56	0.00	0.00	0.00
	Ln	-0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	-0.12	0.00	10.80	0.00	0.00	0.00
	Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	-39.38	0.00	50.78	0.00	0.00	0.00
	E2	-61.58	-0.00	77.21	0.00	0.00	0.00
	E3	-2.40	-0.00	3.86	0.00	0.00	0.00
	E4	6.10	-0.00	-6.50	0.00	0.00	0.00
	E5	-40.10	-0.00	51.94	0.00	0.00	0.00
	E6	-38.66	0.00	49.63	0.00	0.00	0.00
	E7	38.66	-0.00	-49.63	0.00	0.00	0.00
	E8	40.10	0.00	-51.94	0.00	0.00	0.00
	E9	-14.21	-0.00	19.10	0.00	0.00	0.00
	E10	-9.42	0.00	11.37	0.00	0.00	0.00
	E11	9.42	-0.00	-11.37	0.00	0.00	0.00
	E12	14.21	0.00	-19.10	0.00	0.00	0.00
	E13	-37.55	-0.00	48.83	0.00	0.00	0.00
	E14	-41.21	0.00	52.73	0.00	0.00	0.00
	E15	41.21	-0.00	-52.73	0.00	0.00	0.00
	E16	37.55	0.00	-48.83	0.00	0.00	0.00
	E17	-5.72	-0.00	8.74	0.00	0.00	0.00
	E18	-17.91	0.00	21.74	0.00	0.00	0.00
	E19	17.91	-0.00	-21.74	0.00	0.00	0.00
	E20	5.72	0.00	-8.74	0.00	0.00	0.00

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	E21	-62.30	-0.00	78.37	0.00	0.00	0.00
	E22	-60.86	0.00	76.05	0.00	0.00	0.00
	E23	60.86	-0.00	-76.05	0.00	0.00	0.00
	E24	62.30	0.00	-78.37	0.00	0.00	0.00
	E25	-20.87	-0.00	27.02	0.00	0.00	0.00
	E26	-16.08	0.00	19.30	0.00	0.00	0.00
	E27	16.08	-0.00	-19.30	0.00	0.00	0.00
	E28	20.87	0.00	-27.02	0.00	0.00	0.00
	E29	-59.75	-0.00	75.26	0.00	0.00	0.00
	E30	-63.41	0.00	79.16	0.00	0.00	0.00
	E31	63.41	-0.00	-79.16	0.00	0.00	0.00
	E32	59.75	0.00	-75.26	0.00	0.00	0.00
	E33	-12.38	-0.00	16.66	0.00	0.00	0.00
	E34	-24.57	0.00	29.66	0.00	0.00	0.00
	E35	24.57	-0.00	-29.66	0.00	0.00	0.00
	E36	12.38	0.00	-16.66	0.00	0.00	0.00
	E37	-119.15	0.00	166.08	0.00	0.00	0.00
	E38	3.55	-0.00	-2.94	0.00	0.00	0.00
	E39	-57.20	0.00	75.68	0.00	0.00	0.00
	E40	1.84	-0.00	-1.35	0.00	0.00	0.00
253	D	0.00	0.00	37.17	0.00	0.00	0.00
	Lp	0.00	0.00	41.78	0.00	0.00	0.00
	Ln	-0.00	-0.00	0.00	0.00	0.00	0.00
	Sp	0.00	0.00	10.60	0.00	0.00	0.00
	Sn	-0.00	-0.00	0.00	0.00	0.00	0.00
	E1	0.00	0.00	14.88	0.00	0.00	0.00
	E2	-0.00	-0.00	25.48	0.00	0.00	0.00
	E3	-0.00	-0.00	0.13	0.00	0.00	0.00
	E4	0.00	0.00	-3.67	0.00	0.00	0.00
	E5	-0.00	0.00	14.92	0.00	0.00	0.00
	E6	-0.00	0.00	14.84	0.00	0.00	0.00
	E7	-0.00	-0.00	-14.84	0.00	0.00	0.00
	E8	0.00	-0.00	-14.92	0.00	0.00	0.00
	E9	0.00	0.00	4.59	0.00	0.00	0.00
	E10	0.00	0.00	4.33	0.00	0.00	0.00
	E11	-0.00	-0.00	-4.33	0.00	0.00	0.00
	E12	-0.00	-0.00	-4.59	0.00	0.00	0.00
	E13	0.00	0.00	13.78	0.00	0.00	0.00
	E14	-0.00	0.00	15.98	0.00	0.00	0.00

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E15	-0.00	-0.00	-15.98	0.00	0.00	0.00
E16	-0.00	-0.00	-13.78	0.00	0.00	0.00
E17	0.00	0.00	0.80	0.00	0.00	0.00
E18	-0.00	-0.00	8.13	0.00	0.00	0.00
E19	0.00	0.00	-8.13	0.00	0.00	0.00
E20	-0.00	-0.00	-0.80	0.00	0.00	0.00
E21	-0.00	-0.00	25.52	0.00	0.00	0.00
E22	0.00	-0.00	25.45	0.00	0.00	0.00
E23	0.00	0.00	-25.45	0.00	0.00	0.00
E24	0.00	0.00	-25.52	0.00	0.00	0.00
E25	-0.00	-0.00	7.78	0.00	0.00	0.00
E26	-0.00	-0.00	7.52	0.00	0.00	0.00
E27	0.00	0.00	-7.52	0.00	0.00	0.00
E28	0.00	0.00	-7.78	0.00	0.00	0.00
E29	-0.00	-0.00	24.38	0.00	0.00	0.00
E30	0.00	-0.00	26.59	0.00	0.00	0.00
E31	-0.00	0.00	-26.59	0.00	0.00	0.00
E32	0.00	0.00	-24.38	0.00	0.00	0.00
E33	-0.00	0.00	3.98	0.00	0.00	0.00
E34	-0.00	-0.00	11.31	0.00	0.00	0.00
E35	0.00	0.00	-11.31	0.00	0.00	0.00
E36	-0.00	-0.00	-3.98	0.00	0.00	0.00
E37	0.00	0.00	32.56	0.00	0.00	0.00
E38	0.00	0.00	-2.99	0.00	0.00	0.00
E39	0.00	0.00	19.70	0.00	0.00	0.00
E40	0.00	0.00	-1.71	0.00	0.00	0.00

Frame #17:

Node	LdC	Rx	Ry	Rz	Mxx	Myy	Tzz
		kips	kips	kips	kip-ft	kip-ft	kip-ft
202	D	4.37	0.00	33.38	0.00	0.00	0.00
	Lp	5.42	0.00	25.77	0.00	0.00	0.00
	Ln	-0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	-0.03	-0.00	8.36	0.00	0.00	0.00
	Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	-33.93	-0.00	-66.44	0.00	0.00	0.00
	E2	-56.12	0.03	-111.68	0.00	0.00	0.00
	E3	-2.39	-0.01	-5.61	0.00	0.00	0.00
	E4	6.10	0.04	11.98	0.00	0.00	0.00
	E5	-34.65	-0.00	-68.12	0.00	0.00	0.00

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	E6	-33.21	-0.00	-64.76	0.00	0.00	0.00
	E7	33.21	0.00	64.76	0.00	0.00	0.00
	E8	34.65	0.00	68.12	0.00	0.00	0.00
	E9	-12.57	-0.01	-25.54	0.00	0.00	0.00
	E10	-7.79	0.00	-14.33	0.00	0.00	0.00
	E11	7.79	-0.00	14.33	0.00	0.00	0.00
	E12	12.57	0.01	25.54	0.00	0.00	0.00
	E13	-32.10	0.01	-62.85	0.00	0.00	0.00
	E14	-35.76	-0.01	-70.04	0.00	0.00	0.00
	E15	35.76	0.01	70.04	0.00	0.00	0.00
	E16	32.10	-0.01	62.85	0.00	0.00	0.00
	E17	-4.08	0.03	-7.96	0.00	0.00	0.00
	E18	-16.28	-0.04	-31.91	0.00	0.00	0.00
	E19	16.28	0.04	31.91	0.00	0.00	0.00
	E20	4.08	-0.03	7.96	0.00	0.00	0.00
	E21	-56.84	0.03	-113.36	0.00	0.00	0.00
	E22	-55.41	0.03	-109.99	0.00	0.00	0.00
	E23	55.41	-0.03	109.99	0.00	0.00	0.00
	E24	56.84	-0.03	113.36	0.00	0.00	0.00
	E25	-19.22	0.00	-39.11	0.00	0.00	0.00
	E26	-14.45	0.01	-27.90	0.00	0.00	0.00
	E27	14.45	-0.01	27.90	0.00	0.00	0.00
	E28	19.22	-0.00	39.11	0.00	0.00	0.00
	E29	-54.29	0.04	-108.08	0.00	0.00	0.00
	E30	-57.95	0.02	-115.27	0.00	0.00	0.00
	E31	57.95	-0.02	115.27	0.00	0.00	0.00
	E32	54.29	-0.04	108.08	0.00	0.00	0.00
	E33	-10.74	0.04	-21.53	0.00	0.00	0.00
	E34	-22.94	-0.03	-45.48	0.00	0.00	0.00
	E35	22.94	0.03	45.48	0.00	0.00	0.00
	E36	10.74	-0.04	21.53	0.00	0.00	0.00
	E37	-106.18	0.02	-199.38	0.00	0.00	0.00
	E38	3.56	-0.33	6.17	0.00	0.00	0.00
	E39	-51.01	0.01	-98.65	0.00	0.00	0.00
	E40	1.84	-0.06	3.18	0.00	0.00	0.00
216	D	-4.46	0.00	38.12	0.00	0.00	0.00
	Lp	-5.43	0.00	32.47	0.00	0.00	0.00
	Ln	-0.00	-0.00	0.00	0.00	0.00	0.00
	Sp	-0.09	-0.00	8.53	0.00	0.00	0.00

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Sn	-0.00	-0.00	0.00	0.00	0.00	0.00
E1	-33.93	-0.01	66.44	0.00	0.00	0.00
E2	-56.12	-0.00	111.68	0.00	0.00	0.00
E3	-2.39	0.01	5.60	0.00	0.00	0.00
E4	6.10	0.04	-11.98	0.00	0.00	0.00
E5	-34.65	-0.01	68.12	0.00	0.00	0.00
E6	-33.21	-0.02	64.76	0.00	0.00	0.00
E7	33.21	0.02	-64.76	0.00	0.00	0.00
E8	34.65	0.01	-68.12	0.00	0.00	0.00
E9	-12.57	0.00	25.54	0.00	0.00	0.00
E10	-7.79	-0.01	14.33	0.00	0.00	0.00
E11	7.79	0.01	-14.33	0.00	0.00	0.00
E12	12.57	-0.00	-25.54	0.00	0.00	0.00
E13	-32.10	-0.00	62.85	0.00	0.00	0.00
E14	-35.76	-0.02	70.04	0.00	0.00	0.00
E15	35.76	0.02	-70.04	0.00	0.00	0.00
E16	32.10	0.00	-62.85	0.00	0.00	0.00
E17	-4.08	0.04	7.96	0.00	0.00	0.00
E18	-16.28	-0.04	31.91	0.00	0.00	0.00
E19	16.28	0.04	-31.91	0.00	0.00	0.00
E20	4.08	-0.04	-7.96	0.00	0.00	0.00
E21	-56.84	0.00	113.36	0.00	0.00	0.00
E22	-55.41	-0.00	109.99	0.00	0.00	0.00
E23	55.41	0.00	-109.99	0.00	0.00	0.00
E24	56.84	-0.00	-113.36	0.00	0.00	0.00
E25	-19.22	0.01	39.11	0.00	0.00	0.00
E26	-14.45	-0.01	27.90	0.00	0.00	0.00
E27	14.45	0.01	-27.90	0.00	0.00	0.00
E28	19.22	-0.01	-39.11	0.00	0.00	0.00
E29	-54.30	0.01	108.08	0.00	0.00	0.00
E30	-57.95	-0.01	115.27	0.00	0.00	0.00
E31	57.95	0.01	-115.27	0.00	0.00	0.00
E32	54.30	-0.01	-108.08	0.00	0.00	0.00
E33	-10.74	0.04	21.53	0.00	0.00	0.00
E34	-22.94	-0.04	45.48	0.00	0.00	0.00
E35	22.94	0.04	-45.48	0.00	0.00	0.00
E36	10.74	-0.04	-21.53	0.00	0.00	0.00
E37	-106.18	-0.01	199.38	0.00	0.00	0.00
E38	3.56	-0.32	-6.17	0.00	0.00	0.00

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E39	-51.01	-0.01	98.65	0.00	0.00	0.00
E40	1.84	-0.05	-3.18	0.00	0.00	0.00

Frame #18:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
213	D	0.00	1.90	35.67	0.00	0.00	0.00
	Lp	0.00	2.34	33.41	0.00	0.00	0.00
	Ln	-0.00	-0.00	0.02	0.00	0.00	0.00
	Sp	0.00	0.00	4.71	0.00	0.00	0.00
	Sn	-0.00	-0.00	0.00	0.00	0.00	0.00
	E1	0.00	-1.94	-7.74	0.00	0.00	0.00
	E2	-0.00	1.97	2.36	0.00	0.00	0.00
	E3	-0.00	-29.85	-81.04	0.00	0.00	0.00
	E4	0.00	-31.35	-85.78	0.00	0.00	0.00
	E5	0.00	-10.89	-32.06	0.00	0.00	0.00
	E6	0.00	7.02	16.57	0.00	0.00	0.00
	E7	-0.00	-7.02	-16.57	0.00	0.00	0.00
	E8	-0.00	10.89	32.06	0.00	0.00	0.00
	E9	0.00	-30.43	-83.36	0.00	0.00	0.00
	E10	0.00	29.27	78.72	0.00	0.00	0.00
	E11	-0.00	-29.27	-78.72	0.00	0.00	0.00
	E12	-0.00	30.43	83.36	0.00	0.00	0.00
	E13	0.00	-11.34	-33.48	0.00	0.00	0.00
	E14	0.00	7.47	17.99	0.00	0.00	0.00
	E15	-0.00	-7.47	-17.99	0.00	0.00	0.00
	E16	-0.00	11.34	33.48	0.00	0.00	0.00
	E17	0.00	-31.93	-88.10	0.00	0.00	0.00
	E18	-0.00	30.77	83.45	0.00	0.00	0.00
	E19	0.00	-30.77	-83.45	0.00	0.00	0.00
	E20	-0.00	31.93	88.10	0.00	0.00	0.00
	E21	-0.00	-6.99	-21.96	0.00	0.00	0.00
	E22	-0.00	10.92	26.67	0.00	0.00	0.00
	E23	0.00	-10.92	-26.67	0.00	0.00	0.00
	E24	0.00	6.99	21.96	0.00	0.00	0.00
	E25	-0.00	-29.26	-80.33	0.00	0.00	0.00
	E26	-0.00	30.44	81.75	0.00	0.00	0.00
	E27	0.00	-30.44	-81.75	0.00	0.00	0.00
	E28	0.00	29.26	80.33	0.00	0.00	0.00
	E29	-0.00	-7.44	-23.38	0.00	0.00	0.00

Frame Reactions



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	E30	-0.00	11.37	28.09	0.00	0.00	0.00
	E31	0.00	-11.37	-28.09	0.00	0.00	0.00
	E32	0.00	7.44	23.38	0.00	0.00	0.00
	E33	0.00	-30.76	-85.07	0.00	0.00	0.00
	E34	-0.00	31.94	86.48	0.00	0.00	0.00
	E35	0.00	-31.94	-86.48	0.00	0.00	0.00
	E36	-0.00	30.76	85.07	0.00	0.00	0.00
	E37	0.00	-0.22	-4.67	0.00	0.00	0.00
	E38	0.00	-69.85	-166.70	0.00	0.00	0.00
	E39	0.00	-0.03	-2.68	0.00	0.00	0.00
	E40	0.00	-33.32	-85.97	0.00	0.00	0.00
214	D	-0.00	-1.89	71.16	0.00	0.00	0.00
	Lp	-0.00	-2.28	63.67	0.00	0.00	0.00
	Ln	0.00	0.00	-0.02	0.00	0.00	0.00
	Sp	-0.00	-0.06	14.13	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-0.00	-1.95	7.74	0.00	0.00	0.00
	E2	0.00	1.95	-2.36	0.00	0.00	0.00
	E3	0.00	-29.80	81.04	0.00	0.00	0.00
	E4	-0.00	-31.30	85.78	0.00	0.00	0.00
	E5	-0.00	-10.88	32.06	0.00	0.00	0.00
	E6	-0.00	6.99	-16.57	0.00	0.00	0.00
	E7	0.00	-6.99	16.57	0.00	0.00	0.00
	E8	0.00	10.88	-32.06	0.00	0.00	0.00
	E9	-0.00	-30.38	83.36	0.00	0.00	0.00
	E10	-0.00	29.21	-78.72	0.00	0.00	0.00
	E11	0.00	-29.21	78.72	0.00	0.00	0.00
	E12	0.00	30.38	-83.36	0.00	0.00	0.00
	E13	-0.00	-11.33	33.48	0.00	0.00	0.00
	E14	-0.00	7.44	-17.99	0.00	0.00	0.00
	E15	0.00	-7.44	17.99	0.00	0.00	0.00
	E16	0.00	11.33	-33.48	0.00	0.00	0.00
	E17	-0.00	-31.88	88.10	0.00	0.00	0.00
	E18	0.00	30.71	-83.45	0.00	0.00	0.00
	E19	-0.00	-30.71	83.45	0.00	0.00	0.00
	E20	0.00	31.88	-88.10	0.00	0.00	0.00
	E21	0.00	-6.99	21.96	0.00	0.00	0.00
	E22	0.00	10.89	-26.67	0.00	0.00	0.00
	E23	-0.00	-10.89	26.67	0.00	0.00	0.00

Frame Reactions



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E24	-0.00	6.99	-21.96	0.00	0.00	0.00
E25	0.00	-29.21	80.33	0.00	0.00	0.00
E26	0.00	30.38	-81.75	0.00	0.00	0.00
E27	-0.00	-30.38	81.75	0.00	0.00	0.00
E28	-0.00	29.21	-80.33	0.00	0.00	0.00
E29	0.00	-7.44	23.38	0.00	0.00	0.00
E30	0.00	11.34	-28.09	0.00	0.00	0.00
E31	-0.00	-11.34	28.09	0.00	0.00	0.00
E32	-0.00	7.44	-23.38	0.00	0.00	0.00
E33	-0.00	-30.71	85.07	0.00	0.00	0.00
E34	0.00	31.88	-86.48	0.00	0.00	0.00
E35	-0.00	-31.88	86.48	0.00	0.00	0.00
E36	0.00	30.71	-85.07	0.00	0.00	0.00
E37	-0.00	-0.24	4.67	0.00	0.00	0.00
E38	-0.00	-69.62	166.70	0.00	0.00	0.00
E39	-0.00	-0.04	2.68	0.00	0.00	0.00
E40	-0.00	-33.24	85.97	0.00	0.00	0.00

Frame #19:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
250	D	0.00	3.28	83.68	0.00	0.00	0.00
	Lp	0.00	3.93	93.70	0.00	0.00	0.00
	Ln	-0.00	-0.01	-2.45	0.00	0.00	0.00
	Sp	0.00	0.19	5.87	0.00	0.00	0.00
	Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	0.00	1.67	2.77	0.00	0.00	0.00
	E2	-0.00	-1.69	-2.81	0.00	0.00	0.00
	E3	-0.00	-28.24	-47.05	0.00	0.00	0.00
	E4	0.00	-26.97	-44.94	0.00	0.00	0.00
	E5	0.00	-6.81	-11.34	0.00	0.00	0.00
	E6	0.00	10.14	16.89	0.00	0.00	0.00
	E7	-0.00	-10.14	-16.89	0.00	0.00	0.00
	E8	-0.00	6.81	11.34	0.00	0.00	0.00
	E9	0.00	-27.74	-46.22	0.00	0.00	0.00
	E10	0.00	28.73	47.88	0.00	0.00	0.00
	E11	-0.00	-28.73	-47.88	0.00	0.00	0.00
	E12	-0.00	27.74	46.22	0.00	0.00	0.00
	E13	0.00	-6.42	-10.71	0.00	0.00	0.00
	E14	0.00	9.75	16.26	0.00	0.00	0.00

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	E15	-0.00	-9.75	-16.26	0.00	0.00	0.00
	E16	-0.00	6.42	10.71	0.00	0.00	0.00
	E17	0.00	-26.47	-44.10	0.00	0.00	0.00
	E18	-0.00	27.47	45.77	0.00	0.00	0.00
	E19	0.00	-27.47	-45.77	0.00	0.00	0.00
	E20	-0.00	26.47	44.10	0.00	0.00	0.00
	E21	-0.00	-10.16	-16.93	0.00	0.00	0.00
	E22	-0.00	6.78	11.30	0.00	0.00	0.00
	E23	0.00	-6.78	-11.30	0.00	0.00	0.00
	E24	0.00	10.16	16.93	0.00	0.00	0.00
	E25	-0.00	-28.74	-47.89	0.00	0.00	0.00
	E26	-0.00	27.73	46.21	0.00	0.00	0.00
	E27	0.00	-27.73	-46.21	0.00	0.00	0.00
	E28	0.00	28.74	47.89	0.00	0.00	0.00
	E29	-0.00	-9.78	-16.29	0.00	0.00	0.00
	E30	-0.00	6.40	10.67	0.00	0.00	0.00
	E31	0.00	-6.40	-10.67	0.00	0.00	0.00
	E32	0.00	9.78	16.29	0.00	0.00	0.00
	E33	0.00	-27.47	-45.78	0.00	0.00	0.00
	E34	-0.00	26.46	44.09	0.00	0.00	0.00
	E35	0.00	-26.46	-44.09	0.00	0.00	0.00
	E36	-0.00	27.47	45.78	0.00	0.00	0.00
	E37	0.00	0.19	0.32	0.00	0.00	0.00
	E38	0.00	-63.12	-105.18	0.00	0.00	0.00
	E39	0.00	0.03	0.04	0.00	0.00	0.00
	E40	0.00	-30.09	-50.15	0.00	0.00	0.00
251	D	-0.00	-3.26	108.49	0.00	0.00	0.00
	Lp	-0.00	-3.95	79.87	0.00	0.00	0.00
	Ln	0.00	0.01	-3.25	0.00	0.00	0.00
	Sp	-0.00	-0.14	38.13	0.00	0.00	0.00
	Sn	0.00	-0.00	0.00	0.00	0.00	0.00
	E1	-0.00	1.67	-2.77	0.00	0.00	0.00
	E2	0.00	-1.69	2.81	0.00	0.00	0.00
	E3	0.00	-28.24	47.05	0.00	0.00	0.00
	E4	-0.00	-26.97	44.94	0.00	0.00	0.00
	E5	-0.00	-6.81	11.34	0.00	0.00	0.00
	E6	-0.00	10.14	-16.89	0.00	0.00	0.00
	E7	0.00	-10.14	16.89	0.00	0.00	0.00
	E8	0.00	6.81	-11.34	0.00	0.00	0.00

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E9	-0.00	-27.74	46.22	0.00	0.00	0.00
E10	-0.00	28.73	-47.88	0.00	0.00	0.00
E11	0.00	-28.73	47.88	0.00	0.00	0.00
E12	0.00	27.74	-46.22	0.00	0.00	0.00
E13	-0.00	-6.42	10.71	0.00	0.00	0.00
E14	-0.00	9.75	-16.26	0.00	0.00	0.00
E15	0.00	-9.75	16.26	0.00	0.00	0.00
E16	0.00	6.42	-10.71	0.00	0.00	0.00
E17	-0.00	-26.47	44.10	0.00	0.00	0.00
E18	0.00	27.47	-45.77	0.00	0.00	0.00
E19	-0.00	-27.47	45.77	0.00	0.00	0.00
E20	0.00	26.47	-44.10	0.00	0.00	0.00
E21	0.00	-10.16	16.93	0.00	0.00	0.00
E22	0.00	6.78	-11.30	0.00	0.00	0.00
E23	-0.00	-6.78	11.30	0.00	0.00	0.00
E24	-0.00	10.16	-16.93	0.00	0.00	0.00
E25	0.00	-28.74	47.89	0.00	0.00	0.00
E26	0.00	27.73	-46.21	0.00	0.00	0.00
E27	-0.00	-27.73	46.21	0.00	0.00	0.00
E28	-0.00	28.74	-47.89	0.00	0.00	0.00
E29	0.00	-9.78	16.29	0.00	0.00	0.00
E30	0.00	6.40	-10.67	0.00	0.00	0.00
E31	-0.00	-6.40	10.67	0.00	0.00	0.00
E32	-0.00	9.78	-16.29	0.00	0.00	0.00
E33	-0.00	-27.47	45.78	0.00	0.00	0.00
E34	0.00	26.46	-44.09	0.00	0.00	0.00
E35	-0.00	-26.46	44.09	0.00	0.00	0.00
E36	0.00	27.47	-45.78	0.00	0.00	0.00
E37	-0.00	0.19	-0.32	0.00	0.00	0.00
E38	-0.00	-63.12	105.18	0.00	0.00	0.00
E39	-0.00	0.03	-0.04	0.00	0.00	0.00
E40	-0.00	-30.09	50.15	0.00	0.00	0.00

Frame #20:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
249	D	-0.00	0.00	102.03	0.00	0.00	0.00
	Lp	0.00	-0.00	92.60	0.00	0.00	0.00
	Ln	-0.00	-0.00	0.01	0.00	0.00	0.00
	Sp	-0.00	0.00	23.26	0.00	0.00	0.00

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Sn	-0.00	-0.00	0.00	0.00	0.00	0.00
E1	-0.00	0.00	-1.09	0.00	0.00	0.00
E2	-0.00	-0.00	2.56	0.00	0.00	0.00
E3	0.00	-0.00	46.66	0.00	0.00	0.00
E4	-0.00	-0.00	41.79	0.00	0.00	0.00
E5	-0.00	-0.00	12.90	0.00	0.00	0.00
E6	-0.00	0.00	-15.09	0.00	0.00	0.00
E7	0.00	-0.00	15.09	0.00	0.00	0.00
E8	0.00	0.00	-12.90	0.00	0.00	0.00
E9	-0.00	-0.00	46.33	0.00	0.00	0.00
E10	-0.00	0.00	-46.99	0.00	0.00	0.00
E11	0.00	-0.00	46.99	0.00	0.00	0.00
E12	0.00	0.00	-46.33	0.00	0.00	0.00
E13	-0.00	-0.00	11.44	0.00	0.00	0.00
E14	-0.00	0.00	-13.63	0.00	0.00	0.00
E15	0.00	-0.00	13.63	0.00	0.00	0.00
E16	0.00	0.00	-11.44	0.00	0.00	0.00
E17	-0.00	-0.00	41.46	0.00	0.00	0.00
E18	-0.00	0.00	-42.12	0.00	0.00	0.00
E19	0.00	-0.00	42.12	0.00	0.00	0.00
E20	0.00	0.00	-41.46	0.00	0.00	0.00
E21	-0.00	-0.00	16.55	0.00	0.00	0.00
E22	-0.00	0.00	-11.44	0.00	0.00	0.00
E23	0.00	-0.00	11.44	0.00	0.00	0.00
E24	0.00	0.00	-16.55	0.00	0.00	0.00
E25	-0.00	-0.00	47.43	0.00	0.00	0.00
E26	-0.00	0.00	-45.89	0.00	0.00	0.00
E27	0.00	-0.00	45.89	0.00	0.00	0.00
E28	0.00	0.00	-47.43	0.00	0.00	0.00
E29	-0.00	-0.00	15.09	0.00	0.00	0.00
E30	-0.00	0.00	-9.98	0.00	0.00	0.00
E31	0.00	-0.00	9.98	0.00	0.00	0.00
E32	0.00	0.00	-15.09	0.00	0.00	0.00
E33	-0.00	-0.00	42.56	0.00	0.00	0.00
E34	-0.00	0.00	-41.02	0.00	0.00	0.00
E35	0.00	-0.00	41.02	0.00	0.00	0.00
E36	0.00	0.00	-42.56	0.00	0.00	0.00
E37	-0.00	0.00	1.22	0.00	0.00	0.00
E38	0.00	-0.00	69.05	0.00	0.00	0.00

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E39	-0.00	0.00	0.72	0.00	0.00	0.00
E40	0.00	-0.00	41.61	0.00	0.00	0.00

Frame #21:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
210	D	-0.00	7.09	56.23	0.00	0.00	0.00
	Lp	0.00	9.51	46.35	0.00	0.00	0.00
	Ln	-0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	-0.01	0.08	23.47	0.00	0.00	0.00
	Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	-0.05	-2.47	-3.20	0.00	0.00	0.00
	E2	-0.11	2.48	3.21	0.00	0.00	0.00
	E3	-0.00	-37.89	-49.04	0.00	0.00	0.00
	E4	-0.01	-39.80	-51.50	0.00	0.00	0.00
	E5	-0.05	-13.84	-17.91	0.00	0.00	0.00
	E6	-0.05	8.90	11.51	0.00	0.00	0.00
	E7	0.05	-8.90	-11.51	0.00	0.00	0.00
	E8	0.05	13.84	17.91	0.00	0.00	0.00
	E9	-0.02	-38.64	-50.00	0.00	0.00	0.00
	E10	-0.01	37.15	48.08	0.00	0.00	0.00
	E11	0.01	-37.15	-48.08	0.00	0.00	0.00
	E12	0.02	38.64	50.00	0.00	0.00	0.00
	E13	-0.05	-14.41	-18.65	0.00	0.00	0.00
	E14	-0.05	9.47	12.25	0.00	0.00	0.00
	E15	0.05	-9.47	-12.25	0.00	0.00	0.00
	E16	0.05	14.41	18.65	0.00	0.00	0.00
	E17	-0.03	-40.54	-52.46	0.00	0.00	0.00
	E18	-0.00	39.06	50.54	0.00	0.00	0.00
	E19	0.00	-39.06	-50.54	0.00	0.00	0.00
	E20	0.03	40.54	52.46	0.00	0.00	0.00
	E21	-0.11	-8.89	-11.50	0.00	0.00	0.00
	E22	-0.11	13.85	17.92	0.00	0.00	0.00
	E23	0.11	-13.85	-17.92	0.00	0.00	0.00
	E24	0.11	8.89	11.50	0.00	0.00	0.00
	E25	-0.04	-37.15	-48.07	0.00	0.00	0.00
	E26	-0.03	38.64	50.00	0.00	0.00	0.00
	E27	0.03	-38.64	-50.00	0.00	0.00	0.00
	E28	0.04	37.15	48.07	0.00	0.00	0.00
	E29	-0.11	-9.46	-12.24	0.00	0.00	0.00

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	E30	-0.11	14.42	18.66	0.00	0.00	0.00
	E31	0.11	-14.42	-18.66	0.00	0.00	0.00
	E32	0.11	9.46	12.24	0.00	0.00	0.00
	E33	-0.04	-39.06	-50.54	0.00	0.00	0.00
	E34	-0.02	40.55	52.46	0.00	0.00	0.00
	E35	0.02	-40.55	-52.46	0.00	0.00	0.00
	E36	0.04	39.06	50.54	0.00	0.00	0.00
	E37	-0.58	-0.30	-0.39	0.00	0.00	0.00
	E38	-0.01	-88.67	-114.81	0.00	0.00	0.00
	E39	-0.17	-0.05	-0.06	0.00	0.00	0.00
	E40	-0.01	-42.30	-54.76	0.00	0.00	0.00
211	D	-0.00	-7.16	50.44	0.00	0.00	0.00
	Lp	0.00	-9.54	42.15	0.00	0.00	0.00
	Ln	-0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.00	-0.15	20.01	0.00	0.00	0.00
	Sn	-0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-0.00	-2.48	3.22	0.00	0.00	0.00
	E2	-0.00	2.48	-3.23	0.00	0.00	0.00
	E3	0.00	-37.92	49.36	0.00	0.00	0.00
	E4	-0.00	-39.83	51.85	0.00	0.00	0.00
	E5	-0.00	-13.85	18.03	0.00	0.00	0.00
	E6	-0.00	8.90	-11.59	0.00	0.00	0.00
	E7	0.00	-8.90	11.59	0.00	0.00	0.00
	E8	0.00	13.85	-18.03	0.00	0.00	0.00
	E9	-0.00	-38.66	50.33	0.00	0.00	0.00
	E10	-0.00	37.17	-48.39	0.00	0.00	0.00
	E11	0.00	-37.17	48.39	0.00	0.00	0.00
	E12	0.00	38.66	-50.33	0.00	0.00	0.00
	E13	-0.00	-14.42	18.78	0.00	0.00	0.00
	E14	-0.00	9.47	-12.33	0.00	0.00	0.00
	E15	0.00	-9.47	12.33	0.00	0.00	0.00
	E16	0.00	14.42	-18.78	0.00	0.00	0.00
	E17	-0.00	-40.57	52.82	0.00	0.00	0.00
	E18	0.00	39.08	-50.88	0.00	0.00	0.00
	E19	-0.00	-39.08	50.88	0.00	0.00	0.00
	E20	0.00	40.57	-52.82	0.00	0.00	0.00
	E21	-0.00	-8.90	11.58	0.00	0.00	0.00
	E22	-0.00	13.85	-18.03	0.00	0.00	0.00
	E23	0.00	-13.85	18.03	0.00	0.00	0.00

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E24	0.00	8.90	-11.58	0.00	0.00	0.00
E25	-0.00	-37.17	48.39	0.00	0.00	0.00
E26	-0.00	38.66	-50.33	0.00	0.00	0.00
E27	0.00	-38.66	50.33	0.00	0.00	0.00
E28	0.00	37.17	-48.39	0.00	0.00	0.00
E29	-0.00	-9.47	12.33	0.00	0.00	0.00
E30	-0.00	14.43	-18.78	0.00	0.00	0.00
E31	0.00	-14.43	18.78	0.00	0.00	0.00
E32	0.00	9.47	-12.33	0.00	0.00	0.00
E33	-0.00	-39.08	50.88	0.00	0.00	0.00
E34	0.00	40.57	-52.82	0.00	0.00	0.00
E35	-0.00	-40.57	52.82	0.00	0.00	0.00
E36	0.00	39.08	-50.88	0.00	0.00	0.00
E37	-0.00	-0.31	0.40	0.00	0.00	0.00
E38	-0.00	-88.59	115.34	0.00	0.00	0.00
E39	-0.00	-0.05	0.07	0.00	0.00	0.00
E40	-0.00	-42.30	55.07	0.00	0.00	0.00

Frame #22:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
260	D	2.14	5.20	37.83	0.00	0.00	0.00
	Lp	0.00	9.01	28.80	0.00	0.00	0.00
	Ln	-0.00	-0.00	-0.01	0.00	0.00	0.00
	Sp	3.25	0.29	20.95	0.00	0.00	0.00
	Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	0.00	4.91	9.99	0.00	0.00	0.00
	E2	-0.00	-4.96	-7.44	0.00	0.00	0.00
	E3	-0.00	-41.40	-72.39	0.00	0.00	0.00
	E4	0.00	-37.65	-64.57	0.00	0.00	0.00
	E5	0.00	-7.51	-11.73	0.00	0.00	0.00
	E6	0.00	17.33	31.71	0.00	0.00	0.00
	E7	-0.00	-17.33	-31.71	0.00	0.00	0.00
	E8	-0.00	7.51	11.73	0.00	0.00	0.00
	E9	0.00	-39.93	-69.39	0.00	0.00	0.00
	E10	0.00	42.88	75.39	0.00	0.00	0.00
	E11	-0.00	-42.88	-75.39	0.00	0.00	0.00
	E12	-0.00	39.93	69.39	0.00	0.00	0.00
	E13	0.00	-6.38	-9.38	0.00	0.00	0.00
	E14	0.00	16.21	29.36	0.00	0.00	0.00

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	E15	-0.00	-16.21	-29.36	0.00	0.00	0.00
	E16	-0.00	6.38	9.38	0.00	0.00	0.00
	E17	0.00	-36.17	-61.57	0.00	0.00	0.00
	E18	-0.00	39.12	67.56	0.00	0.00	0.00
	E19	0.00	-39.12	-67.56	0.00	0.00	0.00
	E20	-0.00	36.17	61.57	0.00	0.00	0.00
	E21	-0.00	-17.38	-29.16	0.00	0.00	0.00
	E22	-0.00	7.46	14.28	0.00	0.00	0.00
	E23	0.00	-7.46	-14.28	0.00	0.00	0.00
	E24	0.00	17.38	29.16	0.00	0.00	0.00
	E25	-0.00	-42.89	-74.62	0.00	0.00	0.00
	E26	-0.00	39.92	70.16	0.00	0.00	0.00
	E27	0.00	-39.92	-70.16	0.00	0.00	0.00
	E28	0.00	42.89	74.62	0.00	0.00	0.00
	E29	-0.00	-16.26	-26.81	0.00	0.00	0.00
	E30	-0.00	6.33	11.93	0.00	0.00	0.00
	E31	0.00	-6.33	-11.93	0.00	0.00	0.00
	E32	0.00	16.26	26.81	0.00	0.00	0.00
	E33	0.00	-39.14	-66.80	0.00	0.00	0.00
	E34	-0.00	36.16	62.34	0.00	0.00	0.00
	E35	0.00	-36.16	-62.34	0.00	0.00	0.00
	E36	-0.00	39.14	66.80	0.00	0.00	0.00
	E37	0.00	0.58	2.83	0.00	0.00	0.00
	E38	0.00	-90.54	-144.44	0.00	0.00	0.00
	E39	0.00	0.08	1.38	0.00	0.00	0.00
	E40	0.00	-43.14	-72.17	0.00	0.00	0.00
261	D	2.16	-5.12	43.86	0.00	0.00	0.00
	Lp	-0.00	-9.04	39.93	0.00	0.00	0.00
	Ln	0.00	-0.00	0.01	0.00	0.00	0.00
	Sp	3.25	-0.15	21.06	0.00	0.00	0.00
	Sn	0.00	-0.00	0.00	0.00	0.00	0.00
	E1	-0.00	4.91	-9.99	0.00	0.00	0.00
	E2	0.00	-4.96	7.44	0.00	0.00	0.00
	E3	0.00	-41.40	72.39	0.00	0.00	0.00
	E4	-0.00	-37.65	64.57	0.00	0.00	0.00
	E5	-0.00	-7.51	11.73	0.00	0.00	0.00
	E6	-0.00	17.33	-31.71	0.00	0.00	0.00
	E7	0.00	-17.33	31.71	0.00	0.00	0.00
	E8	0.00	7.51	-11.73	0.00	0.00	0.00

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E9	-0.00	-39.93	69.39	0.00	0.00	0.00
E10	-0.00	42.88	-75.39	0.00	0.00	0.00
E11	0.00	-42.88	75.39	0.00	0.00	0.00
E12	0.00	39.93	-69.39	0.00	0.00	0.00
E13	-0.00	-6.38	9.38	0.00	0.00	0.00
E14	-0.00	16.21	-29.36	0.00	0.00	0.00
E15	0.00	-16.21	29.36	0.00	0.00	0.00
E16	0.00	6.38	-9.38	0.00	0.00	0.00
E17	-0.00	-36.17	61.57	0.00	0.00	0.00
E18	0.00	39.12	-67.56	0.00	0.00	0.00
E19	-0.00	-39.12	67.56	0.00	0.00	0.00
E20	0.00	36.17	-61.57	0.00	0.00	0.00
E21	0.00	-17.38	29.16	0.00	0.00	0.00
E22	0.00	7.46	-14.28	0.00	0.00	0.00
E23	-0.00	-7.46	14.28	0.00	0.00	0.00
E24	-0.00	17.38	-29.16	0.00	0.00	0.00
E25	0.00	-42.89	74.62	0.00	0.00	0.00
E26	0.00	39.92	-70.16	0.00	0.00	0.00
E27	-0.00	-39.92	70.16	0.00	0.00	0.00
E28	-0.00	42.89	-74.62	0.00	0.00	0.00
E29	0.00	-16.26	26.81	0.00	0.00	0.00
E30	0.00	6.33	-11.93	0.00	0.00	0.00
E31	-0.00	-6.33	11.93	0.00	0.00	0.00
E32	-0.00	16.26	-26.81	0.00	0.00	0.00
E33	-0.00	-39.14	66.80	0.00	0.00	0.00
E34	0.00	36.16	-62.34	0.00	0.00	0.00
E35	-0.00	-36.16	62.34	0.00	0.00	0.00
E36	0.00	39.14	-66.80	0.00	0.00	0.00
E37	-0.00	0.58	-2.83	0.00	0.00	0.00
E38	-0.00	-90.54	144.44	0.00	0.00	0.00
E39	-0.00	0.08	-1.38	0.00	0.00	0.00
E40	-0.00	-43.14	72.17	0.00	0.00	0.00

Frame #23:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
244	D	-0.00	10.17	63.96	0.00	0.00	0.00
	Lp	-0.00	18.23	79.03	0.00	0.00	0.00
	Ln	-0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	-0.00	0.29	20.09	0.00	0.00	0.00

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Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
E1	-0.00	5.08	9.63	0.00	0.00	0.00
E2	-0.00	-4.82	-6.73	0.00	0.00	0.00
E3	-0.00	-83.35	-141.14	0.00	0.00	0.00
E4	0.00	-79.59	-133.11	0.00	0.00	0.00
E5	-0.00	-19.92	-32.71	0.00	0.00	0.00
E6	-0.00	30.09	51.97	0.00	0.00	0.00
E7	0.00	-30.09	-51.97	0.00	0.00	0.00
E8	0.00	19.92	32.71	0.00	0.00	0.00
E9	-0.00	-81.83	-138.25	0.00	0.00	0.00
E10	0.00	84.88	144.03	0.00	0.00	0.00
E11	-0.00	-84.88	-144.03	0.00	0.00	0.00
E12	0.00	81.83	138.25	0.00	0.00	0.00
E13	-0.00	-18.79	-30.30	0.00	0.00	0.00
E14	-0.00	28.96	49.56	0.00	0.00	0.00
E15	0.00	-28.96	-49.56	0.00	0.00	0.00
E16	0.00	18.79	30.30	0.00	0.00	0.00
E17	-0.00	-78.07	-130.22	0.00	0.00	0.00
E18	-0.00	81.12	136.00	0.00	0.00	0.00
E19	0.00	-81.12	-136.00	0.00	0.00	0.00
E20	0.00	78.07	130.22	0.00	0.00	0.00
E21	-0.00	-29.83	-49.07	0.00	0.00	0.00
E22	-0.00	20.18	35.61	0.00	0.00	0.00
E23	0.00	-20.18	-35.61	0.00	0.00	0.00
E24	0.00	29.83	49.07	0.00	0.00	0.00
E25	-0.00	-84.80	-143.16	0.00	0.00	0.00
E26	-0.00	81.90	139.12	0.00	0.00	0.00
E27	0.00	-81.90	-139.12	0.00	0.00	0.00
E28	0.00	84.80	143.16	0.00	0.00	0.00
E29	-0.00	-28.70	-46.66	0.00	0.00	0.00
E30	-0.00	19.05	33.20	0.00	0.00	0.00
E31	0.00	-19.05	-33.20	0.00	0.00	0.00
E32	0.00	28.70	46.66	0.00	0.00	0.00
E33	-0.00	-81.04	-135.13	0.00	0.00	0.00
E34	-0.00	78.14	131.09	0.00	0.00	0.00
E35	0.00	-78.14	-131.09	0.00	0.00	0.00
E36	0.00	81.04	135.13	0.00	0.00	0.00
E37	-0.00	0.83	3.15	0.00	0.00	0.00
E38	-0.00	-186.18	-290.83	0.00	0.00	0.00

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	E39	-0.00	0.24	1.56	0.00	0.00	0.00
	E40	-0.00	-88.80	-144.82	0.00	0.00	0.00
245	D	-0.91	0.06	67.80	0.00	0.00	0.00
	Lp	-1.08	-0.05	83.09	0.00	0.00	0.00
	Ln	0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.25	0.14	20.60	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-32.96	9.78	84.90	0.00	0.00	0.00
	E2	-31.72	-9.94	80.50	0.00	0.00	0.00
	E3	0.13	-166.02	0.07	0.00	0.00	0.00
	E4	-0.34	-158.60	1.88	0.00	0.00	0.00
	E5	-32.92	-40.03	84.92	0.00	0.00	0.00
	E6	-33.00	59.58	84.88	0.00	0.00	0.00
	E7	33.00	-59.58	-84.88	0.00	0.00	0.00
	E8	32.92	40.03	-84.92	0.00	0.00	0.00
	E9	-9.76	-163.08	25.54	0.00	0.00	0.00
	E10	-10.01	168.95	25.40	0.00	0.00	0.00
	E11	10.01	-168.95	-25.40	0.00	0.00	0.00
	E12	9.76	163.08	-25.54	0.00	0.00	0.00
	E13	-33.06	-37.80	85.47	0.00	0.00	0.00
	E14	-32.86	57.36	84.34	0.00	0.00	0.00
	E15	32.86	-57.36	-84.34	0.00	0.00	0.00
	E16	33.06	37.80	-85.47	0.00	0.00	0.00
	E17	-10.23	-155.66	27.35	0.00	0.00	0.00
	E18	-9.54	161.53	23.59	0.00	0.00	0.00
	E19	9.54	-161.53	-23.59	0.00	0.00	0.00
	E20	10.23	155.66	-27.35	0.00	0.00	0.00
	E21	-31.69	-59.75	80.52	0.00	0.00	0.00
	E22	-31.76	39.86	80.48	0.00	0.00	0.00
	E23	31.76	-39.86	-80.48	0.00	0.00	0.00
	E24	31.69	59.75	-80.52	0.00	0.00	0.00
	E25	-9.39	-169.00	24.22	0.00	0.00	0.00
	E26	-9.64	163.03	24.08	0.00	0.00	0.00
	E27	9.64	-163.03	-24.08	0.00	0.00	0.00
	E28	9.39	169.00	-24.22	0.00	0.00	0.00
	E29	-31.83	-57.52	81.06	0.00	0.00	0.00
	E30	-31.62	37.64	79.94	0.00	0.00	0.00
	E31	31.62	-37.64	-79.94	0.00	0.00	0.00
	E32	31.83	57.52	-81.06	0.00	0.00	0.00

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	E33	-9.86	-161.58	26.03	0.00	0.00	0.00
	E34	-9.17	155.62	22.27	0.00	0.00	0.00
	E35	9.17	-155.62	-22.27	0.00	0.00	0.00
	E36	9.86	161.58	-26.03	0.00	0.00	0.00
	E37	-77.42	1.10	175.26	0.00	0.00	0.00
	E38	-0.21	-371.73	1.62	0.00	0.00	0.00
	E39	-36.84	0.14	89.47	0.00	0.00	0.00
	E40	-0.11	-177.08	0.93	0.00	0.00	0.00
246	D	-0.00	-10.11	64.54	0.00	0.00	0.00
	Lp	0.00	-18.28	79.62	0.00	0.00	0.00
	Ln	-0.00	0.00	0.00	0.00	0.00	0.00
	Sp	-0.00	-0.14	19.89	0.00	0.00	0.00
	Sn	-0.00	0.00	0.00	0.00	0.00	0.00
	E1	-0.03	4.75	-7.86	0.00	0.00	0.00
	E2	-0.04	-5.14	8.22	0.00	0.00	0.00
	E3	-0.01	-83.34	139.51	0.00	0.00	0.00
	E4	-0.01	-79.62	131.62	0.00	0.00	0.00
	E5	-0.03	-20.26	34.00	0.00	0.00	0.00
	E6	-0.03	29.75	-49.71	0.00	0.00	0.00
	E7	0.03	-29.75	49.71	0.00	0.00	0.00
	E8	0.03	20.26	-34.00	0.00	0.00	0.00
	E9	-0.01	-81.92	137.16	0.00	0.00	0.00
	E10	-0.00	84.77	-141.87	0.00	0.00	0.00
	E11	0.00	-84.77	141.87	0.00	0.00	0.00
	E12	0.01	81.92	-137.16	0.00	0.00	0.00
	E13	-0.03	-19.14	31.63	0.00	0.00	0.00
	E14	-0.03	28.63	-47.34	0.00	0.00	0.00
	E15	0.03	-28.63	47.34	0.00	0.00	0.00
	E16	0.03	19.14	-31.63	0.00	0.00	0.00
	E17	-0.01	-78.20	129.26	0.00	0.00	0.00
	E18	-0.00	81.04	-133.98	0.00	0.00	0.00
	E19	0.00	-81.04	133.98	0.00	0.00	0.00
	E20	0.01	78.20	-129.26	0.00	0.00	0.00
	E21	-0.04	-30.14	50.07	0.00	0.00	0.00
	E22	-0.04	19.86	-33.64	0.00	0.00	0.00
	E23	0.04	-19.86	33.64	0.00	0.00	0.00
	E24	0.04	30.14	-50.07	0.00	0.00	0.00
	E25	-0.02	-84.88	141.98	0.00	0.00	0.00
	E26	-0.01	81.80	-137.05	0.00	0.00	0.00

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E27	0.01	-81.80	137.05	0.00	0.00	0.00
E28	0.02	84.88	-141.98	0.00	0.00	0.00
E29	-0.04	-29.03	47.70	0.00	0.00	0.00
E30	-0.04	18.74	-31.27	0.00	0.00	0.00
E31	0.04	-18.74	31.27	0.00	0.00	0.00
E32	0.04	29.03	-47.70	0.00	0.00	0.00
E33	-0.02	-81.16	134.09	0.00	0.00	0.00
E34	-0.01	78.08	-129.15	0.00	0.00	0.00
E35	0.01	-78.08	129.15	0.00	0.00	0.00
E36	0.02	81.16	-134.09	0.00	0.00	0.00
E37	-0.20	0.30	-0.51	0.00	0.00	0.00
E38	-0.01	-186.55	288.44	0.00	0.00	0.00
E39	-0.06	-0.08	0.04	0.00	0.00	0.00
E40	-0.01	-88.88	143.37	0.00	0.00	0.00

Frame #24:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
193	D	0.00	5.31	47.13	0.00	0.00	0.00
	Lp	0.00	9.53	42.02	0.00	0.00	0.00
	Ln	-0.00	0.00	0.00	0.00	0.00	0.00
	Sp	0.00	0.01	20.29	0.00	0.00	0.00
	Sn	-0.00	0.00	0.00	0.00	0.00	0.00
	E1	0.00	-6.35	-12.06	0.00	0.00	0.00
	E2	-0.00	6.38	10.10	0.00	0.00	0.00
	E3	-0.00	-38.97	-67.84	0.00	0.00	0.00
	E4	0.00	-43.85	-77.89	0.00	0.00	0.00
	E5	0.00	-18.04	-32.41	0.00	0.00	0.00
	E6	0.00	5.34	8.29	0.00	0.00	0.00
	E7	-0.00	-5.34	-8.29	0.00	0.00	0.00
	E8	-0.00	18.04	32.41	0.00	0.00	0.00
	E9	0.00	-40.87	-71.45	0.00	0.00	0.00
	E10	0.00	37.06	64.22	0.00	0.00	0.00
	E11	-0.00	-37.06	-64.22	0.00	0.00	0.00
	E12	-0.00	40.87	71.45	0.00	0.00	0.00
	E13	0.00	-19.51	-35.42	0.00	0.00	0.00
	E14	0.00	6.80	11.31	0.00	0.00	0.00
	E15	-0.00	-6.80	-11.31	0.00	0.00	0.00
	E16	-0.00	19.51	35.42	0.00	0.00	0.00
	E17	0.00	-45.76	-81.50	0.00	0.00	0.00

Frame Reactions



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	E18	-0.00	41.95	74.27	0.00	0.00	0.00
	E19	0.00	-41.95	-74.27	0.00	0.00	0.00
	E20	-0.00	45.76	81.50	0.00	0.00	0.00
	E21	-0.00	-5.31	-10.25	0.00	0.00	0.00
	E22	-0.00	18.07	30.46	0.00	0.00	0.00
	E23	0.00	-18.07	-30.46	0.00	0.00	0.00
	E24	0.00	5.31	10.25	0.00	0.00	0.00
	E25	-0.00	-37.05	-64.80	0.00	0.00	0.00
	E26	-0.00	40.88	70.87	0.00	0.00	0.00
	E27	0.00	-40.88	-70.87	0.00	0.00	0.00
	E28	0.00	37.05	64.80	0.00	0.00	0.00
	E29	-0.00	-6.77	-13.26	0.00	0.00	0.00
	E30	-0.00	19.54	33.47	0.00	0.00	0.00
	E31	0.00	-19.54	-33.47	0.00	0.00	0.00
	E32	0.00	6.77	13.26	0.00	0.00	0.00
	E33	0.00	-41.94	-74.86	0.00	0.00	0.00
	E34	-0.00	45.77	80.92	0.00	0.00	0.00
	E35	0.00	-45.77	-80.92	0.00	0.00	0.00
	E36	-0.00	41.94	74.86	0.00	0.00	0.00
	E37	0.00	-0.77	-2.63	0.00	0.00	0.00
	E38	0.00	-94.15	-152.19	0.00	0.00	0.00
	E39	0.00	-0.13	-1.14	0.00	0.00	0.00
	E40	0.00	-45.01	-76.43	0.00	0.00	0.00
194	D	-0.00	-5.46	48.32	0.00	0.00	0.00
	Lp	-0.00	-9.56	42.25	0.00	0.00	0.00
	Ln	0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.00	-0.20	20.92	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-0.00	-6.35	12.06	0.00	0.00	0.00
	E2	0.00	6.38	-10.10	0.00	0.00	0.00
	E3	0.00	-38.97	67.84	0.00	0.00	0.00
	E4	-0.00	-43.85	77.89	0.00	0.00	0.00
	E5	-0.00	-18.04	32.41	0.00	0.00	0.00
	E6	-0.00	5.34	-8.29	0.00	0.00	0.00
	E7	0.00	-5.34	8.29	0.00	0.00	0.00
	E8	0.00	18.04	-32.41	0.00	0.00	0.00
	E9	-0.00	-40.87	71.45	0.00	0.00	0.00
	E10	-0.00	37.06	-64.22	0.00	0.00	0.00
	E11	0.00	-37.06	64.22	0.00	0.00	0.00

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E12	0.00	40.87	-71.45	0.00	0.00	0.00
E13	-0.00	-19.51	35.42	0.00	0.00	0.00
E14	-0.00	6.80	-11.31	0.00	0.00	0.00
E15	0.00	-6.80	11.31	0.00	0.00	0.00
E16	0.00	19.51	-35.42	0.00	0.00	0.00
E17	-0.00	-45.76	81.50	0.00	0.00	0.00
E18	0.00	41.95	-74.27	0.00	0.00	0.00
E19	-0.00	-41.95	74.27	0.00	0.00	0.00
E20	0.00	45.76	-81.50	0.00	0.00	0.00
E21	0.00	-5.31	10.25	0.00	0.00	0.00
E22	0.00	18.07	-30.46	0.00	0.00	0.00
E23	-0.00	-18.07	30.46	0.00	0.00	0.00
E24	-0.00	5.31	-10.25	0.00	0.00	0.00
E25	0.00	-37.05	64.80	0.00	0.00	0.00
E26	0.00	40.88	-70.87	0.00	0.00	0.00
E27	-0.00	-40.88	70.87	0.00	0.00	0.00
E28	-0.00	37.05	-64.80	0.00	0.00	0.00
E29	0.00	-6.77	13.26	0.00	0.00	0.00
E30	0.00	19.54	-33.47	0.00	0.00	0.00
E31	-0.00	-19.54	33.47	0.00	0.00	0.00
E32	-0.00	6.77	-13.26	0.00	0.00	0.00
E33	-0.00	-41.94	74.86	0.00	0.00	0.00
E34	0.00	45.77	-80.92	0.00	0.00	0.00
E35	-0.00	-45.77	80.92	0.00	0.00	0.00
E36	0.00	41.94	-74.86	0.00	0.00	0.00
E37	-0.00	-0.77	2.63	0.00	0.00	0.00
E38	-0.00	-94.15	152.19	0.00	0.00	0.00
E39	-0.00	-0.13	1.14	0.00	0.00	0.00
E40	-0.00	-45.01	76.43	0.00	0.00	0.00

Frame #25:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
207	D	0.76	10.12	67.34	0.00	0.00	0.00
	Lp	1.10	18.29	80.29	0.00	0.00	0.00
	Ln	0.00	0.00	0.01	0.00	0.00	0.00
	Sp	-0.08	0.14	20.10	0.00	0.00	0.00
	Sn	0.00	0.00	0.00	0.00	0.00	0.00
	E1	-34.16	-5.51	-106.61	0.00	0.00	0.00
	E2	-35.19	5.05	-92.60	0.00	0.00	0.00

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E3	-0.18	-80.95	-146.23	0.00	0.00	0.00
E4	0.21	-85.04	-155.65	0.00	0.00	0.00
E5	-34.21	-29.79	-150.48	0.00	0.00	0.00
E6	-34.10	18.78	-62.74	0.00	0.00	0.00
E7	34.10	-18.78	62.74	0.00	0.00	0.00
E8	34.21	29.79	150.48	0.00	0.00	0.00
E9	-10.43	-82.60	-178.21	0.00	0.00	0.00
E10	-10.06	79.30	114.24	0.00	0.00	0.00
E11	10.06	-79.30	-114.24	0.00	0.00	0.00
E12	10.43	82.60	178.21	0.00	0.00	0.00
E13	-34.10	-31.02	-153.31	0.00	0.00	0.00
E14	-34.22	20.00	-59.92	0.00	0.00	0.00
E15	34.22	-20.00	59.92	0.00	0.00	0.00
E16	34.10	31.02	153.31	0.00	0.00	0.00
E17	-10.04	-86.69	-187.63	0.00	0.00	0.00
E18	-10.46	83.38	123.66	0.00	0.00	0.00
E19	10.46	-83.38	-123.66	0.00	0.00	0.00
E20	10.04	86.69	187.63	0.00	0.00	0.00
E21	-35.25	-19.23	-136.47	0.00	0.00	0.00
E22	-35.14	29.34	-48.73	0.00	0.00	0.00
E23	35.14	-29.34	48.73	0.00	0.00	0.00
E24	35.25	19.23	136.47	0.00	0.00	0.00
E25	-10.74	-79.43	-174.01	0.00	0.00	0.00
E26	-10.37	82.47	118.45	0.00	0.00	0.00
E27	10.37	-82.47	-118.45	0.00	0.00	0.00
E28	10.74	79.43	174.01	0.00	0.00	0.00
E29	-35.13	-20.46	-139.29	0.00	0.00	0.00
E30	-35.25	30.56	-45.90	0.00	0.00	0.00
E31	35.25	-30.56	45.90	0.00	0.00	0.00
E32	35.13	20.46	139.29	0.00	0.00	0.00
E33	-10.35	-83.52	-183.43	0.00	0.00	0.00
E34	-10.77	86.55	127.87	0.00	0.00	0.00
E35	10.77	-86.55	-127.87	0.00	0.00	0.00
E36	10.35	83.52	183.43	0.00	0.00	0.00
E37	-82.73	-1.02	-206.14	0.00	0.00	0.00
E38	0.05	-189.17	-313.29	0.00	0.00	0.00
E39	-39.45	-0.34	-106.70	0.00	0.00	0.00
E40	0.02	-90.32	-157.96	0.00	0.00	0.00
D	-0.94	-10.27	62.06	0.00	0.00	0.00

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Lp	-1.10	-18.35	71.52	0.00	0.00	0.00
Ln	0.00	0.00	-0.00	0.00	0.00	0.00
Sp	-0.27	-0.30	20.49	0.00	0.00	0.00
Sn	0.00	0.00	-0.00	0.00	0.00	0.00
E1	-35.34	-5.06	67.12	0.00	0.00	0.00
E2	-34.00	5.53	46.23	0.00	0.00	0.00
E3	0.07	-80.99	149.08	0.00	0.00	0.00
E4	-0.44	-85.06	159.17	0.00	0.00	0.00
E5	-35.32	-29.36	111.85	0.00	0.00	0.00
E6	-35.36	19.23	22.40	0.00	0.00	0.00
E7	35.36	-19.23	-22.40	0.00	0.00	0.00
E8	35.32	29.36	-111.85	0.00	0.00	0.00
E9	-10.53	-82.50	169.21	0.00	0.00	0.00
E10	-10.67	79.47	-128.94	0.00	0.00	0.00
E11	10.67	-79.47	128.94	0.00	0.00	0.00
E12	10.53	82.50	-169.21	0.00	0.00	0.00
E13	-35.47	-30.58	114.88	0.00	0.00	0.00
E14	-35.21	20.46	19.37	0.00	0.00	0.00
E15	35.21	-20.46	-19.37	0.00	0.00	0.00
E16	35.47	30.58	-114.88	0.00	0.00	0.00
E17	-11.04	-86.58	179.31	0.00	0.00	0.00
E18	-10.16	83.54	-139.04	0.00	0.00	0.00
E19	10.16	-83.54	139.04	0.00	0.00	0.00
E20	11.04	86.58	-179.31	0.00	0.00	0.00
E21	-33.98	-18.76	90.95	0.00	0.00	0.00
E22	-34.02	29.83	1.51	0.00	0.00	0.00
E23	34.02	-29.83	-1.51	0.00	0.00	0.00
E24	33.98	18.76	-90.95	0.00	0.00	0.00
E25	-10.13	-79.33	162.94	0.00	0.00	0.00
E26	-10.27	82.65	-135.21	0.00	0.00	0.00
E27	10.27	-82.65	135.21	0.00	0.00	0.00
E28	10.13	79.33	-162.94	0.00	0.00	0.00
E29	-34.14	-19.99	93.98	0.00	0.00	0.00
E30	-33.87	31.05	-1.52	0.00	0.00	0.00
E31	33.87	-31.05	1.52	0.00	0.00	0.00
E32	34.14	19.99	-93.98	0.00	0.00	0.00
E33	-10.64	-83.40	173.04	0.00	0.00	0.00
E34	-9.76	86.72	-145.31	0.00	0.00	0.00
E35	9.76	-86.72	145.31	0.00	0.00	0.00

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E36	10.64	83.40	-173.04	0.00	0.00	0.00
E37	-82.87	-0.28	137.17	0.00	0.00	0.00
E38	-0.33	-189.22	318.27	0.00	0.00	0.00
E39	-39.47	0.11	64.85	0.00	0.00	0.00
E40	-0.18	-90.35	160.95	0.00	0.00	0.00

Frame #26:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
191	D	0.00	5.31	47.97	0.00	0.00	0.00
	Lp	0.00	9.53	42.03	0.00	0.00	0.00
	Ln	-0.00	0.00	0.00	0.00	0.00	0.00
	Sp	0.00	0.01	20.68	0.00	0.00	0.00
	Sn	-0.00	0.00	0.00	0.00	0.00	0.00
	E1	0.00	-6.35	-11.66	0.00	0.00	0.00
	E2	-0.00	6.38	7.71	0.00	0.00	0.00
	E3	-0.00	-38.97	-67.08	0.00	0.00	0.00
	E4	0.00	-43.85	-77.38	0.00	0.00	0.00
	E5	0.00	-18.04	-31.78	0.00	0.00	0.00
	E6	0.00	5.34	8.47	0.00	0.00	0.00
	E7	-0.00	-5.34	-8.47	0.00	0.00	0.00
	E8	-0.00	18.04	31.78	0.00	0.00	0.00
	E9	0.00	-40.87	-70.58	0.00	0.00	0.00
	E10	0.00	37.06	63.59	0.00	0.00	0.00
	E11	-0.00	-37.06	-63.59	0.00	0.00	0.00
	E12	-0.00	40.87	70.58	0.00	0.00	0.00
	E13	0.00	-19.51	-34.87	0.00	0.00	0.00
	E14	0.00	6.80	11.56	0.00	0.00	0.00
	E15	-0.00	-6.80	-11.56	0.00	0.00	0.00
	E16	-0.00	19.51	34.87	0.00	0.00	0.00
	E17	0.00	-45.76	-80.87	0.00	0.00	0.00
	E18	-0.00	41.95	73.88	0.00	0.00	0.00
	E19	0.00	-41.95	-73.88	0.00	0.00	0.00
	E20	-0.00	45.76	80.87	0.00	0.00	0.00
	E21	-0.00	-5.31	-12.41	0.00	0.00	0.00
	E22	-0.00	18.07	27.84	0.00	0.00	0.00
	E23	0.00	-18.07	-27.84	0.00	0.00	0.00
	E24	0.00	5.31	12.41	0.00	0.00	0.00
	E25	-0.00	-37.05	-64.77	0.00	0.00	0.00
	E26	-0.00	40.88	69.40	0.00	0.00	0.00

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	E27	0.00	-40.88	-69.40	0.00	0.00	0.00
	E28	0.00	37.05	64.77	0.00	0.00	0.00
	E29	-0.00	-6.77	-15.50	0.00	0.00	0.00
	E30	-0.00	19.54	30.93	0.00	0.00	0.00
	E31	0.00	-19.54	-30.93	0.00	0.00	0.00
	E32	0.00	6.77	15.50	0.00	0.00	0.00
	E33	0.00	-41.94	-75.06	0.00	0.00	0.00
	E34	-0.00	45.77	79.69	0.00	0.00	0.00
	E35	0.00	-45.77	-79.69	0.00	0.00	0.00
	E36	-0.00	41.94	75.06	0.00	0.00	0.00
	E37	0.00	-0.77	-4.19	0.00	0.00	0.00
	E38	0.00	-94.15	-151.24	0.00	0.00	0.00
	E39	0.00	-0.13	-2.10	0.00	0.00	0.00
	E40	0.00	-45.01	-75.84	0.00	0.00	0.00
192	D	-0.00	-5.46	47.48	0.00	0.00	0.00
	Lp	-0.00	-9.56	42.23	0.00	0.00	0.00
	Ln	0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.00	-0.20	20.53	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-0.00	-6.35	11.66	0.00	0.00	0.00
	E2	0.00	6.38	-7.71	0.00	0.00	0.00
	E3	0.00	-38.97	67.08	0.00	0.00	0.00
	E4	-0.00	-43.85	77.38	0.00	0.00	0.00
	E5	-0.00	-18.04	31.78	0.00	0.00	0.00
	E6	-0.00	5.34	-8.47	0.00	0.00	0.00
	E7	0.00	-5.34	8.47	0.00	0.00	0.00
	E8	0.00	18.04	-31.78	0.00	0.00	0.00
	E9	-0.00	-40.87	70.58	0.00	0.00	0.00
	E10	-0.00	37.06	-63.59	0.00	0.00	0.00
	E11	0.00	-37.06	63.59	0.00	0.00	0.00
	E12	0.00	40.87	-70.58	0.00	0.00	0.00
	E13	-0.00	-19.51	34.87	0.00	0.00	0.00
	E14	-0.00	6.80	-11.56	0.00	0.00	0.00
	E15	0.00	-6.80	11.56	0.00	0.00	0.00
	E16	0.00	19.51	-34.87	0.00	0.00	0.00
	E17	-0.00	-45.76	80.87	0.00	0.00	0.00
	E18	0.00	41.95	-73.88	0.00	0.00	0.00
	E19	-0.00	-41.95	73.88	0.00	0.00	0.00
	E20	0.00	45.76	-80.87	0.00	0.00	0.00

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E21	0.00	-5.31	12.41	0.00	0.00	0.00
E22	0.00	18.07	-27.84	0.00	0.00	0.00
E23	-0.00	-18.07	27.84	0.00	0.00	0.00
E24	-0.00	5.31	-12.41	0.00	0.00	0.00
E25	0.00	-37.05	64.77	0.00	0.00	0.00
E26	0.00	40.88	-69.40	0.00	0.00	0.00
E27	-0.00	-40.88	69.40	0.00	0.00	0.00
E28	-0.00	37.05	-64.77	0.00	0.00	0.00
E29	0.00	-6.77	15.50	0.00	0.00	0.00
E30	0.00	19.54	-30.93	0.00	0.00	0.00
E31	-0.00	-19.54	30.93	0.00	0.00	0.00
E32	-0.00	6.77	-15.50	0.00	0.00	0.00
E33	-0.00	-41.94	75.06	0.00	0.00	0.00
E34	0.00	45.77	-79.69	0.00	0.00	0.00
E35	-0.00	-45.77	79.69	0.00	0.00	0.00
E36	0.00	41.94	-75.06	0.00	0.00	0.00
E37	-0.00	-0.77	4.19	0.00	0.00	0.00
E38	-0.00	-94.15	151.24	0.00	0.00	0.00
E39	-0.00	-0.13	2.10	0.00	0.00	0.00
E40	-0.00	-45.01	75.84	0.00	0.00	0.00

Frame #27:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
204	D	0.00	10.33	82.99	0.00	0.00	0.00
	Lp	0.00	18.35	79.68	0.00	0.00	0.00
	Ln	0.00	0.00	0.00	0.00	0.00	0.00
	Sp	-0.00	0.13	19.87	0.00	0.00	0.00
	Sn	0.00	0.00	0.00	0.00	0.00	0.00
	E1	-0.04	-5.29	-9.82	0.00	0.00	0.00
	E2	-0.04	5.32	7.91	0.00	0.00	0.00
	E3	0.00	-81.00	-159.92	0.00	0.00	0.00
	E4	0.01	-85.04	-172.31	0.00	0.00	0.00
	E5	-0.04	-29.59	-57.79	0.00	0.00	0.00
	E6	-0.04	19.01	38.16	0.00	0.00	0.00
	E7	0.04	-19.01	-38.16	0.00	0.00	0.00
	E8	0.04	29.59	57.79	0.00	0.00	0.00
	E9	-0.01	-82.59	-162.87	0.00	0.00	0.00
	E10	-0.02	79.41	156.97	0.00	0.00	0.00
	E11	0.02	-79.41	-156.97	0.00	0.00	0.00

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	E12	0.01	82.59	162.87	0.00	0.00	0.00
	E13	-0.04	-30.81	-61.51	0.00	0.00	0.00
	E14	-0.05	20.22	41.87	0.00	0.00	0.00
	E15	0.05	-20.22	-41.87	0.00	0.00	0.00
	E16	0.04	30.81	61.51	0.00	0.00	0.00
	E17	-0.00	-86.63	-175.26	0.00	0.00	0.00
	E18	-0.02	83.46	169.37	0.00	0.00	0.00
	E19	0.02	-83.46	-169.37	0.00	0.00	0.00
	E20	0.00	86.63	175.26	0.00	0.00	0.00
	E21	-0.04	-18.98	-40.07	0.00	0.00	0.00
	E22	-0.04	29.61	55.88	0.00	0.00	0.00
	E23	0.04	-29.61	-55.88	0.00	0.00	0.00
	E24	0.04	18.98	40.07	0.00	0.00	0.00
	E25	-0.01	-79.40	-157.55	0.00	0.00	0.00
	E26	-0.02	82.59	162.29	0.00	0.00	0.00
	E27	0.02	-82.59	-162.29	0.00	0.00	0.00
	E28	0.01	79.40	157.55	0.00	0.00	0.00
	E29	-0.04	-20.20	-43.79	0.00	0.00	0.00
	E30	-0.05	30.83	59.60	0.00	0.00	0.00
	E31	0.05	-30.83	-59.60	0.00	0.00	0.00
	E32	0.04	20.20	43.79	0.00	0.00	0.00
	E33	-0.00	-83.45	-169.94	0.00	0.00	0.00
	E34	-0.02	86.64	174.68	0.00	0.00	0.00
	E35	0.02	-86.64	-174.68	0.00	0.00	0.00
	E36	0.00	83.45	169.94	0.00	0.00	0.00
	E37	-0.21	-0.64	-2.40	0.00	0.00	0.00
	E38	0.01	-189.57	-337.18	0.00	0.00	0.00
	E39	-0.07	-0.11	-1.08	0.00	0.00	0.00
	E40	0.01	-90.42	-172.28	0.00	0.00	0.00
205	D	0.00	-10.48	83.77	0.00	0.00	0.00
	Lp	0.00	-18.41	80.31	0.00	0.00	0.00
	Ln	0.00	0.00	-0.00	0.00	0.00	0.00
	Sp	-0.00	-0.30	20.13	0.00	0.00	0.00
	Sn	0.00	0.00	-0.00	0.00	0.00	0.00
	E1	-0.04	-5.29	9.82	0.00	0.00	0.00
	E2	-0.03	5.31	-7.91	0.00	0.00	0.00
	E3	-0.01	-81.00	159.92	0.00	0.00	0.00
	E4	0.00	-85.05	172.31	0.00	0.00	0.00
	E5	-0.04	-29.59	57.80	0.00	0.00	0.00

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E6	-0.04	19.01	-38.16	0.00	0.00	0.00
E7	0.04	-19.01	38.16	0.00	0.00	0.00
E8	0.04	29.59	-57.80	0.00	0.00	0.00
E9	-0.02	-82.59	162.87	0.00	0.00	0.00
E10	-0.01	79.41	-156.97	0.00	0.00	0.00
E11	0.01	-79.41	156.97	0.00	0.00	0.00
E12	0.02	82.59	-162.87	0.00	0.00	0.00
E13	-0.04	-30.81	61.51	0.00	0.00	0.00
E14	-0.04	20.22	-41.87	0.00	0.00	0.00
E15	0.04	-20.22	41.87	0.00	0.00	0.00
E16	0.04	30.81	-61.51	0.00	0.00	0.00
E17	-0.01	-86.63	175.26	0.00	0.00	0.00
E18	-0.01	83.46	-169.37	0.00	0.00	0.00
E19	0.01	-83.46	169.37	0.00	0.00	0.00
E20	0.01	86.63	-175.26	0.00	0.00	0.00
E21	-0.03	-18.98	40.07	0.00	0.00	0.00
E22	-0.03	29.61	-55.88	0.00	0.00	0.00
E23	0.03	-29.61	55.88	0.00	0.00	0.00
E24	0.03	18.98	-40.07	0.00	0.00	0.00
E25	-0.02	-79.40	157.55	0.00	0.00	0.00
E26	-0.00	82.59	-162.29	0.00	0.00	0.00
E27	0.00	-82.59	162.29	0.00	0.00	0.00
E28	0.02	79.40	-157.55	0.00	0.00	0.00
E29	-0.03	-20.20	43.79	0.00	0.00	0.00
E30	-0.03	30.83	-59.60	0.00	0.00	0.00
E31	0.03	-30.83	59.60	0.00	0.00	0.00
E32	0.03	20.20	-43.79	0.00	0.00	0.00
E33	-0.01	-83.45	169.94	0.00	0.00	0.00
E34	-0.01	86.64	-174.68	0.00	0.00	0.00
E35	0.01	-86.64	174.68	0.00	0.00	0.00
E36	0.01	83.45	-169.94	0.00	0.00	0.00
E37	-0.20	-0.64	2.40	0.00	0.00	0.00
E38	-0.00	-189.57	337.18	0.00	0.00	0.00
E39	-0.06	-0.11	1.08	0.00	0.00	0.00
E40	-0.00	-90.42	172.28	0.00	0.00	0.00

Frame #28:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
241	D	0.00	10.23	64.50	0.00	0.00	0.00

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Lp	0.00	18.35	79.76	0.00	0.00	0.00
Ln	-0.00	-0.00	-0.00	0.00	0.00	0.00
Sp	0.00	0.29	20.07	0.00	0.00	0.00
Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
E1	0.00	4.74	8.12	0.00	0.00	0.00
E2	-0.00	-5.23	-6.99	0.00	0.00	0.00
E3	-0.00	-83.19	-139.83	0.00	0.00	0.00
E4	0.00	-79.43	-132.01	0.00	0.00	0.00
E5	0.00	-20.22	-33.83	0.00	0.00	0.00
E6	0.00	29.69	50.07	0.00	0.00	0.00
E7	-0.00	-29.69	-50.07	0.00	0.00	0.00
E8	-0.00	20.22	33.83	0.00	0.00	0.00
E9	0.00	-81.76	-137.40	0.00	0.00	0.00
E10	0.00	84.61	142.27	0.00	0.00	0.00
E11	-0.00	-84.61	-142.27	0.00	0.00	0.00
E12	-0.00	81.76	137.40	0.00	0.00	0.00
E13	0.00	-19.09	-31.49	0.00	0.00	0.00
E14	0.00	28.57	47.72	0.00	0.00	0.00
E15	-0.00	-28.57	-47.72	0.00	0.00	0.00
E16	-0.00	19.09	31.49	0.00	0.00	0.00
E17	0.00	-78.01	-129.58	0.00	0.00	0.00
E18	-0.00	80.85	134.45	0.00	0.00	0.00
E19	0.00	-80.85	-134.45	0.00	0.00	0.00
E20	-0.00	78.01	129.58	0.00	0.00	0.00
E21	-0.00	-30.18	-48.94	0.00	0.00	0.00
E22	-0.00	19.73	34.96	0.00	0.00	0.00
E23	0.00	-19.73	-34.96	0.00	0.00	0.00
E24	0.00	30.18	48.94	0.00	0.00	0.00
E25	-0.00	-84.75	-141.93	0.00	0.00	0.00
E26	-0.00	81.62	137.73	0.00	0.00	0.00
E27	0.00	-81.62	-137.73	0.00	0.00	0.00
E28	0.00	84.75	141.93	0.00	0.00	0.00
E29	-0.00	-29.05	-46.59	0.00	0.00	0.00
E30	-0.00	18.60	32.62	0.00	0.00	0.00
E31	0.00	-18.60	-32.62	0.00	0.00	0.00
E32	0.00	29.05	46.59	0.00	0.00	0.00
E33	0.00	-81.00	-134.11	0.00	0.00	0.00
E34	-0.00	77.86	129.91	0.00	0.00	0.00
E35	0.00	-77.86	-129.91	0.00	0.00	0.00

Frame Reactions



RAM Frame 24.00.01.18

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Bentley Building Code: IBC

	E36	-0.00	81.00	134.11	0.00	0.00	0.00
	E37	0.00	0.22	1.70	0.00	0.00	0.00
	E38	0.00	-185.93	-288.78	0.00	0.00	0.00
	E39	0.00	-0.13	0.69	0.00	0.00	0.00
	E40	0.00	-88.65	-143.66	0.00	0.00	0.00
242	D	0.73	-10.18	65.89	0.00	0.00	0.00
	Lp	1.11	-18.40	81.20	0.00	0.00	0.00
	Ln	-0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	-0.04	-0.14	19.82	0.00	0.00	0.00
	Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	-33.32	5.07	-94.47	0.00	0.00	0.00
	E2	-45.23	-4.71	-115.93	0.00	0.00	0.00
	E3	-1.23	-83.16	135.47	0.00	0.00	0.00
	E4	3.32	-79.44	138.80	0.00	0.00	0.00
	E5	-33.69	-19.88	-53.83	0.00	0.00	0.00
	E6	-32.95	30.02	-135.11	0.00	0.00	0.00
	E7	32.95	-30.02	135.11	0.00	0.00	0.00
	E8	33.69	19.88	53.83	0.00	0.00	0.00
	E9	-11.23	-81.64	107.12	0.00	0.00	0.00
	E10	-8.76	84.68	-163.81	0.00	0.00	0.00
	E11	8.76	-84.68	163.81	0.00	0.00	0.00
	E12	11.23	81.64	-107.12	0.00	0.00	0.00
	E13	-32.32	-18.76	-52.83	0.00	0.00	0.00
	E14	-34.31	28.90	-136.11	0.00	0.00	0.00
	E15	34.31	-28.90	136.11	0.00	0.00	0.00
	E16	32.32	18.76	52.83	0.00	0.00	0.00
	E17	-6.68	-77.92	110.46	0.00	0.00	0.00
	E18	-13.31	80.96	-167.14	0.00	0.00	0.00
	E19	13.31	-80.96	167.14	0.00	0.00	0.00
	E20	6.68	77.92	-110.46	0.00	0.00	0.00
	E21	-45.60	-29.66	-75.29	0.00	0.00	0.00
	E22	-44.86	20.23	-156.57	0.00	0.00	0.00
	E23	44.86	-20.23	156.57	0.00	0.00	0.00
	E24	45.60	29.66	75.29	0.00	0.00	0.00
	E25	-14.80	-84.58	100.68	0.00	0.00	0.00
	E26	-12.34	81.75	-170.25	0.00	0.00	0.00
	E27	12.34	-81.75	170.25	0.00	0.00	0.00
	E28	14.80	84.58	-100.68	0.00	0.00	0.00
	E29	-44.23	-28.55	-74.29	0.00	0.00	0.00

Frame Reactions



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Bentley Building Code: IBC

E30	-46.22	19.12	-157.58	0.00	0.00	0.00
E31	46.22	-19.12	157.58	0.00	0.00	0.00
E32	44.23	28.55	74.29	0.00	0.00	0.00
E33	-10.25	-80.86	104.02	0.00	0.00	0.00
E34	-16.89	78.03	-173.58	0.00	0.00	0.00
E35	16.89	-78.03	173.58	0.00	0.00	0.00
E36	10.25	80.86	-104.02	0.00	0.00	0.00
E37	-93.01	0.91	-219.90	0.00	0.00	0.00
E38	1.99	-185.92	291.30	0.00	0.00	0.00
E39	-44.56	0.28	-113.19	0.00	0.00	0.00
E40	1.03	-88.64	144.89	0.00	0.00	0.00

Frame #29:

Node	LdC	Rx kips	Ry kips	Rz kips	Mxx kip-ft	Myy kip-ft	Tzz kip-ft
258	D	0.00	6.13	42.09	0.00	0.00	0.00
	Lp	0.00	8.67	43.77	0.00	0.00	0.00
	Ln	-0.00	-0.00	-0.00	0.00	0.00	0.00
	Sp	0.00	0.90	13.32	0.00	0.00	0.00
	Sn	-0.00	-0.00	-0.00	0.00	0.00	0.00
	E1	0.00	6.45	12.36	0.00	0.00	0.00
	E2	-0.00	-6.28	-7.61	0.00	0.00	0.00
	E3	-0.00	-54.10	-98.56	0.00	0.00	0.00
	E4	0.00	-49.07	-87.45	0.00	0.00	0.00
	E5	0.00	-9.78	-17.20	0.00	0.00	0.00
	E6	0.00	22.68	41.93	0.00	0.00	0.00
	E7	-0.00	-22.68	-41.93	0.00	0.00	0.00
	E8	-0.00	9.78	17.20	0.00	0.00	0.00
	E9	0.00	-52.16	-94.85	0.00	0.00	0.00
	E10	0.00	56.03	102.27	0.00	0.00	0.00
	E11	-0.00	-56.03	-102.27	0.00	0.00	0.00
	E12	-0.00	52.16	94.85	0.00	0.00	0.00
	E13	0.00	-8.27	-13.87	0.00	0.00	0.00
	E14	0.00	21.17	38.60	0.00	0.00	0.00
	E15	-0.00	-21.17	-38.60	0.00	0.00	0.00
	E16	-0.00	8.27	13.87	0.00	0.00	0.00
	E17	0.00	-47.14	-83.74	0.00	0.00	0.00
	E18	-0.00	51.01	91.16	0.00	0.00	0.00
	E19	0.00	-51.01	-91.16	0.00	0.00	0.00
	E20	-0.00	47.14	83.74	0.00	0.00	0.00

Frame Reactions



RAM Frame 24.00.01.18

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Bentley Building Code: IBC

	E21	-0.00	-22.51	-37.18	0.00	0.00	0.00
	E22	-0.00	9.95	21.95	0.00	0.00	0.00
	E23	0.00	-9.95	-21.95	0.00	0.00	0.00
	E24	0.00	22.51	37.18	0.00	0.00	0.00
	E25	-0.00	-55.98	-100.85	0.00	0.00	0.00
	E26	-0.00	52.21	96.28	0.00	0.00	0.00
	E27	0.00	-52.21	-96.28	0.00	0.00	0.00
	E28	0.00	55.98	100.85	0.00	0.00	0.00
	E29	-0.00	-21.00	-33.85	0.00	0.00	0.00
	E30	-0.00	8.44	18.62	0.00	0.00	0.00
	E31	0.00	-8.44	-18.62	0.00	0.00	0.00
	E32	0.00	21.00	33.85	0.00	0.00	0.00
	E33	0.00	-50.96	-89.74	0.00	0.00	0.00
	E34	-0.00	47.19	85.17	0.00	0.00	0.00
	E35	0.00	-47.19	-85.17	0.00	0.00	0.00
	E36	-0.00	50.96	89.74	0.00	0.00	0.00
	E37	0.00	0.92	4.85	0.00	0.00	0.00
	E38	0.00	-116.98	-193.43	0.00	0.00	0.00
	E39	0.00	0.22	2.50	0.00	0.00	0.00
	E40	0.00	-56.05	-97.46	0.00	0.00	0.00
259	D	-0.00	-5.83	45.97	0.00	0.00	0.00
	Lp	-0.00	-8.43	46.18	0.00	0.00	0.00
	Ln	0.00	-0.00	0.00	0.00	0.00	0.00
	Sp	-0.00	-0.69	15.56	0.00	0.00	0.00
	Sn	0.00	-0.00	0.00	0.00	0.00	0.00
	E1	-0.00	6.37	-12.36	0.00	0.00	0.00
	E2	0.00	-6.12	7.62	0.00	0.00	0.00
	E3	0.00	-53.34	98.56	0.00	0.00	0.00
	E4	-0.00	-48.37	87.45	0.00	0.00	0.00
	E5	-0.00	-9.63	17.21	0.00	0.00	0.00
	E6	-0.00	22.37	-41.93	0.00	0.00	0.00
	E7	0.00	-22.37	41.93	0.00	0.00	0.00
	E8	0.00	9.63	-17.21	0.00	0.00	0.00
	E9	-0.00	-51.43	94.85	0.00	0.00	0.00
	E10	-0.00	55.25	-102.27	0.00	0.00	0.00
	E11	0.00	-55.25	102.27	0.00	0.00	0.00
	E12	0.00	51.43	-94.85	0.00	0.00	0.00
	E13	-0.00	-8.14	13.87	0.00	0.00	0.00
	E14	-0.00	20.88	-38.60	0.00	0.00	0.00

Frame Reactions



RAM Frame 24.00.01.18

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Bentley Building Code: IBC

E15	0.00	-20.88	38.60	0.00	0.00	0.00
E16	0.00	8.14	-13.87	0.00	0.00	0.00
E17	-0.00	-46.46	83.74	0.00	0.00	0.00
E18	0.00	50.28	-91.16	0.00	0.00	0.00
E19	-0.00	-50.28	91.16	0.00	0.00	0.00
E20	0.00	46.46	-83.74	0.00	0.00	0.00
E21	0.00	-22.12	37.18	0.00	0.00	0.00
E22	0.00	9.88	-21.95	0.00	0.00	0.00
E23	-0.00	-9.88	21.95	0.00	0.00	0.00
E24	-0.00	22.12	-37.18	0.00	0.00	0.00
E25	0.00	-55.18	100.85	0.00	0.00	0.00
E26	0.00	51.51	-96.28	0.00	0.00	0.00
E27	-0.00	-51.51	96.28	0.00	0.00	0.00
E28	-0.00	55.18	-100.85	0.00	0.00	0.00
E29	0.00	-20.63	33.85	0.00	0.00	0.00
E30	0.00	8.39	-18.62	0.00	0.00	0.00
E31	-0.00	-8.39	18.62	0.00	0.00	0.00
E32	-0.00	20.63	-33.85	0.00	0.00	0.00
E33	-0.00	-50.20	89.74	0.00	0.00	0.00
E34	0.00	46.53	-85.17	0.00	0.00	0.00
E35	-0.00	-46.53	85.17	0.00	0.00	0.00
E36	0.00	50.20	-89.74	0.00	0.00	0.00
E37	-0.00	0.97	-4.85	0.00	0.00	0.00
E38	-0.00	-115.41	193.43	0.00	0.00	0.00
E39	-0.00	0.26	-2.50	0.00	0.00	0.00
E40	-0.00	-55.28	97.46	0.00	0.00	0.00



QUANTUM | CONSULTING ENGINEERS

**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

1015 39th AVENUE SE
PUYALLUP, WA 98374

QUANTUM JOB NUMBER: 19305.04

RAM BUILDING ANALYSIS

STEEL FRAMING AISC 360-16 CODE CHECKS

Code Check Summary



RAM Frame 24.00.01.18

DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy
Cat 4

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Steel Code: AISC360-16 LRFD

Building Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor (DL): 1.40 Scale Factor (LL): 0.50
Scale Factor (Roof): 0.50 Scale Factor (Snow): 0.20
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 8.00
Advanced Wall Mesh Criteria: Optimization Level : 3 Shape Quality Level : 0.60
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Use Reduced Stiffness for Steel Members (AISC 360): $\tau_b = 1.00$
Sign not considered for Dynamic Load Case Results.
Rigid Links Not Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

LOAD COMBINATION CRITERIA:

Live Load factor f_l (0.5 or 1.0) 0.500
Sds (for Ev) 1.006
RhoX 1.000
RhoY 1.000

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
Sp	PosSnowLoad	RAMUSER
Sn	NegSnowLoad	RAMUSER
E1	Seismic	EQ_ASCE716_X_+E_F
E2	Seismic	EQ_ASCE716_X_-E_F
E3	Seismic	EQ_ASCE716_Y_+E_F
E4	Seismic	EQ_ASCE716_Y_-E_F
E5	Seismic	EQ_ASCE716_X_+E_0.3Y_+E_F
E6	Seismic	EQ_ASCE716_X_+E_-0.3Y_+E_F
E7	Seismic	EQ_ASCE716_-X_+E_0.3Y_+E_F
E8	Seismic	EQ_ASCE716_-X_+E_-0.3Y_+E_F
E9	Seismic	EQ_ASCE716_0.3X_+E_Y_+E_F
E10	Seismic	EQ_ASCE716_0.3X_+E_-Y_+E_F
E11	Seismic	EQ_ASCE716_-0.3X_+E_Y_+E_F
E12	Seismic	EQ_ASCE716_-0.3X_+E_-Y_+E_F

Code Check Summary



RAM Frame 24.00.01.18

DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy

Cat 4

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Steel Code: AISC360-16 LRFD

Building Code: IBC

E13	Seismic	EQ_ASCE716_X_+E_0.3Y_-E_F
E14	Seismic	EQ_ASCE716_X_+E_-0.3Y_-E_F
E15	Seismic	EQ_ASCE716_-X_+E_0.3Y_-E_F
E16	Seismic	EQ_ASCE716_-X_+E_-0.3Y_-E_F
E17	Seismic	EQ_ASCE716_0.3X_+E_Y_-E_F
E18	Seismic	EQ_ASCE716_0.3X_+E_-Y_-E_F
E19	Seismic	EQ_ASCE716_-0.3X_+E_Y_-E_F
E20	Seismic	EQ_ASCE716_-0.3X_+E_-Y_-E_F
E21	Seismic	EQ_ASCE716_X_-E_0.3Y_+E_F
E22	Seismic	EQ_ASCE716_X_-E_-0.3Y_+E_F
E23	Seismic	EQ_ASCE716_-X_-E_0.3Y_+E_F
E24	Seismic	EQ_ASCE716_-X_-E_-0.3Y_+E_F
E25	Seismic	EQ_ASCE716_0.3X_-E_Y_+E_F
E26	Seismic	EQ_ASCE716_0.3X_-E_-Y_+E_F
E27	Seismic	EQ_ASCE716_-0.3X_-E_Y_+E_F
E28	Seismic	EQ_ASCE716_-0.3X_-E_-Y_+E_F
E29	Seismic	EQ_ASCE716_X_-E_0.3Y_-E_F
E30	Seismic	EQ_ASCE716_X_-E_-0.3Y_-E_F
E31	Seismic	EQ_ASCE716_-X_-E_0.3Y_-E_F
E32	Seismic	EQ_ASCE716_-X_-E_-0.3Y_-E_F
E33	Seismic	EQ_ASCE716_0.3X_-E_Y_-E_F
E34	Seismic	EQ_ASCE716_0.3X_-E_-Y_-E_F
E35	Seismic	EQ_ASCE716_-0.3X_-E_Y_-E_F
E36	Seismic	EQ_ASCE716_-0.3X_-E_-Y_-E_F

LOAD COMBINATIONS: IBC 2021 / ASCE 7-16 LRFD

1	* 1.400 D
2	* 1.200 D + 1.600 Lp + 0.500 Sp
3	* 1.200 D + 1.600 Ln + 0.500 Sn
4	* 1.200 D + 1.600 Lp
5	* 1.200 D + 1.600 Ln
6	* 1.200 D + 0.500 Lp + 1.600 Sp
7	* 1.200 D + 0.500 Ln + 1.600 Sn
8	* 1.200 D + 1.600 Sp
9	* 1.200 D + 1.600 Sn
10	* 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E1
11	* 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E2
12	* 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E3
13	* 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E4
14	* 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E5

Code Check Summary



RAM Frame 24.00.01.18

DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy

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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

15	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E6
16	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E7
17	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E8
18	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E9
19	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E10
20	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E11
21	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E12
22	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E13
23	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E14
24	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E15
25	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E16
26	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E17
27	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E18
28	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E19
29	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E20
30	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E21
31	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E22
32	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E23
33	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E24
34	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E25
35	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E26
36	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E27
37	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E28
38	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E29
39	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E30
40	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E31
41	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E32
42	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E33
43	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E34
44	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E35
45	*	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E36
46	*	1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E1
47	*	1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E2
48	*	1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E3
49	*	1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E4
50	*	1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E5
51	*	1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E6
52	*	1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E7
53	*	1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E8

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Steel Code: AISC360-16 LRFD

Building Code: IBC

54	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E9
55	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E10
56	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E11
57	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E12
58	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E13
59	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E14
60	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E15
61	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E16
62	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E17
63	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E18
64	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E19
65	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E20
66	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E21
67	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E22
68	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E23
69	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E24
70	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E25
71	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E26
72	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E27
73	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E28
74	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E29
75	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E30
76	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E31
77	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E32
78	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E33
79	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E34
80	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E35
81	* 1.401 D + 0.500 Lp + 0.200 Sp - 1.000 E36
82	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E1
83	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E2
84	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E3
85	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E4
86	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E5
87	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E6
88	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E7
89	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E8
90	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E9
91	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E10
92	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E11

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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

93	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E12
94	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E13
95	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E14
96	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E15
97	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E16
98	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E17
99	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E18
100	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E19
101	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E20
102	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E21
103	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E22
104	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E23
105	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E24
106	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E25
107	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E26
108	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E27
109	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E28
110	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E29
111	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E30
112	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E31
113	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E32
114	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E33
115	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E34
116	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E35
117	* 1.401 D + 0.500 Ln + 0.200 Sn + 1.000 E36
118	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E1
119	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E2
120	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E3
121	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E4
122	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E5
123	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E6
124	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E7
125	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E8
126	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E9
127	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E10
128	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E11
129	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E12
130	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E13
131	* 1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E14

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Steel Code: AISC360-16 LRFD

Building Code: IBC

132	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E15
133	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E16
134	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E17
135	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E18
136	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E19
137	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E20
138	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E21
139	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E22
140	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E23
141	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E24
142	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E25
143	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E26
144	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E27
145	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E28
146	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E29
147	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E30
148	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E31
149	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E32
150	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E33
151	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E34
152	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E35
153	*	1.401 D + 0.500 Ln + 0.200 Sn - 1.000 E36
154	*	1.401 D + 0.500 Lp + 1.000 E1
155	*	1.401 D + 0.500 Lp + 1.000 E2
156	*	1.401 D + 0.500 Lp + 1.000 E3
157	*	1.401 D + 0.500 Lp + 1.000 E4
158	*	1.401 D + 0.500 Lp + 1.000 E5
159	*	1.401 D + 0.500 Lp + 1.000 E6
160	*	1.401 D + 0.500 Lp + 1.000 E7
161	*	1.401 D + 0.500 Lp + 1.000 E8
162	*	1.401 D + 0.500 Lp + 1.000 E9
163	*	1.401 D + 0.500 Lp + 1.000 E10
164	*	1.401 D + 0.500 Lp + 1.000 E11
165	*	1.401 D + 0.500 Lp + 1.000 E12
166	*	1.401 D + 0.500 Lp + 1.000 E13
167	*	1.401 D + 0.500 Lp + 1.000 E14
168	*	1.401 D + 0.500 Lp + 1.000 E15
169	*	1.401 D + 0.500 Lp + 1.000 E16
170	*	1.401 D + 0.500 Lp + 1.000 E17

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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

171	*	1.401 D + 0.500 Lp + 1.000 E18
172	*	1.401 D + 0.500 Lp + 1.000 E19
173	*	1.401 D + 0.500 Lp + 1.000 E20
174	*	1.401 D + 0.500 Lp + 1.000 E21
175	*	1.401 D + 0.500 Lp + 1.000 E22
176	*	1.401 D + 0.500 Lp + 1.000 E23
177	*	1.401 D + 0.500 Lp + 1.000 E24
178	*	1.401 D + 0.500 Lp + 1.000 E25
179	*	1.401 D + 0.500 Lp + 1.000 E26
180	*	1.401 D + 0.500 Lp + 1.000 E27
181	*	1.401 D + 0.500 Lp + 1.000 E28
182	*	1.401 D + 0.500 Lp + 1.000 E29
183	*	1.401 D + 0.500 Lp + 1.000 E30
184	*	1.401 D + 0.500 Lp + 1.000 E31
185	*	1.401 D + 0.500 Lp + 1.000 E32
186	*	1.401 D + 0.500 Lp + 1.000 E33
187	*	1.401 D + 0.500 Lp + 1.000 E34
188	*	1.401 D + 0.500 Lp + 1.000 E35
189	*	1.401 D + 0.500 Lp + 1.000 E36
190	*	1.401 D + 0.500 Lp - 1.000 E1
191	*	1.401 D + 0.500 Lp - 1.000 E2
192	*	1.401 D + 0.500 Lp - 1.000 E3
193	*	1.401 D + 0.500 Lp - 1.000 E4
194	*	1.401 D + 0.500 Lp - 1.000 E5
195	*	1.401 D + 0.500 Lp - 1.000 E6
196	*	1.401 D + 0.500 Lp - 1.000 E7
197	*	1.401 D + 0.500 Lp - 1.000 E8
198	*	1.401 D + 0.500 Lp - 1.000 E9
199	*	1.401 D + 0.500 Lp - 1.000 E10
200	*	1.401 D + 0.500 Lp - 1.000 E11
201	*	1.401 D + 0.500 Lp - 1.000 E12
202	*	1.401 D + 0.500 Lp - 1.000 E13
203	*	1.401 D + 0.500 Lp - 1.000 E14
204	*	1.401 D + 0.500 Lp - 1.000 E15
205	*	1.401 D + 0.500 Lp - 1.000 E16
206	*	1.401 D + 0.500 Lp - 1.000 E17
207	*	1.401 D + 0.500 Lp - 1.000 E18
208	*	1.401 D + 0.500 Lp - 1.000 E19
209	*	1.401 D + 0.500 Lp - 1.000 E20

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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

210	*	1.401 D + 0.500 Lp - 1.000 E21
211	*	1.401 D + 0.500 Lp - 1.000 E22
212	*	1.401 D + 0.500 Lp - 1.000 E23
213	*	1.401 D + 0.500 Lp - 1.000 E24
214	*	1.401 D + 0.500 Lp - 1.000 E25
215	*	1.401 D + 0.500 Lp - 1.000 E26
216	*	1.401 D + 0.500 Lp - 1.000 E27
217	*	1.401 D + 0.500 Lp - 1.000 E28
218	*	1.401 D + 0.500 Lp - 1.000 E29
219	*	1.401 D + 0.500 Lp - 1.000 E30
220	*	1.401 D + 0.500 Lp - 1.000 E31
221	*	1.401 D + 0.500 Lp - 1.000 E32
222	*	1.401 D + 0.500 Lp - 1.000 E33
223	*	1.401 D + 0.500 Lp - 1.000 E34
224	*	1.401 D + 0.500 Lp - 1.000 E35
225	*	1.401 D + 0.500 Lp - 1.000 E36
226	*	1.401 D + 0.500 Ln + 1.000 E1
227	*	1.401 D + 0.500 Ln + 1.000 E2
228	*	1.401 D + 0.500 Ln + 1.000 E3
229	*	1.401 D + 0.500 Ln + 1.000 E4
230	*	1.401 D + 0.500 Ln + 1.000 E5
231	*	1.401 D + 0.500 Ln + 1.000 E6
232	*	1.401 D + 0.500 Ln + 1.000 E7
233	*	1.401 D + 0.500 Ln + 1.000 E8
234	*	1.401 D + 0.500 Ln + 1.000 E9
235	*	1.401 D + 0.500 Ln + 1.000 E10
236	*	1.401 D + 0.500 Ln + 1.000 E11
237	*	1.401 D + 0.500 Ln + 1.000 E12
238	*	1.401 D + 0.500 Ln + 1.000 E13
239	*	1.401 D + 0.500 Ln + 1.000 E14
240	*	1.401 D + 0.500 Ln + 1.000 E15
241	*	1.401 D + 0.500 Ln + 1.000 E16
242	*	1.401 D + 0.500 Ln + 1.000 E17
243	*	1.401 D + 0.500 Ln + 1.000 E18
244	*	1.401 D + 0.500 Ln + 1.000 E19
245	*	1.401 D + 0.500 Ln + 1.000 E20
246	*	1.401 D + 0.500 Ln + 1.000 E21
247	*	1.401 D + 0.500 Ln + 1.000 E22
248	*	1.401 D + 0.500 Ln + 1.000 E23

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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

249	*	1.401 D + 0.500 Ln + 1.000 E24
250	*	1.401 D + 0.500 Ln + 1.000 E25
251	*	1.401 D + 0.500 Ln + 1.000 E26
252	*	1.401 D + 0.500 Ln + 1.000 E27
253	*	1.401 D + 0.500 Ln + 1.000 E28
254	*	1.401 D + 0.500 Ln + 1.000 E29
255	*	1.401 D + 0.500 Ln + 1.000 E30
256	*	1.401 D + 0.500 Ln + 1.000 E31
257	*	1.401 D + 0.500 Ln + 1.000 E32
258	*	1.401 D + 0.500 Ln + 1.000 E33
259	*	1.401 D + 0.500 Ln + 1.000 E34
260	*	1.401 D + 0.500 Ln + 1.000 E35
261	*	1.401 D + 0.500 Ln + 1.000 E36
262	*	1.401 D + 0.500 Ln - 1.000 E1
263	*	1.401 D + 0.500 Ln - 1.000 E2
264	*	1.401 D + 0.500 Ln - 1.000 E3
265	*	1.401 D + 0.500 Ln - 1.000 E4
266	*	1.401 D + 0.500 Ln - 1.000 E5
267	*	1.401 D + 0.500 Ln - 1.000 E6
268	*	1.401 D + 0.500 Ln - 1.000 E7
269	*	1.401 D + 0.500 Ln - 1.000 E8
270	*	1.401 D + 0.500 Ln - 1.000 E9
271	*	1.401 D + 0.500 Ln - 1.000 E10
272	*	1.401 D + 0.500 Ln - 1.000 E11
273	*	1.401 D + 0.500 Ln - 1.000 E12
274	*	1.401 D + 0.500 Ln - 1.000 E13
275	*	1.401 D + 0.500 Ln - 1.000 E14
276	*	1.401 D + 0.500 Ln - 1.000 E15
277	*	1.401 D + 0.500 Ln - 1.000 E16
278	*	1.401 D + 0.500 Ln - 1.000 E17
279	*	1.401 D + 0.500 Ln - 1.000 E18
280	*	1.401 D + 0.500 Ln - 1.000 E19
281	*	1.401 D + 0.500 Ln - 1.000 E20
282	*	1.401 D + 0.500 Ln - 1.000 E21
283	*	1.401 D + 0.500 Ln - 1.000 E22
284	*	1.401 D + 0.500 Ln - 1.000 E23
285	*	1.401 D + 0.500 Ln - 1.000 E24
286	*	1.401 D + 0.500 Ln - 1.000 E25
287	*	1.401 D + 0.500 Ln - 1.000 E26

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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

288	* 1.401 D + 0.500 Ln - 1.000 E27
289	* 1.401 D + 0.500 Ln - 1.000 E28
290	* 1.401 D + 0.500 Ln - 1.000 E29
291	* 1.401 D + 0.500 Ln - 1.000 E30
292	* 1.401 D + 0.500 Ln - 1.000 E31
293	* 1.401 D + 0.500 Ln - 1.000 E32
294	* 1.401 D + 0.500 Ln - 1.000 E33
295	* 1.401 D + 0.500 Ln - 1.000 E34
296	* 1.401 D + 0.500 Ln - 1.000 E35
297	* 1.401 D + 0.500 Ln - 1.000 E36
298	* 1.401 D + 0.200 Sp + 1.000 E1
299	* 1.401 D + 0.200 Sp + 1.000 E2
300	* 1.401 D + 0.200 Sp + 1.000 E3
301	* 1.401 D + 0.200 Sp + 1.000 E4
302	* 1.401 D + 0.200 Sp + 1.000 E5
303	* 1.401 D + 0.200 Sp + 1.000 E6
304	* 1.401 D + 0.200 Sp + 1.000 E7
305	* 1.401 D + 0.200 Sp + 1.000 E8
306	* 1.401 D + 0.200 Sp + 1.000 E9
307	* 1.401 D + 0.200 Sp + 1.000 E10
308	* 1.401 D + 0.200 Sp + 1.000 E11
309	* 1.401 D + 0.200 Sp + 1.000 E12
310	* 1.401 D + 0.200 Sp + 1.000 E13
311	* 1.401 D + 0.200 Sp + 1.000 E14
312	* 1.401 D + 0.200 Sp + 1.000 E15
313	* 1.401 D + 0.200 Sp + 1.000 E16
314	* 1.401 D + 0.200 Sp + 1.000 E17
315	* 1.401 D + 0.200 Sp + 1.000 E18
316	* 1.401 D + 0.200 Sp + 1.000 E19
317	* 1.401 D + 0.200 Sp + 1.000 E20
318	* 1.401 D + 0.200 Sp + 1.000 E21
319	* 1.401 D + 0.200 Sp + 1.000 E22
320	* 1.401 D + 0.200 Sp + 1.000 E23
321	* 1.401 D + 0.200 Sp + 1.000 E24
322	* 1.401 D + 0.200 Sp + 1.000 E25
323	* 1.401 D + 0.200 Sp + 1.000 E26
324	* 1.401 D + 0.200 Sp + 1.000 E27
325	* 1.401 D + 0.200 Sp + 1.000 E28
326	* 1.401 D + 0.200 Sp + 1.000 E29

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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

327	* 1.401 D + 0.200 Sp + 1.000 E30
328	* 1.401 D + 0.200 Sp + 1.000 E31
329	* 1.401 D + 0.200 Sp + 1.000 E32
330	* 1.401 D + 0.200 Sp + 1.000 E33
331	* 1.401 D + 0.200 Sp + 1.000 E34
332	* 1.401 D + 0.200 Sp + 1.000 E35
333	* 1.401 D + 0.200 Sp + 1.000 E36
334	* 1.401 D + 0.200 Sp - 1.000 E1
335	* 1.401 D + 0.200 Sp - 1.000 E2
336	* 1.401 D + 0.200 Sp - 1.000 E3
337	* 1.401 D + 0.200 Sp - 1.000 E4
338	* 1.401 D + 0.200 Sp - 1.000 E5
339	* 1.401 D + 0.200 Sp - 1.000 E6
340	* 1.401 D + 0.200 Sp - 1.000 E7
341	* 1.401 D + 0.200 Sp - 1.000 E8
342	* 1.401 D + 0.200 Sp - 1.000 E9
343	* 1.401 D + 0.200 Sp - 1.000 E10
344	* 1.401 D + 0.200 Sp - 1.000 E11
345	* 1.401 D + 0.200 Sp - 1.000 E12
346	* 1.401 D + 0.200 Sp - 1.000 E13
347	* 1.401 D + 0.200 Sp - 1.000 E14
348	* 1.401 D + 0.200 Sp - 1.000 E15
349	* 1.401 D + 0.200 Sp - 1.000 E16
350	* 1.401 D + 0.200 Sp - 1.000 E17
351	* 1.401 D + 0.200 Sp - 1.000 E18
352	* 1.401 D + 0.200 Sp - 1.000 E19
353	* 1.401 D + 0.200 Sp - 1.000 E20
354	* 1.401 D + 0.200 Sp - 1.000 E21
355	* 1.401 D + 0.200 Sp - 1.000 E22
356	* 1.401 D + 0.200 Sp - 1.000 E23
357	* 1.401 D + 0.200 Sp - 1.000 E24
358	* 1.401 D + 0.200 Sp - 1.000 E25
359	* 1.401 D + 0.200 Sp - 1.000 E26
360	* 1.401 D + 0.200 Sp - 1.000 E27
361	* 1.401 D + 0.200 Sp - 1.000 E28
362	* 1.401 D + 0.200 Sp - 1.000 E29
363	* 1.401 D + 0.200 Sp - 1.000 E30
364	* 1.401 D + 0.200 Sp - 1.000 E31
365	* 1.401 D + 0.200 Sp - 1.000 E32

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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

366	* 1.401 D + 0.200 Sp - 1.000 E33
367	* 1.401 D + 0.200 Sp - 1.000 E34
368	* 1.401 D + 0.200 Sp - 1.000 E35
369	* 1.401 D + 0.200 Sp - 1.000 E36
370	* 1.401 D + 0.200 Sn + 1.000 E1
371	* 1.401 D + 0.200 Sn + 1.000 E2
372	* 1.401 D + 0.200 Sn + 1.000 E3
373	* 1.401 D + 0.200 Sn + 1.000 E4
374	* 1.401 D + 0.200 Sn + 1.000 E5
375	* 1.401 D + 0.200 Sn + 1.000 E6
376	* 1.401 D + 0.200 Sn + 1.000 E7
377	* 1.401 D + 0.200 Sn + 1.000 E8
378	* 1.401 D + 0.200 Sn + 1.000 E9
379	* 1.401 D + 0.200 Sn + 1.000 E10
380	* 1.401 D + 0.200 Sn + 1.000 E11
381	* 1.401 D + 0.200 Sn + 1.000 E12
382	* 1.401 D + 0.200 Sn + 1.000 E13
383	* 1.401 D + 0.200 Sn + 1.000 E14
384	* 1.401 D + 0.200 Sn + 1.000 E15
385	* 1.401 D + 0.200 Sn + 1.000 E16
386	* 1.401 D + 0.200 Sn + 1.000 E17
387	* 1.401 D + 0.200 Sn + 1.000 E18
388	* 1.401 D + 0.200 Sn + 1.000 E19
389	* 1.401 D + 0.200 Sn + 1.000 E20
390	* 1.401 D + 0.200 Sn + 1.000 E21
391	* 1.401 D + 0.200 Sn + 1.000 E22
392	* 1.401 D + 0.200 Sn + 1.000 E23
393	* 1.401 D + 0.200 Sn + 1.000 E24
394	* 1.401 D + 0.200 Sn + 1.000 E25
395	* 1.401 D + 0.200 Sn + 1.000 E26
396	* 1.401 D + 0.200 Sn + 1.000 E27
397	* 1.401 D + 0.200 Sn + 1.000 E28
398	* 1.401 D + 0.200 Sn + 1.000 E29
399	* 1.401 D + 0.200 Sn + 1.000 E30
400	* 1.401 D + 0.200 Sn + 1.000 E31
401	* 1.401 D + 0.200 Sn + 1.000 E32
402	* 1.401 D + 0.200 Sn + 1.000 E33
403	* 1.401 D + 0.200 Sn + 1.000 E34
404	* 1.401 D + 0.200 Sn + 1.000 E35

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405	* 1.401 D + 0.200 Sn + 1.000 E36
406	* 1.401 D + 0.200 Sn - 1.000 E1
407	* 1.401 D + 0.200 Sn - 1.000 E2
408	* 1.401 D + 0.200 Sn - 1.000 E3
409	* 1.401 D + 0.200 Sn - 1.000 E4
410	* 1.401 D + 0.200 Sn - 1.000 E5
411	* 1.401 D + 0.200 Sn - 1.000 E6
412	* 1.401 D + 0.200 Sn - 1.000 E7
413	* 1.401 D + 0.200 Sn - 1.000 E8
414	* 1.401 D + 0.200 Sn - 1.000 E9
415	* 1.401 D + 0.200 Sn - 1.000 E10
416	* 1.401 D + 0.200 Sn - 1.000 E11
417	* 1.401 D + 0.200 Sn - 1.000 E12
418	* 1.401 D + 0.200 Sn - 1.000 E13
419	* 1.401 D + 0.200 Sn - 1.000 E14
420	* 1.401 D + 0.200 Sn - 1.000 E15
421	* 1.401 D + 0.200 Sn - 1.000 E16
422	* 1.401 D + 0.200 Sn - 1.000 E17
423	* 1.401 D + 0.200 Sn - 1.000 E18
424	* 1.401 D + 0.200 Sn - 1.000 E19
425	* 1.401 D + 0.200 Sn - 1.000 E20
426	* 1.401 D + 0.200 Sn - 1.000 E21
427	* 1.401 D + 0.200 Sn - 1.000 E22
428	* 1.401 D + 0.200 Sn - 1.000 E23
429	* 1.401 D + 0.200 Sn - 1.000 E24
430	* 1.401 D + 0.200 Sn - 1.000 E25
431	* 1.401 D + 0.200 Sn - 1.000 E26
432	* 1.401 D + 0.200 Sn - 1.000 E27
433	* 1.401 D + 0.200 Sn - 1.000 E28
434	* 1.401 D + 0.200 Sn - 1.000 E29
435	* 1.401 D + 0.200 Sn - 1.000 E30
436	* 1.401 D + 0.200 Sn - 1.000 E31
437	* 1.401 D + 0.200 Sn - 1.000 E32
438	* 1.401 D + 0.200 Sn - 1.000 E33
439	* 1.401 D + 0.200 Sn - 1.000 E34
440	* 1.401 D + 0.200 Sn - 1.000 E35
441	* 1.401 D + 0.200 Sn - 1.000 E36
442	* 1.401 D + 1.000 E1
443	* 1.401 D + 1.000 E2

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Bentley Building Code: IBC

444	*	1.401 D + 1.000 E3
445	*	1.401 D + 1.000 E4
446	*	1.401 D + 1.000 E5
447	*	1.401 D + 1.000 E6
448	*	1.401 D + 1.000 E7
449	*	1.401 D + 1.000 E8
450	*	1.401 D + 1.000 E9
451	*	1.401 D + 1.000 E10
452	*	1.401 D + 1.000 E11
453	*	1.401 D + 1.000 E12
454	*	1.401 D + 1.000 E13
455	*	1.401 D + 1.000 E14
456	*	1.401 D + 1.000 E15
457	*	1.401 D + 1.000 E16
458	*	1.401 D + 1.000 E17
459	*	1.401 D + 1.000 E18
460	*	1.401 D + 1.000 E19
461	*	1.401 D + 1.000 E20
462	*	1.401 D + 1.000 E21
463	*	1.401 D + 1.000 E22
464	*	1.401 D + 1.000 E23
465	*	1.401 D + 1.000 E24
466	*	1.401 D + 1.000 E25
467	*	1.401 D + 1.000 E26
468	*	1.401 D + 1.000 E27
469	*	1.401 D + 1.000 E28
470	*	1.401 D + 1.000 E29
471	*	1.401 D + 1.000 E30
472	*	1.401 D + 1.000 E31
473	*	1.401 D + 1.000 E32
474	*	1.401 D + 1.000 E33
475	*	1.401 D + 1.000 E34
476	*	1.401 D + 1.000 E35
477	*	1.401 D + 1.000 E36
478	*	1.401 D - 1.000 E1
479	*	1.401 D - 1.000 E2
480	*	1.401 D - 1.000 E3
481	*	1.401 D - 1.000 E4
482	*	1.401 D - 1.000 E5

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Bentley Building Code: IBC

483	* 1.401 D - 1.000 E6
484	* 1.401 D - 1.000 E7
485	* 1.401 D - 1.000 E8
486	* 1.401 D - 1.000 E9
487	* 1.401 D - 1.000 E10
488	* 1.401 D - 1.000 E11
489	* 1.401 D - 1.000 E12
490	* 1.401 D - 1.000 E13
491	* 1.401 D - 1.000 E14
492	* 1.401 D - 1.000 E15
493	* 1.401 D - 1.000 E16
494	* 1.401 D - 1.000 E17
495	* 1.401 D - 1.000 E18
496	* 1.401 D - 1.000 E19
497	* 1.401 D - 1.000 E20
498	* 1.401 D - 1.000 E21
499	* 1.401 D - 1.000 E22
500	* 1.401 D - 1.000 E23
501	* 1.401 D - 1.000 E24
502	* 1.401 D - 1.000 E25
503	* 1.401 D - 1.000 E26
504	* 1.401 D - 1.000 E27
505	* 1.401 D - 1.000 E28
506	* 1.401 D - 1.000 E29
507	* 1.401 D - 1.000 E30
508	* 1.401 D - 1.000 E31
509	* 1.401 D - 1.000 E32
510	* 1.401 D - 1.000 E33
511	* 1.401 D - 1.000 E34
512	* 1.401 D - 1.000 E35
513	* 1.401 D - 1.000 E36
514	* 0.699 D + 1.000 E1
515	* 0.699 D + 1.000 E2
516	* 0.699 D + 1.000 E3
517	* 0.699 D + 1.000 E4
518	* 0.699 D + 1.000 E5
519	* 0.699 D + 1.000 E6
520	* 0.699 D + 1.000 E7
521	* 0.699 D + 1.000 E8

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Bentley Building Code: IBC

522	* 0.699 D + 1.000 E9
523	* 0.699 D + 1.000 E10
524	* 0.699 D + 1.000 E11
525	* 0.699 D + 1.000 E12
526	* 0.699 D + 1.000 E13
527	* 0.699 D + 1.000 E14
528	* 0.699 D + 1.000 E15
529	* 0.699 D + 1.000 E16
530	* 0.699 D + 1.000 E17
531	* 0.699 D + 1.000 E18
532	* 0.699 D + 1.000 E19
533	* 0.699 D + 1.000 E20
534	* 0.699 D + 1.000 E21
535	* 0.699 D + 1.000 E22
536	* 0.699 D + 1.000 E23
537	* 0.699 D + 1.000 E24
538	* 0.699 D + 1.000 E25
539	* 0.699 D + 1.000 E26
540	* 0.699 D + 1.000 E27
541	* 0.699 D + 1.000 E28
542	* 0.699 D + 1.000 E29
543	* 0.699 D + 1.000 E30
544	* 0.699 D + 1.000 E31
545	* 0.699 D + 1.000 E32
546	* 0.699 D + 1.000 E33
547	* 0.699 D + 1.000 E34
548	* 0.699 D + 1.000 E35
549	* 0.699 D + 1.000 E36
550	* 0.699 D - 1.000 E1
551	* 0.699 D - 1.000 E2
552	* 0.699 D - 1.000 E3
553	* 0.699 D - 1.000 E4
554	* 0.699 D - 1.000 E5
555	* 0.699 D - 1.000 E6
556	* 0.699 D - 1.000 E7
557	* 0.699 D - 1.000 E8
558	* 0.699 D - 1.000 E9
559	* 0.699 D - 1.000 E10
560	* 0.699 D - 1.000 E11

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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

561	*	0.699 D - 1.000 E12
562	*	0.699 D - 1.000 E13
563	*	0.699 D - 1.000 E14
564	*	0.699 D - 1.000 E15
565	*	0.699 D - 1.000 E16
566	*	0.699 D - 1.000 E17
567	*	0.699 D - 1.000 E18
568	*	0.699 D - 1.000 E19
569	*	0.699 D - 1.000 E20
570	*	0.699 D - 1.000 E21
571	*	0.699 D - 1.000 E22
572	*	0.699 D - 1.000 E23
573	*	0.699 D - 1.000 E24
574	*	0.699 D - 1.000 E25
575	*	0.699 D - 1.000 E26
576	*	0.699 D - 1.000 E27
577	*	0.699 D - 1.000 E28
578	*	0.699 D - 1.000 E29
579	*	0.699 D - 1.000 E30
580	*	0.699 D - 1.000 E31
581	*	0.699 D - 1.000 E32
582	*	0.699 D - 1.000 E33
583	*	0.699 D - 1.000 E34
584	*	0.699 D - 1.000 E35
585	*	0.699 D - 1.000 E36

* = Load combination currently selected to use

Frame #1:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
36	50.20	0.00	0.00	-0.03	-0.01	25	0.10 Axial	36	W12X65
56	57.22	0.00	0.00	-0.06	0.03	6	0.12 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
2	-2.07	0.08	0.00	37.92	0.06	25	0.18 Shear	36	W24X68
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size

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39	-61.11	0.00	0.00	0.00	0.00	169	0.87 Axial	39	CoreBRB_2.00
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Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
-----------	-----------	---------------	---------------	------------	------------	----	-----------	-----------	------

6	100.49	0.00	0.00	0.00	0.00	25	0.23 H1-1a	36	W12X65
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48	172.32	0.00	0.00	0.00	-0.00	2	0.40 Axial	36	W12X65
----	--------	------	------	------	-------	---	------------	----	--------

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
-----------	-----------	---------------	---------------	------------	------------	----	-----------	-----------	------

3	0.00	370.38	0.00	48.77	-0.00	2	0.87 H1-1b	50	W24X55
---	------	--------	------	-------	-------	---	------------	----	--------

Frame #2:

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
-----------	-----------	---------------	---------------	------------	------------	----	-----------	-----------	------

82	162.47	0.00	0.00	-0.00	-0.00	2	0.38 Axial	36	W12X65
----	--------	------	------	-------	-------	---	------------	----	--------

108	93.67	0.00	0.00	-0.00	-0.00	2	0.22 Axial	36	W12X65
-----	-------	------	------	-------	-------	---	------------	----	--------

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
-----------	-----------	---------------	---------------	------------	------------	----	-----------	-----------	------

2520	118.38	-66.14	0.00	-37.14	0.00	22	0.53 H1-3a (H1-1a)	50	W18X35
------	--------	--------	------	--------	------	----	-----------------------	----	--------

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
------------	-----------	---------------	---------------	------------	------------	----	-----------	-----------	------

37	253.95	0.00	0.00	0.00	0.00	25	0.96 Axial	39	CoreBRB_7.50
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38	257.60	0.00	0.00	0.00	0.00	22	0.98 Axial	39	CoreBRB_7.50
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Frame #3:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
-----------	-----------	---------------	---------------	------------	------------	----	-----------	-----------	------

32	30.43	0.00	0.00	-0.03	-0.02	6	0.06 Axial	36	W12X65
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58	54.17	0.00	0.00	-0.06	0.02	6	0.11 Axial	36	W12X65
----	-------	------	------	-------	------	---	------------	----	--------

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
-----------	-----------	---------------	---------------	------------	------------	----	-----------	-----------	------

17	-0.42	27.42	-0.01	-6.85	-0.00	6	0.11 H1-1b	36	W21X44
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Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
------------	-----------	---------------	---------------	------------	------------	----	-----------	-----------	------

95	64.05	0.00	0.00	0.00	0.00	25	0.83 Axial	38	CoreBRB_2.25
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96	59.33	0.00	0.00	0.00	0.00	310	0.77	Axial	38	CoreBRB_2.25
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Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
10	113.06	0.00	0.00	0.00	0.00	25	0.26 H1-1a	36	W12X65
51	183.31	0.00	0.00	-0.00	0.00	2	0.43 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
19	56.30	-42.86	0.00	-24.38	0.00	25	0.20 H1-1b	50	W21X44
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
39	127.53	0.00	0.00	0.00	0.00	169	0.97 Axial	39	CoreBRB_3.75
40	128.24	0.00	0.00	0.00	0.00	22	0.97 Axial	39	CoreBRB_3.75

Frame #4:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
74	78.56	0.00	0.00	-0.10	-0.01	8	0.16 Axial	36	W12X65
78	70.17	0.00	0.00	0.07	-0.02	37	0.15 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
20	31.25	12.45	0.16	6.72	0.00	22	0.10 H1-1b	36	W21X44
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
91	62.27	0.00	0.00	0.00	0.00	313	0.81 Axial	38	CoreBRB_2.25
92	66.44	0.00	0.00	0.00	0.00	22	0.86 Axial	38	CoreBRB_2.25

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
84	207.64	0.00	0.00	-0.00	-0.00	2	0.48 Axial	36	W12X65
112	180.74	0.36	-0.01	-0.04	0.01	2	0.42 H1-1a	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
22	48.26	-45.33	0.00	-24.65	-0.00	15	0.20 H1-1b	50	W21X44
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size

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#	kip	kip-ft	kip-ft	kip	kip				ksi	
41	111.98	0.00	0.00	0.00	0.00	169	0.98	Axial	39	CoreBRB_3.25
42	113.16	0.00	0.00	0.00	0.00	22	0.99	Axial	39	CoreBRB_3.25

Frame #6:

Level: Roof

Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
69	67.22	0.00	0.00	0.01	0.12	6	0.14 Axial	36	W12X65
79	26.28	0.00	0.00	0.04	-0.03	6	0.05 Axial	36	W12X65
90	39.88	0.00	0.00	0.06	-0.04	6	0.08 Axial	36	W12X65

Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
198	-0.28	33.94	0.00	-7.94	0.00	8	0.13 H1-1b	36	W21X44
199	0.69	30.89	-0.00	8.43	0.00	6	0.12 H1-3a (H1-1b)	36	W21X44

Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
6	64.32	0.00	0.00	0.00	0.00	304	0.67 Axial	39	CoreBRB_2.75
7	66.04	0.00	0.00	0.00	0.00	15	0.68 Axial	39	CoreBRB_2.75
41	51.63	0.00	0.00	0.00	0.00	304	0.75 Axial	38	CoreBRB_2.00
42	52.38	0.00	0.00	0.00	0.00	15	0.77 Axial	38	CoreBRB_2.00

Level: 2nd Floor

Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
86	226.18	0.30	0.03	-0.07	0.01	2	0.53 H1-1a	36	W12X65
103	212.10	0.27	0.04	-0.04	0.01	2	0.49 H1-1a	36	W12X65
124	193.71	0.25	0.04	-0.03	0.01	2	0.45 H1-1a	36	W12X65

Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
302	38.93	0.00	0.00	-4.81	0.00	22	0.14 Axial	50	W18X35
303	38.93	0.00	0.00	-4.72	0.00	22	0.14 Axial	50	W18X35

Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
45	77.92	0.00	0.00	0.00	0.00	169	0.74 Axial	39	CoreBRB_3.00
46	79.16	0.00	0.00	0.00	0.00	22	0.75 Axial	39	CoreBRB_3.00
93	78.24	0.00	0.00	0.00	0.00	169	0.74 Axial	39	CoreBRB_3.00

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Steel Code: AISC360-16 LRFD

94	78.98	0.00	0.00	0.00	0.00	22	0.75	Axial	39	CoreBRB_3.00
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Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

49	71.76	0.00	0.00	0.00	0.00	169	0.74	Axial	39	CoreBRB_2.75
50	72.67	0.00	0.00	0.00	0.00	22	0.75	Axial	39	CoreBRB_2.75

Frame #9:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
71	67.63	0.00	0.00	0.04	-0.03	6	0.14 Axial	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
266	2.28	33.48	0.01	8.01	0.00	6	0.13 H1-3a (H1-1b)	36	W21X44

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
8	46.81	0.00	0.00	0.00	0.00	304	0.67 Axial	39	CoreBRB_2.00
9	52.82	0.00	0.00	0.00	0.00	15	0.75 Axial	39	CoreBRB_2.00

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
88	222.74	0.23	0.03	-0.05	0.00	2	0.52 H1-1a	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
319	33.23	0.00	0.00	-4.51	0.00	22	0.12 Axial	50	W18X35

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
113	67.07	0.00	0.00	0.00	0.00	169	0.71 Axial	38	CoreBRB_2.75
114	67.81	0.00	0.00	0.00	0.00	22	0.72 Axial	38	CoreBRB_2.75

Frame #10:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
53	112.65	0.02	-0.01	-0.14	-0.03	6	0.23 H1-1a	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
899	40.91	0.00	0.00	-8.40	0.00	32	0.05 Axial	50	W27X94

Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

#	kip	kip-ft	kip-ft	kip	kip			ksi	
89	71.90	0.00	0.00	0.00	0.00	32	0.76 Axial	38	CoreBRB_2.75
90	61.79	0.00	0.00	0.00	0.00	319	0.66 Axial	38	CoreBRB_2.75

Level: 2nd Floor

Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
45	163.51	0.00	0.00	0.00	0.00	29	0.38 H1-1a	36	W12X65
56	267.02	0.00	0.00	-0.00	0.00	2	0.62 Axial	36	W12X65
Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
367	35.48	0.00	0.00	-4.78	0.00	39	0.13 Axial	50	W18X35
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
51	71.66	0.00	0.00	0.00	0.00	184	0.74 Axial	39	CoreBRB_2.75
52	72.51	0.00	0.00	0.00	0.00	39	0.75 Axial	39	CoreBRB_2.75

Frame #11:

Level: Roof

Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
19	62.90	0.00	0.00	-0.06	0.01	6	0.13 Axial	36	W12X65
147	39.34	0.00	0.00	-0.04	0.01	6	0.08 Axial	36	W12X65
Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
656	-0.77	31.11	0.00	-8.39	-0.00	6	0.12 H1-1b	36	W21X44
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
35	62.18	0.00	0.00	0.00	0.00	328	0.71 Axial	39	CoreBRB_2.50
36	64.60	0.00	0.00	0.00	0.00	39	0.74 Axial	39	CoreBRB_2.50

Level: 2nd Floor

Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
32	222.84	0.00	0.00	0.00	0.00	2	0.52 Axial	36	W12X65
43	211.43	0.00	0.00	-0.00	0.00	2	0.49 Axial	36	W12X65
Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	

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1084	26.22	-13.12	0.00	-4.70	0.00	39	0.10	H1-1b	50	W18X35
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.		Fy ksi	Size
109	53.88	0.00	0.00	0.00	0.00	184	0.79	Axial	38	CoreBRB_2.00
110	54.29	0.00	0.00	0.00	0.00	39	0.79	Axial	38	CoreBRB_2.00

Frame #12:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.		Fy ksi	Size
44	54.92	0.00	0.00	-0.05	0.01	6	0.11	Axial	36	W12X65
173	54.77	-0.00	0.00	-0.05	-0.01	6	0.34	H1-1a	50	HSS6X6X5/16

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.		Fy ksi	Size
63	210.93	0.00	0.00	0.00	0.00	2	0.49	Axial	36	W12X65
69	210.78	0.00	0.00	-0.00	0.00	2	0.49	Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.		Fy ksi	Size
1086	38.21	0.00	0.00	-4.77	0.00	39	0.14	Axial	50	W18X35
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.		Fy ksi	Size
91	77.05	0.00	0.00	0.00	0.00	184	0.80	Axial	39	CoreBRB_2.75
92	77.63	0.00	0.00	0.00	0.00	39	0.80	Axial	39	CoreBRB_2.75

Frame #13:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.		Fy ksi	Size
82	39.55	0.00	0.00	-0.04	-0.01	6	0.08	Axial	36	W12X65
85	39.54	0.00	0.00	-0.04	-0.01	6	0.08	Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.		Fy ksi	Size
335	-0.11	32.40	0.00	8.18	-0.00	6	0.13	H1-1b	36	W21X44
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.		Fy ksi	Size
37	57.41	0.00	0.00	0.00	0.00	328	0.73	Axial	39	CoreBRB_2.25

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38	58.28	0.00	0.00	0.00	0.00	39	0.74 Axial	39	CoreBRB_2.25
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Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
102	209.79	0.00	0.00	0.00	0.00	2	0.49 Axial	36	W12X65
129	210.24	0.00	0.00	-0.00	-0.00	2	0.49 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
1089	26.22	-12.66	0.00	-4.66	0.00	39	0.10 H1-1b	50	W18X35
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
111	53.85	0.00	0.00	0.00	0.00	184	0.79 Axial	38	CoreBRB_2.00
112	54.25	0.00	0.00	0.00	0.00	39	0.79 Axial	38	CoreBRB_2.00

Frame #14:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
87	40.81	0.00	0.00	0.09	-0.02	6	0.08 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
889	40.35	0.00	0.00	-7.67	-0.00	534	0.05 Axial	50	W27X94
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
59	70.46	0.00	0.00	0.00	0.00	33	0.75 Axial	38	CoreBRB_2.75
60	68.41	0.00	0.00	0.00	0.00	318	0.73 Axial	38	CoreBRB_2.75

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
127	208.96	0.17	-0.05	-0.07	0.05	2	0.49 H1-1a	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
1092	46.34	0.00	0.00	-4.64	0.00	39	0.17 Axial	50	W18X35
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
53	92.89	0.00	0.00	0.00	0.00	184	0.81 Axial	39	CoreBRB_3.25

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54	93.33	0.00	0.00	0.00	0.00	39	0.82 Axial	39	CoreBRB_3.25
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Frame #15:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
16	35.62	0.00	0.00	0.08	0.12	6	0.07 Axial	36	W12X65
41	36.07	0.00	0.00	0.06	0.04	6	0.07 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
478	-0.32	27.39	0.00	7.58	0.00	6	0.11 H1-1b	36	W21X44
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
10	64.49	0.00	0.00	0.00	0.00	328	0.61 Axial	39	CoreBRB_3.00
11	65.79	0.00	0.00	0.00	0.00	39	0.62 Axial	39	CoreBRB_3.00

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
36	168.91	0.11	-0.03	-0.07	-0.08	2	0.39 H1-1a	36	W12X65
39	164.13	0.12	-0.03	-0.04	-0.07	2	0.38 H1-1a	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
1100	63.56	-13.58	0.00	-6.31	-0.00	39	0.28 H1-1a	50	W18X35
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
55	131.59	0.00	0.00	0.00	0.00	184	0.88 Axial	39	CoreBRB_4.25
56	132.00	0.00	0.00	0.00	0.00	39	0.88 Axial	39	CoreBRB_4.25

Frame #16:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
63	31.75	0.00	0.00	-0.03	-0.00	6	0.07 Axial	36	W12X65
83	23.63	0.00	0.00	-0.02	-0.00	6	0.05 Axial	36	W12X65
84	23.63	0.00	0.00	-0.02	-0.00	6	0.05 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size

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473	19.03	11.81	-0.26	-4.90	-0.00	39	0.08 H1-1b	36	W21X44
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
12	38.52	0.00	0.00	0.00	0.00	328	0.88 Axial	39	CoreBRB_1.25
13	38.88	0.00	0.00	0.00	0.00	39	0.89 Axial	39	CoreBRB_1.25

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
97	112.75	0.00	0.00	0.00	0.00	2	0.26 Axial	36	W12X65
98	112.47	0.00	0.00	-0.00	0.00	2	0.26 Axial	36	W12X65
125	116.75	0.00	0.00	-0.00	0.00	2	0.27 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
1095	0.00	32.10	0.00	-5.14	0.00	2	0.14 H1-1b	50	W18X35
1096	63.35	-10.16	0.00	-3.73	0.00	40	0.27 H1-1a	50	W18X35
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
57	125.90	0.00	0.00	0.00	0.00	184	0.84 Axial	39	CoreBRB_4.25
58	126.13	0.00	0.00	0.00	0.00	39	0.85 Axial	39	CoreBRB_4.25

Frame #17:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
42	19.72	0.00	0.00	0.05	0.12	6	0.04 Axial	36	W12X65
43	19.67	0.00	0.00	0.02	-0.13	6	0.04 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
480	-0.07	18.11	-0.00	3.66	0.00	6	0.07 H1-1b	36	W21X44
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
16	28.06	0.00	0.00	0.00	0.00	40	0.80 Axial	39	CoreBRB_1.00
17	27.92	0.00	0.00	0.00	0.00	327	0.80 Axial	39	CoreBRB_1.00

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
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38	65.46	0.00	0.00	-0.05	-0.02	40	0.15	Axial	36	W12X65
58	78.49	0.00	0.00	-0.05	0.03	2	0.18	Axial	36	W12X65
Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.		Fy	Size
#	kip	kip-ft	kip-ft	kip	kip				ksi	
1104	57.94	-37.83	0.00	-23.32	0.00	39	0.31	H1-3b	50	W18X35
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.		Fy	Size
#	kip	kip-ft	kip-ft	kip	kip				ksi	
59	129.62	0.00	0.00	0.00	0.00	184	0.98	Axial	39	CoreBRB_3.75
60	129.91	0.00	0.00	0.00	0.00	39	0.99	Axial	39	CoreBRB_3.75

Frame #18:

Level: Roof

Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.		Fy	Size
#	kip	kip-ft	kip-ft	kip	kip				ksi	
33	10.54	0.00	0.00	-0.02	0.02	6	0.02	Axial	36	W12X65
35	36.86	0.00	0.00	-0.05	-0.02	6	0.08	Axial	36	W12X65
Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.		Fy	Size
#	kip	kip-ft	kip-ft	kip	kip				ksi	
14	20.92	8.15	0.43	6.96	-0.01	314	0.08	H1-1b	36	W21X44
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.		Fy	Size
#	kip	kip-ft	kip-ft	kip	kip				ksi	
26	48.87	0.00	0.00	0.00	0.00	29	0.71	Axial	38	CoreBRB_2.00
27	46.03	0.00	0.00	0.00	0.00	314	0.67	Axial	38	CoreBRB_2.00

Level: 2nd Floor

Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.		Fy	Size
#	kip	kip-ft	kip-ft	kip	kip				ksi	
7	184.42	0.00	0.00	0.00	0.00	2	0.43	Axial	36	W12X65
8	96.14	0.00	0.16	0.00	-0.02	29	0.22	H1-1a	36	W12X65
Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.		Fy	Size
#	kip	kip-ft	kip-ft	kip	kip				ksi	
33	31.92	-18.77	0.00	-9.96	-0.00	43	0.13	H1-1b	50	W18X35
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.		Fy	Size
#	kip	kip-ft	kip-ft	kip	kip				ksi	
73	69.49	0.00	0.00	0.00	0.00	43	0.72	Axial	39	CoreBRB_2.75
74	69.35	0.00	0.00	0.00	0.00	44	0.72	Axial	39	CoreBRB_2.75

Frame #19:

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Steel Code: AISC360-16 LRFD

Bentley Building Code: IBC

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
109	259.88	0.00	0.00	0.00	-0.00	2	0.60 Axial	36	W12X65
110	236.09	0.00	0.00	0.00	0.00	2	0.55 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
57	-0.01	-46.35	0.00	17.55	-0.00	2	0.26 H1-1b	50	W14X30
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
75	68.68	0.00	0.00	0.00	0.00	37	0.78 Axial	39	CoreBRB_2.50
76	68.63	0.00	0.00	0.00	0.00	34	0.78 Axial	39	CoreBRB_2.50

Frame #20:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
77	57.26	0.00	0.00	-0.05	-0.02	6	0.12 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
24	-27.63	0.11	-0.00	52.38	-0.11	70	0.25 Shear	36	W24X68
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
40	-86.96	0.00	0.00	0.00	0.00	181	0.55 Axial	39	CoreBRB_4.50

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
111	282.22	0.00	0.00	-0.00	-0.00	2	0.65 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
59	0.00	331.63	0.00	39.67	0.00	2	1.33 H1-1b	50	W18X35

Frame #21:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
31	60.00	0.00	0.00	-0.06	-0.02	6	0.12 Axial	36	W12X65

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37	56.87	0.00	0.00	-0.18	0.00	6	0.12	Axial	36	W12X65
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Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
11	106.81	0.00	0.00	0.00	0.00	2	0.25 Axial	36	W12X65
12	122.44	-0.04	0.12	0.12	-0.01	2	0.28 H1-1a	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
37	-0.07	63.91	0.00	-25.36	-0.00	4	0.15 H1-1b	50	W24X55

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
71	90.74	0.00	0.00	0.00	0.00	43	0.94 Axial	39	CoreBRB_2.75
72	90.96	0.00	0.00	0.00	0.00	44	0.94 Axial	39	CoreBRB_2.75

Frame #22:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
99	30.17	0.00	0.00	-0.03	-0.00	6	0.06 Axial	36	W12X65
101	28.38	0.00	0.00	0.03	-0.01	6	0.06 Axial	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
207	15.60	6.63	-0.23	-3.55	0.00	19	0.06 H1-1b	36	W21X44

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
31	30.05	0.00	0.00	0.00	0.00	19	0.86 Axial	39	CoreBRB_1.00
32	30.22	0.00	0.00	0.00	0.00	308	0.86 Axial	39	CoreBRB_1.00

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
135	92.10	162.17	0.00	-7.79	-0.00	6	0.70 H1-3a (H1-1b)	36	W12X65
136	78.71	161.66	0.00	-7.76	-0.00	6	0.69 H1-3a (H1-1b)	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
72	-0.02	60.16	0.00	-24.25	-0.00	4	0.16 H1-1b	50	W24X55

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Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
83	89.87	0.00	0.00	0.00	0.00	37	0.85 Axial	39	CoreBRB_3.00
84	89.66	0.00	0.00	0.00	0.00	34	0.85 Axial	39	CoreBRB_3.00

Frame #23:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
80	32.51	0.00	0.00	0.09	-0.02	6	0.07 Axial	36	W12X65
81	51.48	0.00	0.00	0.08	-0.03	6	0.11 Axial	36	W12X65
176	51.17	-0.00	-0.00	-0.04	0.01	6	0.29 H1-1a	46	HSS6X6X3/8

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
263	28.98	0.00	0.00	4.38	-0.00	541	0.12 Axial	36	W21X44
274	25.82	-8.39	-0.01	-4.53	-0.00	20	0.23 H1-1a	36	W21X44

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
29	46.60	0.00	0.00	0.00	0.00	322	0.76 Axial	39	CoreBRB_1.75
30	47.21	0.00	0.00	0.00	0.00	37	0.77 Axial	39	CoreBRB_1.75
63	48.57	0.00	0.00	0.00	0.00	307	0.71 Axial	38	CoreBRB_2.00
64	51.48	0.00	0.00	0.00	0.00	20	0.75 Axial	38	CoreBRB_2.00

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
104	160.80	0.25	0.03	-0.04	0.02	2	0.37 H1-1a	36	W12X65
105	111.69	0.22	0.04	-0.05	0.01	2	0.26 H1-1a	36	W12X65
106	159.21	0.00	0.00	0.00	-0.00	2	0.37 Axial	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
357	-0.02	-143.86	0.00	-48.49	-0.00	2	0.16 H1-1b	50	W27X84
358	-0.02	-143.68	0.00	-48.48	-0.00	2	0.16 H1-1b	50	W27X84

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
77	177.09	0.00	0.00	0.00	0.00	34	0.86 Axial	38	CoreBRB_6.00
78	177.77	0.00	0.00	0.00	0.00	19	0.87 Axial	38	CoreBRB_6.00
97	177.20	0.00	0.00	0.00	0.00	19	0.86 Axial	38	CoreBRB_6.00

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98	177.67	0.00	0.00	0.00	0.00	34	0.87	Axial	38	CoreBRB_6.00
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Frame #24:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
10	56.23	0.00	0.00	-0.05	-0.01	6	0.12 Axial	36	W12X65
11	57.97	0.00	0.00	-0.06	0.00	6	0.12 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
556	15.95	6.47	0.23	3.65	-0.00	26	0.07 H1-1b	36	W21X44
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
20	33.38	0.00	0.00	0.00	0.00	317	0.95 Axial	39	CoreBRB_1.00
21	33.50	0.00	0.00	0.00	0.00	26	0.95 Axial	39	CoreBRB_1.00

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
1	107.47	0.00	0.00	0.00	0.00	2	0.25 Axial	36	W12X65
2	105.77	0.00	0.00	-0.00	0.00	2	0.25 Axial	36	W12X65
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
326	-0.11	62.46	0.00	-25.60	0.00	4	0.17 H1-1b	50	W24X55
Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
65	95.18	0.00	0.00	0.00	0.00	43	0.90 Axial	39	CoreBRB_3.00
66	95.61	0.00	0.00	0.00	0.00	44	0.91 Axial	39	CoreBRB_3.00

Frame #25:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
177	36.15	0.00	0.00	-0.01	-0.03	6	0.19 Axial	50	HSS6X6X3/8
Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
897	37.76	0.00	0.00	-4.83	-0.00	547	0.12 Axial	50	W21X50
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size

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#	kip	kip-ft	kip-ft	kip	kip			ksi	
77	69.31	0.00	0.00	0.00	0.00	331	0.68 Axial	38	CoreBRB_3.00
78	72.28	0.00	0.00	0.00	0.00	44	0.70 Axial	38	CoreBRB_3.00

Level: 2nd Floor

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
338	0.18	-147.06	0.00	-48.69	-0.00	2	0.16 H1-3a (H1-1b)	50	W27X84

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
69	180.63	0.00	0.00	0.00	0.00	29	0.88 Axial	38	CoreBRB_6.00
70	181.14	0.00	0.00	0.00	0.00	44	0.88 Axial	38	CoreBRB_6.00

Frame #26:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
2	57.98	0.00	0.00	-0.06	-0.02	6	0.12 Axial	36	W12X65
3	56.24	0.00	0.00	-0.06	0.01	6	0.12 Axial	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
619	15.48	0.00	0.00	-3.64	0.00	547	0.06 Axial	36	W21X44

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
18	32.32	0.00	0.00	0.00	0.00	317	0.92 Axial	39	CoreBRB_1.00
19	32.82	0.00	0.00	0.00	0.00	26	0.94 Axial	39	CoreBRB_1.00

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
3	106.25	0.00	0.00	0.00	0.00	2	0.25 Axial	36	W12X65
4	107.00	0.00	0.00	-0.00	0.00	2	0.25 Axial	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
1126	-0.11	62.46	0.00	-25.60	0.00	4	0.17 H1-1b	50	W24X55

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
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63	95.18	0.00	0.00	0.00	0.00	43	0.90 Axial	39	CoreBRB_3.00
64	95.61	0.00	0.00	0.00	0.00	44	0.91 Axial	39	CoreBRB_3.00

Frame #27:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
39	70.30	0.00	0.00	0.11	0.02	6	0.15 Axial	36	W12X65
40	70.27	0.00	0.00	0.11	0.04	6	0.15 Axial	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
407	17.33	31.89	0.05	-9.40	0.00	26	0.15 H1-3a (H1-1b)	36	W21X44

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
22	94.59	0.00	0.00	0.00	0.00	317	0.67 Axial	39	CoreBRB_4.00
23	95.87	0.00	0.00	0.00	0.00	26	0.68 Axial	39	CoreBRB_4.00

Level: 2nd Floor

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
40	191.17	0.11	0.49	-0.04	-0.02	43	0.45 H1-1a	36	W12X65
41	192.85	0.10	0.16	-0.04	-0.00	26	0.45 H1-1a	36	W12X65

Beam #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
1138	0.18	-154.59	0.00	-49.16	-0.00	2	0.17 H1-3a (H1-1b)	50	W27X84

Brace #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
67	181.10	0.00	0.00	0.00	0.00	43	0.88 Axial	38	CoreBRB_6.00
68	181.55	0.00	0.00	0.00	0.00	44	0.88 Axial	38	CoreBRB_6.00

Frame #28:

Level: Roof

Col. #	Pu kip	Mux kip-ft	Muy kip-ft	Vux kip	Vuy kip	LC	Interact.	Fy ksi	Size
174	36.98	0.00	0.00	0.01	0.03	6	0.20 Axial	50	HSS6X6X3/8
175	51.09	-0.00	0.01	0.01	0.05	6	0.27 H1-1a	50	HSS6X6X3/8

Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
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#	kip	kip-ft	kip-ft	kip	kip			ksi	
892	22.79	0.00	0.00	-4.72	0.00	541	0.07 Axial	50	W21X50
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
81	47.21	0.00	0.00	0.00	0.00	307	0.69 Axial	38	CoreBRB_2.00
82	47.94	0.00	0.00	0.00	0.00	20	0.70 Axial	38	CoreBRB_2.00
Level: 2nd Floor									
Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
99	160.66	0.00	0.00	0.00	0.00	2	0.37 Axial	36	W12X65
100	160.18	0.00	0.00	0.00	-0.00	2	0.37 Axial	36	W12X65
Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
1156	-0.02	-149.36	0.00	-48.83	-0.00	2	0.16 H1-1b	50	W27X84
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
79	177.82	0.00	0.00	0.00	0.00	37	0.87 Axial	38	CoreBRB_6.00
80	177.58	0.00	0.00	0.00	0.00	20	0.87 Axial	38	CoreBRB_6.00
Frame #29:									
Level: Roof									
Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
108	55.91	0.00	0.00	-0.04	0.01	19	0.12 Axial	36	W12X65
109	56.95	0.00	0.00	-0.04	0.01	20	0.12 Axial	36	W12X65
Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
405	-0.59	36.64	-0.00	-4.58	0.00	8	0.16 H1-1b	36	W21X44
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
33	-44.29	0.00	0.00	0.00	0.00	164	0.72 Axial	39	CoreBRB_1.75
34	-42.85	0.00	0.00	0.00	0.00	163	0.70 Axial	39	CoreBRB_1.75
Level: 2nd Floor									
Col.	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
133	109.72	0.00	0.00	0.00	-0.00	2	0.25 Axial	36	W12X65

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134	98.97	0.00	0.00	-0.00	-0.00	2	0.23 Axial	36	W12X65
Beam	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
1164	-0.55	-95.17	0.00	32.59	0.00	2	0.19 H1-1b	50	W24X55
Brace	Pu	Mux	Muy	Vux	Vuy	LC	Interact.	Fy	Size
#	kip	kip-ft	kip-ft	kip	kip			ksi	
81	113.50	0.00	0.00	0.00	0.00	19	0.86 Axial	39	CoreBRB_3.75
82	111.26	0.00	0.00	0.00	0.00	20	0.85 Axial	39	CoreBRB_3.75



QUANTUM | CONSULTING ENGINEERS

**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

1015 39th AVENUE SE
PUYALLUP, WA 98374

QUANTUM JOB NUMBER: 19305.04

RAM BUILDING ANALYSIS

STEEL FRAMING AISC 341-16 CODE CHECKS



Seismic Provisions Member Code Check

RAM Structural System 24.00.01.18

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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: Roof

Frame No: 1

Member No: 36

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 15.90 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.03 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	15.90
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 15.90 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 86.99 OK

ALL B.F. COLUMNS TO
BE RETROFIT TO
REDUCE B/Tf AS
REQ'D

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 1

Member No: 56

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 32.93 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.07 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	32.93
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 32.93 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 81.87 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 1

Member No: 2

Fy (ksi): 36.00

Size: W24X68

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

F4.3 Non-V and Non-Inverted V-Braced Frame Unbalanced Forces

Beam must be designed for unbalanced brace axial horizontal force component on beam ends.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange $b/t_f = 7.67$ Limit = 9.27 OK

Web $h/t_w = 51.86$ Limit = 36.38 NG

Note: Beam axial load from F4.3 Analysis not determined; Web h/t_w limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

ADDITIONAL
CHEVRON CHECKS
FOR ALL FRAMES
ARE INCLUDED IN
THE FOLLOWING
SECTIONS

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 1

Member No: 39

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 125.05$ kip

Tmax = $\omega R_y P_{yscmax} = 112.66$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 125.05$ kip

Tmax = $\omega R_y P_{yscmax} = 112.66$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E16

Pu (kip) = 61.11 ϕF_{yscAsc} (kip) = 70.20 $P_u/\phi F_{yscAsc} = 0.87$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 70.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 23.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 93.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.171 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 0.846%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.380 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.318%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 125.05

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: 2nd Floor

Frame No: 1

Member No: 6

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 66.19 --- Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$

Ecl (kips) = 0.00

Max Pu/ ϕP_n = 0.15 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$	66.19
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 66.19 ϕP_y (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 71.88 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 1

Member No: 48

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 193.32 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 73.21

Max Pu/φPn = 0.45 OK

Tension: Max Pu (kip) = 21.51 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -65.95

Max Pu/φPn = 0.03 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	193.32
<i>Tension:</i>	0.699 D + 1.000 Ecl	21.51

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 193.32 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 57.15 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 1

Member No: 3

Fy (ksi): 50.00

Size: W24X55

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

F4.3 Non-V and Non-Inverted V-Braced Frame Unbalanced Forces

Beam must be designed for unbalanced brace axial horizontal force component on beam ends.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange $b/t_f = 6.94$ Limit = 9.18 **OK**

Web $h/t_w = 54.63$ Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/t_w limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 2

Member No: 82

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 123.13 --- Combination: $1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$

Ecl (kips) = 0.00

Max Pu/ ϕPn = 0.29 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$	123.13
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 123.13 ϕPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 59.41 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 2

Member No: 108

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 67.10 --- Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$

Ecl (kips) = 0.00

Max Pu/ ϕP_n = 0.16 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$	67.10
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 67.10 ϕP_y (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 71.60 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 2

Member No: 2520

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 445.05

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 498.46

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 445.05

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 498.46

Vertical Resultant (kip) = 45.8

Horizontal Resultant (kip) = 485.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 445.05

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 498.46

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 445.05

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 498.46

Vertical Resultant (kip) = 45.8

Horizontal Resultant (kip) = 485.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 100.00 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 2

Member No: 37

Fymin (ksi): 39.00

Fymax (ksi): 46.00

Size: CoreBRB_7.50 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.29$

Cmax = $\beta\omega R_y P_{yscmax} = 498.46$ kip

Tmax = $\omega R_y P_{yscmax} = 445.05$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.29$

Cmax = $\beta\omega R_y P_{yscmax} = 498.46$ kip

Tmax = $\omega R_y P_{yscmax} = 445.05$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E16

Pu (kip) = 253.95 ϕF_{yscAsc} (kip) = 263.25 Pu/ ϕF_{yscAsc} = 0.96

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 263.25

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 87.75 \leq P_{ysc} \leq 1.2 P_{ysc} = 351.00$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.314%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.313%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 498.46

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 2

Member No: 38

Fymin (ksi): 39.00

Fymax (ksi): 46.00

Size: CoreBRB_7.50 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.29$

Cmax = $\beta\omega R_y P_{yscmax} = 498.46$ kip

Tmax = $\omega R_y P_{yscmax} = 445.05$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.29$

Cmax = $\beta\omega R_y P_{yscmax} = 498.46$ kip

Tmax = $\omega R_y P_{yscmax} = 445.05$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E13

Pu (kip) = 257.60 ϕF_{yscAsc} (kip) = 263.25 Pu/ ϕF_{yscAsc} = 0.98

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 263.25

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 87.75 \leq P_{ysc} \leq 1.2 P_{ysc} = 351.00$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.314%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.313%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 498.46

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 3

Member No: 32

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 19.81 --- Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$

Ecl (kips) = 0.00

Max Pu/ ϕP_n = 0.04 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$	19.81
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 19.81 ϕP_y (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 85.81 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣM_{pc} (kip-ft)	580.80	264.60

Column Splice Shear Force Required $\Sigma M_{pc} / H$

Where ΣM_{pc} is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 3

Member No: 58

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 31.25 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.06 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	31.25
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 31.25 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 82.38 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Cat 4

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 3

Member No: 17

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 139.73

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 156.49

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 139.73

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 156.49

Vertical Resultant (kip) = 13.6

Horizontal Resultant (kip) = 173.3

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 139.73

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 156.49

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 139.73

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 156.49

Vertical Resultant (kip) = 13.6

Horizontal Resultant (kip) = 173.3

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 3

Member No: 95

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.25

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

Cmax = $\beta\omega R_y P_{yscmax} = 156.49$ kip

Tmax = $\omega R_y P_{yscmax} = 139.73$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

Cmax = $\beta\omega R_y P_{yscmax} = 156.49$ kip

Tmax = $\omega R_y P_{yscmax} = 139.73$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E16

Pu (kip) = 64.05 ϕF_{yscAsc} (kip) = 76.95 Pu/ ϕF_{yscAsc} = 0.83

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 76.95

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 25.65 \leq P_{ysc} \leq 1.2 P_{ysc} = 102.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.306 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 0.807%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.518%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 156.49

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 3

Member No: 96

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 156.49$ kip

$T_{max} = \omega R_y P_{yscmax} = 139.73$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 156.49$ kip

$T_{max} = \omega R_y P_{yscmax} = 139.73$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E13

P_u (kip) = 59.33 ϕF_{yscAsc} (kip) = 76.95 $P_u/\phi F_{yscAsc} = 0.77$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 76.95

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 25.65 \leq P_{ysc} \leq 1.2 P_{ysc} = 102.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.306 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.807%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.518%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 156.49

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 3

Member No: 10

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 189.33 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 120.12

Max Pu/φPn = 0.44 OK

Tension: Max Pu (kip) = 94.20 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -120.12

Max Pu/φPn = 0.15 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	189.33
Tension:	0.699 D + 1.000 Ecl	94.20

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 189.33 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 57.28 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 3

Member No: 51

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 242.56 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 120.12

Max Pu/φPn = 0.56 OK

Tension: Max Pu (kip) = 76.42 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -120.12

Max Pu/φPn = 0.12 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	242.56
Tension:	0.699 D + 1.000 Ecl	76.42

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 242.56 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.56 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 3

Member No: 19

Fy (ksi): 50.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 211.24

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 234.47

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 208.01

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 230.89

Vertical Resultant (kip) = 22.7

Horizontal Resultant (kip) = 227.7

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 211.24

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 234.47

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 208.01

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 230.89

Vertical Resultant (kip) = 22.7

Horizontal Resultant (kip) = 227.7

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 126.42 **OK**

Required strength of lateral brace (A-6-7) = 5.18 kip

Required stiffness of bracing (A-6-8) = 3454.81 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.18 **OK**

Web h/tw = 53.71 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



RAM Structural System 24.00.01.18

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 3

Member No: 39

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 234.47$ kip

Tmax = $\omega R_y P_{yscmax} = 211.24$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 234.47$ kip

Tmax = $\omega R_y P_{yscmax} = 211.24$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E16

Pu (kip) = 127.53 ϕF_{yscAsc} (kip) = 131.63 Pu/ ϕF_{yscAsc} = 0.97

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 131.63

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 43.88 \leq P_{ysc} \leq 1.2 P_{ysc} = 175.50$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.759 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.334%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.258%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 234.47

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 3

Member No: 40

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.29$

Cmax = $\beta\omega R_y P_{yscmax} = 230.89$ kip

Tmax = $\omega R_y P_{yscmax} = 208.01$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.29$

Cmax = $\beta\omega R_y P_{yscmax} = 230.89$ kip

Tmax = $\omega R_y P_{yscmax} = 208.01$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E13

Pu (kip) = 128.24 ϕF_{yscAsc} (kip) = 131.63 Pu/ ϕF_{yscAsc} = 0.97

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 131.63

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 43.88 \leq P_{ysc} \leq 1.2 P_{ysc} = 175.50$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.853 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.305%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.191%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 230.89

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 4

Member No: 74

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 59.83 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.12 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	59.83
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 59.83 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 73.79 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 4

Member No: 78

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 22.92 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.05 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	22.92
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 22.92 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 84.88 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 4

Member No: 20

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 139.73

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 156.49

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 139.73

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 156.49

Vertical Resultant (kip) = 13.6

Horizontal Resultant (kip) = 173.3

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 139.73

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 156.49

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 139.73

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 156.49

Vertical Resultant (kip) = 13.6

Horizontal Resultant (kip) = 173.3

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 4

Member No: 91

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 156.49$ kip

$T_{max} = \omega R_y P_{yscmax} = 139.73$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 156.49$ kip

$T_{max} = \omega R_y P_{yscmax} = 139.73$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E16

P_u (kip) = 62.27 ϕF_{yscAsc} (kip) = 76.95 $P_u/\phi F_{yscAsc} = 0.81$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 76.95

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 25.65 \leq P_{ysc} \leq 1.2 P_{ysc} = 102.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.355 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.838%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.518%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 156.49

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 4

Member No: 92

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 156.49$ kip

$T_{max} = \omega R_y P_{yscmax} = 139.73$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 156.49$ kip

$T_{max} = \omega R_y P_{yscmax} = 139.73$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E13

P_u (kip) = 66.44 ϕF_{yscAsc} (kip) = 76.95 $P_u/\phi F_{yscAsc} = 0.86$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 76.95

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 25.65 \leq P_{ysc} \leq 1.2 P_{ysc} = 102.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.356 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.838%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.518%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 156.49

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 4

Member No: 84

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 271.25 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 120.12

Max Pu/φPn = 0.63 OK

Tension: Max Pu (kip) = 62.06 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -120.12

Max Pu/φPn = 0.10 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	271.25
Tension:	0.699 D + 1.000 Ecl	62.06

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 271.25 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 54.64 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 4

Member No: 112

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 238.71 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 120.12

Max Pu/φPn = 0.55 OK

Tension: Max Pu (kip) = 78.31 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -120.12

Max Pu/φPn = 0.13 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	238.71
Tension:	0.699 D + 1.000 Ecl	78.31

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 238.71 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.69 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 4

Member No: 22

Fy (ksi): 50.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 180.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 200.11

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 180.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 200.11

Vertical Resultant (kip) = 17.0

Horizontal Resultant (kip) = 195.7

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 180.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 200.11

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 180.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 200.11

Vertical Resultant (kip) = 17.0

Horizontal Resultant (kip) = 195.7

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 126.42 **OK**

Required strength of lateral brace (A-6-7) = 5.18 kip

Required stiffness of bracing (A-6-8) = 3454.81 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.18 **OK**

Web h/tw = 53.71 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 4

Member No: 41

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.29$

Cmax = $\beta\omega R_y P_{yscmax} = 200.11$ kip

Tmax = $\omega R_y P_{yscmax} = 180.28$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.29$

Cmax = $\beta\omega R_y P_{yscmax} = 200.11$ kip

Tmax = $\omega R_y P_{yscmax} = 180.28$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E16

Pu (kip) = 111.98 ϕF_{yscAsc} (kip) = 114.07 Pu/ ϕF_{yscAsc} = 0.98

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 114.07

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 38.03 \leq P_{ysc} \leq 1.2 P_{ysc} = 152.10$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.828 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.285%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.183%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 200.11

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 4

Member No: 42

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.29$

$C_{max} = \beta\omega R_y P_{yscmax} = 200.11$ kip

$T_{max} = \omega R_y P_{yscmax} = 180.28$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.29$

$C_{max} = \beta\omega R_y P_{yscmax} = 200.11$ kip

$T_{max} = \omega R_y P_{yscmax} = 180.28$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E13

P_u (kip) = 113.16 ϕF_{yscAsc} (kip) = 114.07 $P_u/\phi F_{yscAsc} = 0.99$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 114.07

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 38.03 \leq P_{ysc} \leq 1.2 P_{ysc} = 152.10$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.828 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.285%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.183%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 200.11

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Cat 4

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 6

Member No: 69

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 37.80 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.08 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	37.80
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 37.80 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 80.41 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 6

Member No: 79

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 15.75 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.03 OK

Tension: Max Pu (kip) = 6.74 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -13.70

Max Pu/φPn = 0.01 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	15.75
Tension:	0.699 D + 1.000 Ecl	6.74

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 15.75 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 87.04 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 6

Member No: 90

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 22.26 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.05 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	22.26
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 22.26 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 85.08 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Cat 4

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 6

Member No: 198

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 139.10

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 139.10

Vertical Resultant (kip) = 12.1

Horizontal Resultant (kip) = 154.0

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 139.10

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 139.10

Vertical Resultant (kip) = 12.1

Horizontal Resultant (kip) = 154.0

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



RAM Structural System 24.00.01.18

DataBase: _Current - South Hill Bldg C II - IBC 2021 - Occupancy

Cat 4

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof Frame No: 6 Member No: 199
Fy (ksi): 36.00 Size: W21X44
Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 157.27
Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 176.15
Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 157.27
Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 176.15
Vertical Resultant (kip) = 15.3
Horizontal Resultant (kip) = 195.0

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 157.27
Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 176.15
Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 157.27
Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 176.15
Vertical Resultant (kip) = 15.3
Horizontal Resultant (kip) = 195.0
Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 6

Member No: 6

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.33$

Cmax = $\beta\omega R_y P_{yscmax} = 176.15$ kip

Tmax = $\omega R_y P_{yscmax} = 157.27$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.33$

Cmax = $\beta\omega R_y P_{yscmax} = 176.15$ kip

Tmax = $\omega R_y P_{yscmax} = 157.27$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E7

Pu (kip) = 64.32 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.67$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.200 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.736%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.506%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 176.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



RAM Structural System 24.00.01.18

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 6

Member No: 7

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.33$

$C_{max} = \beta\omega R_y P_{yscmax} = 176.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 157.27$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.33$

$C_{max} = \beta\omega R_y P_{yscmax} = 176.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 157.27$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E6

P_u (kip) = 66.04 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.68$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.200 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.736%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.506%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 176.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



RAM Structural System 24.00.01.18

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 6

Member No: 41

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 139.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 139.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E7

P_u (kip) = 51.63 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.75$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.259 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.704%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.373%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 139.10

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 6

Member No: 42

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 139.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 139.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_6$

P_u (kip) = 52.38 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.77$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.259 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.704%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.373%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 139.10

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 6

Member No: 86

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 281.30 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 135.21

Max Pu/φPn = 0.65 OK

Tension: Max Pu (kip) = 84.74 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -135.21

Max Pu/φPn = 0.14 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	281.30
Tension:	0.699 D + 1.000 Ecl	84.74

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 281.30 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 54.32 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 6

Member No: 103

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 161.33 --- Combination: $1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$

Ecl (kips) = 28.43

Max Pu/ ϕPn = 0.37 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$	161.33
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 161.33 ϕPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 58.18 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



RAM Structural System 24.00.01.18

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 6

Member No: 124

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 229.01 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 106.77

Max Pu/φPn = 0.53 OK

Tension: Max Pu (kip) = 65.35 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -106.77

Max Pu/φPn = 0.11 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	229.01
Tension:	0.699 D + 1.000 Ecl	65.35

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 229.01 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 56.00 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: 2nd Floor

Frame No: 6

Member No: 302

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 156.09

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 182.63

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 156.09

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 182.63

Vertical Resultant (kip) = 22.8

Horizontal Resultant (kip) = 174.3

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 156.09

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 182.63

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 156.09

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 182.63

Vertical Resultant (kip) = 22.8

Horizontal Resultant (kip) = 174.3

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 6

Member No: 303

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 156.09

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 182.63

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 156.09

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 182.63

Vertical Resultant (kip) = 22.8

Horizontal Resultant (kip) = 174.3

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 156.09

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 182.63

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 156.09

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 182.63

Vertical Resultant (kip) = 22.8

Horizontal Resultant (kip) = 174.3

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 6

Member No: 45

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.21$

$C_{max} = \beta\omega R_y P_{yscmax} = 182.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 156.09$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.21$

$C_{max} = \beta\omega R_y P_{yscmax} = 182.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 156.09$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E16

P_u (kip) = 77.92 ϕF_{yscAsc} (kip) = 105.30 $P_u/\phi F_{yscAsc} = 0.74$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.805 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.260%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.169%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 182.63

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 6

Member No: 46

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.21$

$C_{max} = \beta\omega R_y P_{yscmax} = 182.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 156.09$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.21$

$C_{max} = \beta\omega R_y P_{yscmax} = 182.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 156.09$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E13

P_u (kip) = 79.16 ϕF_{yscAsc} (kip) = 105.30 $P_u/\phi F_{yscAsc} = 0.75$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.812 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.263%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.169%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 182.63

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 6

Member No: 93

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.21$

$C_{max} = \beta\omega R_y P_{yscmax} = 182.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 156.09$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.21$

$C_{max} = \beta\omega R_y P_{yscmax} = 182.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 156.09$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E16

P_u (kip) = 78.24 ϕF_{yscAsc} (kip) = 105.30 $P_u/\phi F_{yscAsc} = 0.74$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.811 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.263%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.169%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 182.63

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 6

Member No: 94

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.21$

$C_{max} = \beta\omega R_y P_{yscmax} = 182.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 156.09$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.21$

$C_{max} = \beta\omega R_y P_{yscmax} = 182.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 156.09$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E13

P_u (kip) = 78.98 ϕF_{yscAsc} (kip) = 105.30 $P_u/\phi F_{yscAsc} = 0.75$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.806 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.260%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.169%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 182.63

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 7

Member No: 13

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 19.34 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.04 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	19.34
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 19.34 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 85.96 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 7

Member No: 21

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 35.63 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.07 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	35.63
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 35.63 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 81.06 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 7

Member No: 27

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 72.89 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.17 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	72.89
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 72.89 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 69.86 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 7

Member No: 28

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 98.91 --- Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$

Ecl (kips) = 0.00

Max Pu/ ϕP_n = 0.23 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$	98.91
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 98.91 ϕP_y (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 62.04 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Beam Parameters

Story: 2nd Floor

Frame No: 7

Member No: 306

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 153.72

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 170.63

Right Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 153.72

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 170.63

Vertical Resultant (kip) = 14.5

Horizontal Resultant (kip) = 166.9

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 153.72

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 170.63

Right Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 153.72

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 170.63

Vertical Resultant (kip) = 14.5

Horizontal Resultant (kip) = 166.9

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 7

Member No: 47

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 170.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 153.72$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 170.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 153.72$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E16

P_u (kip) = 74.06 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.77$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.805 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.346%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 170.63

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



RAM Structural System 24.00.01.18

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Steel Code: AISC341-16 - LRFD

Brace Parameters

Story: 2nd Floor

Frame No: 7

Member No: 48

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 170.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 153.72$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 170.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 153.72$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E13$

P_u (kip) = 74.92 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.78$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.805 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.346%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 170.63

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 8

Member No: 22

Fy (ksi): 36.00

Size: W12X65

Frame Type: No Frame Type

Criteria

Apply one and two story exceptions where applicable

Warning: This frame type is not a valid designation in the current building code.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof Frame No: 8 Member No: 172

Fy (ksi): 50.00 Size: HSS6X6X3/8

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 29.89 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.16 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	29.89
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf = 14.19 Limit = 24.02 OK

Web h/tw = 14.19 Limit = 24.02 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	356.23	198.13

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Cat 4

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 8

Member No: 29

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

A Non-BRBF Frame Type was encountered in this analysis.

Verify the correctness and applicability of the results.

Compression: Max Pu (kip) = 147.79 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.34 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	147.79

Tension: No tension on column

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 147.79 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 58.62 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 8

Member No: 46

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 230.39 --- Combination: $1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$

Ecl (kips) = 137.52

Max Pu/ ϕPn = 0.53 OK

Tension: Max Pu (kip) = 104.58 --- Combination: $0.699 D + 1.000 Ecl$

Ecl (kips) = -137.52

Max Pu/ ϕPn = 0.17 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$	230.39
Tension:	$0.699 D + 1.000 Ecl$	104.58

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 230.39 ϕPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.96 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 8

Member No: 315

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 168.2

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 168.2

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 8

Member No: 49

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

$C_{max} = \beta\omega R_y P_{yscmax} = 171.95$ kip

$T_{max} = \omega R_y P_{yscmax} = 154.91$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

$C_{max} = \beta\omega R_y P_{yscmax} = 171.95$ kip

$T_{max} = \omega R_y P_{yscmax} = 154.91$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E16

P_u (kip) = 71.76 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.74$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.801 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.344%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 171.95

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 8

Member No: 50

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 171.95$ kip

Tmax = $\omega R_y P_{yscmax} = 154.91$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 171.95$ kip

Tmax = $\omega R_y P_{yscmax} = 154.91$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E13

Pu (kip) = 72.67 ϕF_{yscAsc} (kip) = 96.52 Pu/ ϕF_{yscAsc} = 0.75

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift (2 Δ_{bm}): 2.801 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.344%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 171.95

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 9

Member No: 71

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 38.02 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/ ϕ Pn = 0.08 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	38.02
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 38.02 ϕ Py (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 80.34 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
Σ Mpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required Σ Mpc / H

Where Σ Mpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 9

Member No: 266

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 111.80

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 124.10

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 111.80

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 124.10

Vertical Resultant (kip) = 10.0

Horizontal Resultant (kip) = 138.0

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 111.80

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 124.10

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 111.80

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 124.10

Vertical Resultant (kip) = 10.0

Horizontal Resultant (kip) = 138.0

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 9

Member No: 8

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.00

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 124.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 111.80$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 124.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 111.80$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E7

P_u (kip) = 46.81 ϕF_{yscAsc} (kip) = 70.20 $P_u/\phi F_{yscAsc} = 0.67$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 70.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 23.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 93.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.375 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.769%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.373%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 124.10

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 9

Member No: 9

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.00

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 124.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 111.80$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 124.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 111.80$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_6$

P_u (kip) = 52.82 ϕF_{yscAsc} (kip) = 70.20 $P_u/\phi F_{yscAsc} = 0.75$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 70.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 23.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 93.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.374 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.769%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.373%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 124.10

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 9

Member No: 88

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 239.77 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 95.66

Max Pu/φPn = 0.56 OK

Tension: Max Pu (kip) = 45.81 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -95.66

Max Pu/φPn = 0.07 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	239.77
<i>Tension:</i>	0.699 D + 1.000 Ecl	45.81

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 239.77 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.65 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 9

Member No: 319

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 169.51

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 189.85

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 169.51

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 189.85

Vertical Resultant (kip) = 17.4

Horizontal Resultant (kip) = 184.9

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 169.51

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 189.85

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 169.51

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 189.85

Vertical Resultant (kip) = 17.4

Horizontal Resultant (kip) = 184.9

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 9

Member No: 113

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

Cmax = $\beta\omega R_y P_{yscmax} = 189.85$ kip

Tmax = $\omega R_y P_{yscmax} = 169.51$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

Cmax = $\beta\omega R_y P_{yscmax} = 189.85$ kip

Tmax = $\omega R_y P_{yscmax} = 169.51$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E16

Pu (kip) = 67.07 ϕF_{yscAsc} (kip) = 94.05 $P_u/\phi F_{yscAsc} = 0.71$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 94.05

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 31.35 \leq P_{ysc} \leq 1.2 P_{ysc} = 125.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.811 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.348%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 189.85

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 9

Member No: 114

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 189.85$ kip

$T_{max} = \omega R_y P_{yscmax} = 169.51$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 189.85$ kip

$T_{max} = \omega R_y P_{yscmax} = 169.51$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{13}$

P_u (kip) = 67.81 ϕF_{yscAsc} (kip) = 94.05 $P_u/\phi F_{yscAsc} = 0.72$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 94.05

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 31.35 \leq P_{ysc} \leq 1.2 P_{ysc} = 125.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.810 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.348%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 189.85

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 10

Member No: 53

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 90.25 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.19 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	90.25
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 90.25 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 64.64 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: Roof Frame No: 10 Member No: 899

Fy (ksi): 50.00 Size: W27X94

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 170.78

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 192.98

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 172.04

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 194.41

Vertical Resultant (kip) = 19.2

Horizontal Resultant (kip) = 213.6

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 170.78

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 192.98

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 172.04

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 194.41

Vertical Resultant (kip) = 19.2

Horizontal Resultant (kip) = 213.6

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 150.00 Lu Limit = 211.96 **OK**

Required strength of lateral brace (A-6-7) = 11.69 kip

Required stiffness of bracing (A-6-8) = 7794.56 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 6.71 Limit = 9.18 **OK**

Web h/tw = 49.43 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 10

Member No: 89

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 192.98$ kip

$T_{max} = \omega R_y P_{yscmax} = 170.78$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 192.98$ kip

$T_{max} = \omega R_y P_{yscmax} = 170.78$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E23

P_u (kip) = 71.90 ϕF_{yscAsc} (kip) = 94.05 $P_u/\phi F_{yscAsc} = 0.76$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 94.05

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 31.35 \leq P_{ysc} \leq 1.2 P_{ysc} = 125.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.348 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.847%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.542%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 192.98

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 10

Member No: 90

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.36$

$C_{max} = \beta\omega R_y P_{yscmax} = 194.41$ kip

$T_{max} = \omega R_y P_{yscmax} = 172.04$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.36$

$C_{max} = \beta\omega R_y P_{yscmax} = 194.41$ kip

$T_{max} = \omega R_y P_{yscmax} = 172.04$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E22

P_u (kip) = 61.79 ϕF_{yscAsc} (kip) = 94.05 $P_u/\phi F_{yscAsc} = 0.66$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 94.05

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 31.35 \leq P_{ysc} \leq 1.2 P_{ysc} = 125.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.310 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.823%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.542%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 194.41

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 10

Member No: 45

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 253.48 --- Combination: $1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$

Ecl (kips) = 148.02

Max Pu/ ϕPn = 0.59 OK

Tension: Max Pu (kip) = 111.12 --- Combination: $0.699 D + 1.000 Ecl$

Ecl (kips) = -148.09

Max Pu/ ϕPn = 0.18 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	$1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$	253.48
<i>Tension:</i>	$0.699 D + 1.000 Ecl$	111.12

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 253.48 ϕPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.21 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 10

Member No: 56

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 344.39 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 149.18

Max Pu/φPn = 0.80 OK

Tension: Max Pu (kip) = 73.88 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -149.11

Max Pu/φPn = 0.12 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	344.39
<i>Tension:</i>	0.699 D + 1.000 Ecl	73.88

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 344.39 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 52.29 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 10

Member No: 367

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 168.2

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 168.2

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 10

Member No: 51

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 171.95$ kip

Tmax = $\omega R_y P_{yscmax} = 154.91$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 171.95$ kip

Tmax = $\omega R_y P_{yscmax} = 154.91$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E31

Pu (kip) = 71.66 ϕF_{yscAsc} (kip) = 96.52 Pu/ ϕF_{yscAsc} = 0.74

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.796 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.341%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 171.95

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 10

Member No: 52

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 171.95$ kip

Tmax = $\omega R_y P_{yscmax} = 154.91$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 171.95$ kip

Tmax = $\omega R_y P_{yscmax} = 154.91$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E30

Pu (kip) = 72.51 ϕF_{yscAsc} (kip) = 96.52 Pu/ ϕF_{yscAsc} = 0.75

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.796 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.341%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 171.95

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Cat 4

Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 11

Member No: 19

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 35.42 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.07 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	35.42
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 35.42 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 81.13 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 11

Member No: 147

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 21.92 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.05 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	21.92
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 21.92 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 85.18 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Beam Parameters

Story: Roof

Frame No: 11

Member No: 656

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 142.98

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 160.13

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 144.05

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 161.34

Vertical Resultant (kip) = 14.9

Horizontal Resultant (kip) = 178.0

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 142.98

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 160.13

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 144.05

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 161.34

Vertical Resultant (kip) = 14.9

Horizontal Resultant (kip) = 178.0

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Brace Parameters

Story: Roof

Frame No: 11

Member No: 35

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.50

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.33$

$C_{max} = \beta\omega R_y P_{yscmax} = 160.13$ kip

$T_{max} = \omega R_y P_{yscmax} = 142.98$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.33$

$C_{max} = \beta\omega R_y P_{yscmax} = 160.13$ kip

$T_{max} = \omega R_y P_{yscmax} = 142.98$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E31

P_u (kip) = 62.18 ϕF_{yscAsc} (kip) = 87.75 $P_u/\phi F_{yscAsc} = 0.71$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 87.75

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 29.25 \leq P_{ysc} \leq 1.2 P_{ysc} = 117.00$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.267 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.776%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.503%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 160.13

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 11

Member No: 36

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.50 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 161.34$ kip

$T_{max} = \omega R_y P_{yscmax} = 144.05$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 161.34$ kip

$T_{max} = \omega R_y P_{yscmax} = 144.05$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E30

P_u (kip) = 64.60 ϕF_{yscAsc} (kip) = 87.75 $P_u/\phi F_{yscAsc} = 0.74$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 87.75

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 29.25 \leq P_{ysc} \leq 1.2 P_{ysc} = 117.00$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.267 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.776%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.503%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 161.34

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 11

Member No: 32

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 265.26 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 122.43

Max Pu/φPn = 0.62 OK

Tension: Max Pu (kip) = 73.47 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -122.48

Max Pu/φPn = 0.12 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	265.26
<i>Tension:</i>	0.699 D + 1.000 Ecl	73.47

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 265.26 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 54.83 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 11

Member No: 43

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 256.33 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 124.33

Max Pu/ ϕ Pn = 0.59 OK

Tension: Max Pu (kip) = 79.98 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -124.28

Max Pu/ ϕ Pn = 0.13 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	256.33
<i>Tension:</i>	0.699 D + 1.000 Ecl	79.98

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 256.33 ϕ Py (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.12 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 11

Member No: 1084

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 123.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 123.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Vertical Resultant (kip) = 12.7

Horizontal Resultant (kip) = 134.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 123.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 123.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Vertical Resultant (kip) = 12.7

Horizontal Resultant (kip) = 134.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Brace Parameters

Story: 2nd Floor

Frame No: 11

Member No: 109

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 123.28$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 123.28$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E31

P_u (kip) = 53.88 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.79$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.831 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.262%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.161%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 138.07

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 11

Member No: 110

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

Cmax = $\beta\omega R_y P_{yscmax} = 138.07$ kip

Tmax = $\omega R_y P_{yscmax} = 123.28$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

Cmax = $\beta\omega R_y P_{yscmax} = 138.07$ kip

Tmax = $\omega R_y P_{yscmax} = 123.28$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E30

Pu (kip) = 54.29 ϕF_{yscAsc} (kip) = 68.40 Pu/ ϕF_{yscAsc} = 0.79

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift (2 Δ_{bm}): 2.831 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.262%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.161%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 138.07

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Building Code: IBC

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 12

Member No: 44

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 30.76 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.06 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	30.76
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 30.76 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 82.53 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 12

Member No: 173

Fy (ksi): 50.00

Size: HSS6X6X5/16

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 30.59 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.19 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	30.59
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf = 17.62 Limit = 24.02 OK

Web h/tw = 17.62 Limit = 24.02 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	347.07	188.97

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 12

Member No: 63

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 131.82 --- Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$

Ecl (kips) = 0.00

Max Pu/ ϕP_n = 0.31 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$	131.82
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 131.82 ϕP_y (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 59.13 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 12

Member No: 69

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 131.65 --- Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$

Ecl (kips) = 0.00

Max Pu/ ϕP_n = 0.31 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$	131.65
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 131.65 ϕP_y (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 59.14 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 12

Member No: 1086

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 153.72

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 170.63

Vertical Resultant (kip) = 15.6

Horizontal Resultant (kip) = 167.6

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 153.72

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 170.63

Vertical Resultant (kip) = 15.6

Horizontal Resultant (kip) = 167.6

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 12

Member No: 91

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 171.95$ kip

Tmax = $\omega R_y P_{yscmax} = 154.91$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 171.95$ kip

Tmax = $\omega R_y P_{yscmax} = 154.91$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E31

Pu (kip) = 77.05 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.80$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.786 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.337%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 171.95

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 12

Member No: 92

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 170.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 153.72$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 170.63$ kip

$T_{max} = \omega R_y P_{yscmax} = 153.72$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{30}$

P_u (kip) = 77.63 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.80$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.786 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.337%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 170.63

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 13

Member No: 82

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 22.03 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.05 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	22.03
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 22.03 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 85.15 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 13

Member No: 85

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 22.03 --- Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$

Ecl (kips) = 0.00

Max Pu/ ϕP_n = 0.05 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$	22.03
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 22.03 ϕP_y (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 85.15 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣM_{pc} (kip-ft)	580.80	264.60

Column Splice Shear Force Required $\Sigma M_{pc} / H$

Where ΣM_{pc} is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: Roof Frame No: 13 Member No: 335

Fy (ksi): 36.00 Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 129.65

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 145.20

Right Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 129.65

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 145.20

Vertical Resultant (kip) = 12.6

Horizontal Resultant (kip) = 160.8

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 129.65

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 145.20

Right Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 129.65

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 145.20

Vertical Resultant (kip) = 12.6

Horizontal Resultant (kip) = 160.8

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 13

Member No: 37

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 145.20$ kip

$T_{max} = \omega R_y P_{yscmax} = 129.65$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 145.20$ kip

$T_{max} = \omega R_y P_{yscmax} = 129.65$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E31

P_u (kip) = 57.41 ϕF_{yscAsc} (kip) = 78.97 $P_u/\phi F_{yscAsc} = 0.73$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 78.97

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 26.33 \leq P_{ysc} \leq 1.2 P_{ysc} = 105.30$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.266 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.774%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.501%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 145.20

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 13

Member No: 38

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.25

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 145.20$ kip

$T_{max} = \omega R_y P_{yscmax} = 129.65$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 145.20$ kip

$T_{max} = \omega R_y P_{yscmax} = 129.65$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E30

P_u (kip) = 58.28 ϕF_{yscAsc} (kip) = 78.97 $P_u/\phi F_{yscAsc} = 0.74$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 78.97

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 26.33 \leq P_{ysc} \leq 1.2 P_{ysc} = 105.30$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.266 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.774%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.501%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 145.20

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 13

Member No: 102

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 242.08 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 111.46

Max Pu/φPn = 0.56 OK

Tension: Max Pu (kip) = 67.74 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -111.46

Max Pu/φPn = 0.11 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	242.08
Tension:	0.699 D + 1.000 Ecl	67.74

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 242.08 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.58 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 13

Member No: 129

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 242.24 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 111.46

Max Pu/φPn = 0.56 OK

Tension: Max Pu (kip) = 67.72 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -111.46

Max Pu/φPn = 0.11 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	242.24
Tension:	0.699 D + 1.000 Ecl	67.72

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 242.24 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.58 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 13

Member No: 1089

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 123.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 123.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Vertical Resultant (kip) = 12.7

Horizontal Resultant (kip) = 134.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 123.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 123.28

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Vertical Resultant (kip) = 12.7

Horizontal Resultant (kip) = 134.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 13

Member No: 111

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 123.28$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 123.28$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E31

P_u (kip) = 53.85 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.79$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.831 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.262%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.161%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 138.07

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



RAM Structural System 24.00.01.18

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 13

Member No: 112

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 123.28$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.34$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 123.28$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E30

P_u (kip) = 54.25 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.79$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.831 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.262%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.161%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 138.07

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 14

Member No: 87

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 22.91 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.05 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	22.91
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 22.91 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 84.88 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 14

Member No: 889

Fy (ksi): 50.00

Size: W27X94

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 172.04

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 196.13

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 172.04

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 196.13

Vertical Resultant (kip) = 19.5

Horizontal Resultant (kip) = 215.4

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 172.04

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 196.13

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 172.04

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 196.13

Vertical Resultant (kip) = 19.5

Horizontal Resultant (kip) = 215.4

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 150.00 Lu Limit = 211.96 **OK**

Required strength of lateral brace (A-6-7) = 11.69 kip

Required stiffness of bracing (A-6-8) = 7794.56 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 6.71 Limit = 9.18 **OK**

Web h/tw = 49.43 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 14

Member No: 59

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.75

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.14$ $\omega = 1.36$

$C_{max} = \beta\omega R_y P_{yscmax} = 196.13$ kip

$T_{max} = \omega R_y P_{yscmax} = 172.04$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.14$ $\omega = 1.36$

$C_{max} = \beta\omega R_y P_{yscmax} = 196.13$ kip

$T_{max} = \omega R_y P_{yscmax} = 172.04$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E24

P_u (kip) = 70.46 ϕF_{yscAsc} (kip) = 94.05 $P_u/\phi F_{yscAsc} = 0.75$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 94.05

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 31.35 \leq P_{ysc} \leq 1.2 P_{ysc} = 125.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.192 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.749%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.542%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 196.13

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 14

Member No: 60

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.14$ $\omega = 1.36$

$C_{max} = \beta\omega R_y P_{yscmax} = 196.13$ kip

$T_{max} = \omega R_y P_{yscmax} = 172.04$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.14$ $\omega = 1.36$

$C_{max} = \beta\omega R_y P_{yscmax} = 196.13$ kip

$T_{max} = \omega R_y P_{yscmax} = 172.04$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E21

P_u (kip) = 68.41 ϕF_{yscAsc} (kip) = 94.05 $P_u/\phi F_{yscAsc} = 0.73$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 94.05

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 31.35 \leq P_{ysc} \leq 1.2 P_{ysc} = 125.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.192 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.749%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.542%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 196.13

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 14

Member No: 127

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 280.11 --- Combination: $1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$

Ecl (kips) = 149.30

Max Pu/ ϕPn = 0.65 OK

Tension: Max Pu (kip) = 105.28 --- Combination: $0.699 D + 1.000 Ecl$

Ecl (kips) = -149.30

Max Pu/ ϕPn = 0.17 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$	280.11
Tension:	$0.699 D + 1.000 Ecl$	105.28

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 280.11 ϕPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 54.36 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Beam Parameters

Story: 2nd Floor

Frame No: 14

Member No: 1092

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 178.88

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 198.56

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 178.88

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 198.56

Vertical Resultant (kip) = 16.9

Horizontal Resultant (kip) = 194.2

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 178.88

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 198.56

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 178.88

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 198.56

Vertical Resultant (kip) = 16.9

Horizontal Resultant (kip) = 194.2

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 14

Member No: 53

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta\omega R_y P_{yscmax} = 198.56$ kip

$T_{max} = \omega R_y P_{yscmax} = 178.88$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta\omega R_y P_{yscmax} = 198.56$ kip

$T_{max} = \omega R_y P_{yscmax} = 178.88$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E31

P_u (kip) = 92.89 ϕF_{yscAsc} (kip) = 114.07 $P_u/\phi F_{yscAsc} = 0.81$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 114.07

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 38.03 \leq P_{ysc} \leq 1.2 P_{ysc} = 152.10$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.758 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.243%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.173%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 198.56

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 14

Member No: 54

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

Cmax = $\beta\omega R_y P_{yscmax} = 198.56$ kip

Tmax = $\omega R_y P_{yscmax} = 178.88$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

Cmax = $\beta\omega R_y P_{yscmax} = 198.56$ kip

Tmax = $\omega R_y P_{yscmax} = 178.88$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E30

Pu (kip) = 93.33 ϕF_{yscAsc} (kip) = 114.07 Pu/ ϕF_{yscAsc} = 0.82

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 114.07

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 38.03 \leq P_{ysc} \leq 1.2 P_{ysc} = 152.10$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift (2 Δ_{bm}): 2.758 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 1.243%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.173%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 198.56

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 15

Member No: 16

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 20.57 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.04 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	20.57
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 20.57 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 85.59 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 15

Member No: 41

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 20.77 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.04 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	20.77
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 20.77 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 85.53 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: Roof Frame No: 15 Member No: 478

Fy (ksi): 36.00 Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 186.15

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 168.99

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 187.58

Vertical Resultant (kip) = 16.1

Horizontal Resultant (kip) = 207.8

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 186.15

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 168.99

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 187.58

Vertical Resultant (kip) = 16.1

Horizontal Resultant (kip) = 207.8

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 15

Member No: 10

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 186.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 167.70$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 186.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 167.70$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E31

P_u (kip) = 64.49 ϕF_{yscAsc} (kip) = 105.30 $P_u/\phi F_{yscAsc} = 0.61$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.018 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.574%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.386%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 186.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 15

Member No: 11

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 187.58$ kip

Tmax = $\omega R_y P_{yscmax} = 168.99$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

Cmax = $\beta\omega R_y P_{yscmax} = 187.58$ kip

Tmax = $\omega R_y P_{yscmax} = 168.99$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E30

Pu (kip) = 65.79 ϕF_{yscAsc} (kip) = 105.30 Pu/ ϕF_{yscAsc} = 0.62

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.018 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 0.574%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.386%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 187.58

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 15

Member No: 36

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 253.76 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 142.91

Max Pu/φPn = 0.59 OK

Tension: Max Pu (kip) = 103.95 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -142.97

Max Pu/φPn = 0.17 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	253.76
Tension:	0.699 D + 1.000 Ecl	103.95

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 253.76 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.20 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 15

Member No: 39

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 252.93 --- Combination: $1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$

Ecl (kips) = 145.18

Max Pu/ ϕPn = 0.59 OK

Tension: Max Pu (kip) = 107.20 --- Combination: $0.699 D + 1.000 Ecl$

Ecl (kips) = -145.12

Max Pu/ ϕPn = 0.17 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	$1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$	252.93
<i>Tension:</i>	$0.699 D + 1.000 Ecl$	107.20

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 252.93 ϕPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.23 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 15

Member No: 1100

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 233.92

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 259.65

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 233.92

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 259.65

Vertical Resultant (kip) = 22.1

Horizontal Resultant (kip) = 254.0

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 233.92

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 259.65

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 233.92

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 259.65

Vertical Resultant (kip) = 22.1

Horizontal Resultant (kip) = 254.0

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 15

Member No: 55

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_4.25

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta \omega R_y P_{yscmax} = 259.65$ kip

$T_{max} = \omega R_y P_{yscmax} = 233.92$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta \omega R_y P_{yscmax} = 259.65$ kip

$T_{max} = \omega R_y P_{yscmax} = 233.92$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E31

P_u (kip) = 131.59 ϕF_{yscAsc} (kip) = 149.17 $P_u / \phi F_{yscAsc} = 0.88$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 149.17

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 49.73 \leq P_{ysc} \leq 1.2 P_{ysc} = 198.90$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.703 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.247%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.201%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 259.65

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 15

Member No: 56

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_4.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta\omega R_y P_{yscmax} = 259.65$ kip

$T_{max} = \omega R_y P_{yscmax} = 233.92$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta\omega R_y P_{yscmax} = 259.65$ kip

$T_{max} = \omega R_y P_{yscmax} = 233.92$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E30

P_u (kip) = 132.00 ϕF_{yscAsc} (kip) = 149.17 $P_u/\phi F_{yscAsc} = 0.88$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 149.17

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 49.73 \leq P_{ysc} \leq 1.2 P_{ysc} = 198.90$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.703 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.247%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.201%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 259.65

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 16

Member No: 63

Fy (ksi): 36.00

Size: W12X65

Frame Type: No Frame Type

Criteria

Apply one and two story exceptions where applicable

Warning: This frame type is not a valid designation in the current building code.



Seismic Provisions Member Code Check

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof Frame No: 16 Member No: 83
 Fy (ksi): 36.00 Size: W12X65
 Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 14.57 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.03 OK

Tension: Max Pu (kip) = 4.26 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -10.74

Max Pu/φPn = 0.01 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	14.57
<i>Tension:</i>	0.699 D + 1.000 Ecl	4.26

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 14.57 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 87.39 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 16

Member No: 84

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 14.57 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/ ϕ Pn = 0.03 OK

Tension: Max Pu (kip) = 4.26 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -10.74

Max Pu/ ϕ Pn = 0.01 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	14.57
Tension:	0.699 D + 1.000 Ecl	4.26

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 14.57 ϕ Py (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 87.39 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
Σ Mpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required Σ Mpc / H

Where Σ Mpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Beam Parameters

Story: Roof Frame No: 16 Member No: 473

Fy (ksi): 36.00 Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 76.32
Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 102.28
Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 75.79
Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 101.56
Vertical Resultant (kip) = 21.5
Horizontal Resultant (kip) = 104.2

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 76.32
Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 102.28
Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 75.79
Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 101.56
Vertical Resultant (kip) = 21.5
Horizontal Resultant (kip) = 104.2
Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 16

Member No: 12

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.34$ $\omega = 1.42$

$C_{max} = \beta\omega R_y P_{yscmax} = 102.28$ kip

$T_{max} = \omega R_y P_{yscmax} = 76.32$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.34$ $\omega = 1.42$

$C_{max} = \beta\omega R_y P_{yscmax} = 102.28$ kip

$T_{max} = \omega R_y P_{yscmax} = 76.32$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.200 Sp + 1.000 E31$

P_u (kip) = 38.52 ϕF_{yscAsc} (kip) = 43.88 $P_u / \phi F_{yscAsc} = 0.88$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 43.88

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 14.63 \leq P_{ysc} \leq 1.2 P_{ysc} = 58.50$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.146 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.701%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.501%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 102.28

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 16

Member No: 13

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.25 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.34$ $\omega = 1.41$

$C_{max} = \beta\omega R_y P_{yscmax} = 101.56$ kip

$T_{max} = \omega R_y P_{yscmax} = 75.79$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.34$ $\omega = 1.41$

$C_{max} = \beta\omega R_y P_{yscmax} = 101.56$ kip

$T_{max} = \omega R_y P_{yscmax} = 75.79$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{30}$

P_u (kip) = 38.88 ϕF_{yscAsc} (kip) = 43.88 $P_u / \phi F_{yscAsc} = 0.89$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 43.88

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 14.63 \leq P_{ysc} \leq 1.2 P_{ysc} = 58.50$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.146 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.701%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.501%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 101.56

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 16

Member No: 97

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

A Non-BRBF Frame Type was encountered in this analysis.

Verify the correctness and applicability of the results.

Compression: Max Pu (kip) = 72.61 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.17 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	72.61
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 72.61 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 69.95 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 16

Member No: 98

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 145.16 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 72.72

Max Pu/φPn = 0.34 OK

Tension: Max Pu (kip) = 47.57 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -72.64

Max Pu/φPn = 0.08 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	145.16
Tension:	0.699 D + 1.000 Ecl	47.57

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 145.16 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 58.70 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 16

Member No: 125

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 146.71 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 71.62

Max Pu/φPn = 0.34 OK

Tension: Max Pu (kip) = 45.73 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -71.70

Max Pu/φPn = 0.07 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	146.71
Tension:	0.699 D + 1.000 Ecl	45.73

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 146.71 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 58.65 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: 2nd Floor

Frame No: 16

Member No: 1095

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 OK

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

$Cd = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($Cd=2$)

Design Check Error

Only one of two braces framing into beam has a BRB size.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/t_f = 7.06 Limit = 9.18 OK

Web h/t_w = 53.49 Limit = 36.05 NG

Note: Beam axial load from F4.3 Analysis not determined; Web h/t_w limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 16

Member No: 1096

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 233.92

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 259.65

Right Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 233.92

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 259.65

Vertical Resultant (kip) = 22.1

Horizontal Resultant (kip) = 254.0

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 233.92

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 259.65

Right Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 233.92

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 259.65

Vertical Resultant (kip) = 22.1

Horizontal Resultant (kip) = 254.0

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 16

Member No: 57

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_4.25

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta\omega R_y P_{yscmax} = 259.65$ kip

$T_{max} = \omega R_y P_{yscmax} = 233.92$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta\omega R_y P_{yscmax} = 259.65$ kip

$T_{max} = \omega R_y P_{yscmax} = 233.92$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E31

P_u (kip) = 125.90 ϕF_{yscAsc} (kip) = 149.17 $P_u/\phi F_{yscAsc} = 0.84$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 149.17

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 49.73 \leq P_{ysc} \leq 1.2 P_{ysc} = 198.90$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.704 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.248%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.201%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 259.65

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 16

Member No: 58

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_4.25

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta\omega R_y P_{yscmax} = 259.65$ kip

$T_{max} = \omega R_y P_{yscmax} = 233.92$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta\omega R_y P_{yscmax} = 259.65$ kip

$T_{max} = \omega R_y P_{yscmax} = 233.92$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E30

P_u (kip) = 126.13 ϕF_{yscAsc} (kip) = 149.17 $P_u/\phi F_{yscAsc} = 0.85$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 149.17

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 49.73 \leq P_{ysc} \leq 1.2 P_{ysc} = 198.90$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.702 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.247%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.201%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 259.65

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 17

Member No: 42

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 12.56 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.03 OK

Tension: Max Pu (kip) = 1.88 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -7.51

Max Pu/φPn = 0.00 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	12.56
Tension:	0.699 D + 1.000 Ecl	1.88

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 12.56 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 87.99 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 17

Member No: 43

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 12.51 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.03 OK

Tension: Max Pu (kip) = 1.90 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -7.51

Max Pu/φPn = 0.00 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	12.51
Tension:	0.699 D + 1.000 Ecl	1.90

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 12.51 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 88.01 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: Roof

Frame No: 17

Member No: 480

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.30

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.30

Vertical Resultant (kip) = 15.0

Horizontal Resultant (kip) = 80.8

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.30

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.30

Vertical Resultant (kip) = 15.0

Horizontal Resultant (kip) = 80.8

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 17

Member No: 16

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.31$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.30$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.31$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.30$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{31}$

P_u (kip) = 28.06 ϕF_{yscAsc} (kip) = 35.10 $P_u / \phi F_{yscAsc} = 0.80$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 35.10

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 11.70 \leq P_{ysc} \leq 1.2 P_{ysc} = 46.80$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.038 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 0.638%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.507%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 78.30

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 17

Member No: 17

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.31$ $\omega = 1.39$

Cmax = $\beta\omega R_y P_{yscmax} = 78.30$ kip

Tmax = $\omega R_y P_{yscmax} = 59.77$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.31$ $\omega = 1.39$

Cmax = $\beta\omega R_y P_{yscmax} = 78.30$ kip

Tmax = $\omega R_y P_{yscmax} = 59.77$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E30

Pu (kip) = 27.92 ϕF_{yscAsc} (kip) = 35.10 $P_u / \phi F_{yscAsc} = 0.80$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 35.10

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 11.70 \leq P_{ysc} \leq 1.2 P_{ysc} = 46.80$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.038 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2x Design Story Drift: 0.638%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E37 Fpx_min X EQ_User

Required core strain at 2% Design Story Drift: 1.507%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 78.30

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 17

Member No: 38

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 102.59 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 55.99

Max Pu/φPn = 0.24 OK

Tension: Max Pu (kip) = 37.76 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -55.99

Max Pu/φPn = 0.06 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	102.59
<i>Tension:</i>	0.699 D + 1.000 Ecl	37.76

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 102.59 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 60.94 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 17

Member No: 58

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 112.37 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 55.99

Max Pu/φPn = 0.26 OK

Tension: Max Pu (kip) = 34.55 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -55.99

Max Pu/φPn = 0.06 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	112.37
<i>Tension:</i>	0.699 D + 1.000 Ecl	34.55

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 112.37 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 59.76 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 17

Member No: 1104

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 206.40

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 229.10

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 206.40

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 229.10

Vertical Resultant (kip) = 19.5

Horizontal Resultant (kip) = 224.1

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 206.40

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 229.10

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 206.40

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 229.10

Vertical Resultant (kip) = 19.5

Horizontal Resultant (kip) = 224.1

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 17

Member No: 59

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta\omega R_y P_{yscmax} = 229.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 206.40$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta\omega R_y P_{yscmax} = 229.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 206.40$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E31

P_u (kip) = 129.62 ϕF_{yscAsc} (kip) = 131.63 $P_u/\phi F_{yscAsc} = 0.98$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 131.63

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 43.88 \leq P_{ysc} \leq 1.2 P_{ysc} = 175.50$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.722 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.235%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.181%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 229.10

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 17

Member No: 60

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta \omega R_y P_{yscmax} = 229.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 206.40$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.28$

$C_{max} = \beta \omega R_y P_{yscmax} = 229.10$ kip

$T_{max} = \omega R_y P_{yscmax} = 206.40$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{30}$

P_u (kip) = 129.91 ϕF_{yscAsc} (kip) = 131.63 $P_u / \phi F_{yscAsc} = 0.99$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 131.63

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 43.88 \leq P_{ysc} \leq 1.2 P_{ysc} = 175.50$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.722 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2x Design Story Drift: 1.235%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E37 F_{px_min} X EQ_User

Required core strain at 2% Design Story Drift: 1.181%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 229.10

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: Roof

Frame No: 18

Member No: 33

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 7.74 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.02 OK

Tension: Max Pu (kip) = 2.96 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -6.55

Max Pu/φPn = 0.00 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	7.74
Tension:	0.699 D + 1.000 Ecl	2.96

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 7.74 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 89.44 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 18

Member No: 35

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 22.18 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.05 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	22.18
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 22.18 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 85.10 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 18

Member No: 14

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Vertical Resultant (kip) = 13.1

Horizontal Resultant (kip) = 154.8

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Vertical Resultant (kip) = 13.1

Horizontal Resultant (kip) = 154.8

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 18

Member No: 26

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 140.35$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 140.35$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E20

P_u (kip) = 48.87 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.71$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.361 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 0.761%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.373%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 140.35

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 18

Member No: 27

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

Cmax = $\beta\omega R_y P_{yscmax} = 140.35$ kip

Tmax = $\omega R_y P_{yscmax} = 124.20$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

Cmax = $\beta\omega R_y P_{yscmax} = 140.35$ kip

Tmax = $\omega R_y P_{yscmax} = 124.20$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E17

Pu (kip) = 46.03 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.67$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.361 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 0.761%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.455 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.373%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 140.35

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 18

Member No: 7

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 235.31 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 107.28

Max Pu/φPn = 0.55 OK

Tension: Max Pu (kip) = 59.75 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -107.28

Max Pu/φPn = 0.10 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	235.31
<i>Tension:</i>	0.699 D + 1.000 Ecl	59.75

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 235.31 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.80 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 18

Member No: 8

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 168.53 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 107.28

Max Pu/φPn = 0.39 OK

Tension: Max Pu (kip) = 84.56 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -107.28

Max Pu/φPn = 0.14 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	168.53
Tension:	0.699 D + 1.000 Ecl	84.56

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 168.53 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 57.95 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 18

Member No: 33

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 168.2

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 154.91

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 171.95

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 168.2

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 99.96 Lu Limit = 122.10 **OK**

Required strength of lateral brace (A-6-7) = 4.23 kip

Required stiffness of bracing (A-6-8) = 2822.96 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.06 Limit = 9.18 **OK**

Web h/tw = 53.49 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 18

Member No: 73

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

$C_{max} = \beta\omega R_y P_{yscmax} = 171.95$ kip

$T_{max} = \omega R_y P_{yscmax} = 154.91$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

$C_{max} = \beta\omega R_y P_{yscmax} = 171.95$ kip

$T_{max} = \omega R_y P_{yscmax} = 154.91$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E34

P_u (kip) = 69.49 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.72$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.275 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.092%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 171.95

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 18

Member No: 74

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

$C_{max} = \beta\omega R_y P_{yscmax} = 171.95$ kip

$T_{max} = \omega R_y P_{yscmax} = 154.91$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.31$

$C_{max} = \beta\omega R_y P_{yscmax} = 171.95$ kip

$T_{max} = \omega R_y P_{yscmax} = 154.91$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E35

P_u (kip) = 69.35 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.72$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.353 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.129%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.249%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 171.95

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 19

Member No: 109

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 188.63 --- Combination: $1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$

Ecl (kips) = 0.00

Max Pu/ ϕPn = 0.44 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl$	188.63
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 188.63 ϕPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 57.30 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 19

Member No: 110

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 154.29 --- Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$

Ecl (kips) = 0.00

Max Pu/ ϕP_n = 0.36 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$	154.29
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 154.29 ϕP_y (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 58.41 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 19

Member No: 57

Fy (ksi): 50.00

Size: W14X30

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 141.90

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 158.93

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 141.90

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 158.93

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 154.8

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 141.90

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 158.93

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 141.90

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 158.93

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 154.8

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 75.00 Lu Limit = 149.09 **OK**

Required strength of lateral brace (A-6-7) = 3.88 kip

Required stiffness of bracing (A-6-8) = 2585.66 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 8.74 Limit = 9.18 **OK**

Web h/tw = 45.30 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 19

Member No: 75

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.50 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.32$

$C_{max} = \beta\omega R_y P_{yscmax} = 158.93$ kip

$T_{max} = \omega R_y P_{yscmax} = 141.90$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.32$

$C_{max} = \beta\omega R_y P_{yscmax} = 158.93$ kip

$T_{max} = \omega R_y P_{yscmax} = 141.90$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E28

P_u (kip) = 68.68 ϕF_{yscAsc} (kip) = 87.75 $P_u/\phi F_{yscAsc} = 0.78$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 87.75

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 29.25 \leq P_{ysc} \leq 1.2 P_{ysc} = 117.00$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.265 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.137%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.307%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 158.93

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 19

Member No: 76

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.50 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.32$

$C_{max} = \beta\omega R_y P_{yscmax} = 158.93$ kip

$T_{max} = \omega R_y P_{yscmax} = 141.90$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.32$

$C_{max} = \beta\omega R_y P_{yscmax} = 158.93$ kip

$T_{max} = \omega R_y P_{yscmax} = 141.90$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E25

P_u (kip) = 68.63 ϕF_{yscAsc} (kip) = 87.75 $P_u/\phi F_{yscAsc} = 0.78$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 87.75

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 29.25 \leq P_{ysc} \leq 1.2 P_{ysc} = 117.00$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.265 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.137%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.604 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.307%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 158.93

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 20

Member No: 77

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 33.02 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.07 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	33.02
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 33.02 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 81.85 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 20

Member No: 24

Fy (ksi): 36.00

Size: W24X68

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

F4.3 Non-V and Non-Inverted V-Braced Frame Unbalanced Forces

Beam must be designed for unbalanced brace axial horizontal force component on beam ends.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange $b/t_f = 7.67$ Limit = 9.27 OK

Web $h/t_w = 51.86$ Limit = 36.38 NG

Note: Beam axial load from F4.3 Analysis not determined; Web h/t_w limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 20

Member No: 40

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_4.50 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

Cmax = $\beta\omega R_y P_{yscmax} = 279.22$ kip

Tmax = $\omega R_y P_{yscmax} = 251.55$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

Cmax = $\beta\omega R_y P_{yscmax} = 279.22$ kip

Tmax = $\omega R_y P_{yscmax} = 251.55$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E28

Pu (kip) = 86.96 ϕF_{yscAsc} (kip) = 157.95 Pu/ ϕF_{yscAsc} = 0.55

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 157.95

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 52.65 \leq P_{ysc} \leq 1.2 P_{ysc} = 210.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.713 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2x Design Story Drift: 0.676%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.380 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2% Design Story Drift: 1.334%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 279.22

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 20

Member No: 111

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 357.37 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 163.46

Max Pu/φPn = 0.83 OK

Tension: Max Pu (kip) = 75.97 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -147.26

Max Pu/φPn = 0.12 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	357.37
Tension:	0.699 D + 1.000 Ecl	75.97

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 357.37 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 51.87 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: 2nd Floor

Frame No: 20

Member No: 59

Fy (ksi): 50.00

Size: W18X35

Frame Type: Buckling Restrained Braced Frame

Criteria

Apply one and two story exceptions where applicable

F4.3 Non-V and Non-Inverted V-Braced Frame Unbalanced Forces

Beam must be designed for unbalanced brace axial horizontal force component on beam ends.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange $b/t_f = 7.06$ Limit = 9.18 OK

Web $h/t_w = 53.49$ Limit = 36.05 NG

Note: Beam axial load from F4.3 Analysis not determined; Web h/t_w limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 21

Member No: 31

Fy (ksi): 36.00

Size: W12X65

Frame Type: No Frame Type

Criteria

Apply one and two story exceptions where applicable

Warning: This frame type is not a valid designation in the current building code.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 21

Member No: 37

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 33.11 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.07 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	33.11
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 33.11 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 81.82 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 21

Member No: 11

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

A Non-BRBF Frame Type was encountered in this analysis.

Verify the correctness and applicability of the results.

Compression: Max Pu (kip) = 76.43 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.18 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	76.43
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 76.43 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 68.80 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 21

Member No: 12

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 87.52 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.20 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	87.52
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 87.52 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 65.47 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 21

Member No: 37

Fy (ksi): 50.00

Size: W24X55

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 157.27

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 176.15

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 157.27

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 176.15

Vertical Resultant (kip) = 15.0

Horizontal Resultant (kip) = 203.1

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 157.27

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 176.15

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 157.27

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 176.15

Vertical Resultant (kip) = 15.0

Horizontal Resultant (kip) = 203.1

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 96.00 Lu Limit = 134.27 **OK**

Required strength of lateral brace (A-6-7) = 6.38 kip

Required stiffness of bracing (A-6-8) = 4254.89 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 6.94 Limit = 9.18 **OK**

Web h/tw = 54.63 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 21

Member No: 71

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.33$

$C_{max} = \beta\omega R_y P_{yscmax} = 176.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 157.27$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.33$

$C_{max} = \beta\omega R_y P_{yscmax} = 176.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 157.27$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E34

P_u (kip) = 90.74 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.94$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.734 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.197%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.343%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 176.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 21

Member No: 72

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_2.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.33$

$C_{max} = \beta\omega R_y P_{yscmax} = 176.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 157.27$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.33$

$C_{max} = \beta\omega R_y P_{yscmax} = 176.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 157.27$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E35

P_u (kip) = 90.96 ϕF_{yscAsc} (kip) = 96.52 $P_u/\phi F_{yscAsc} = 0.94$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 96.52

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 32.18 \leq P_{ysc} \leq 1.2 P_{ysc} = 128.70$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.734 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.197%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.343%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 176.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 22

Member No: 99

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 18.79 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.04 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	18.79
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 18.79 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 86.12 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 22

Member No: 101

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 17.38 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.04 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	17.38
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 17.38 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 86.55 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 22

Member No: 207

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.90

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.90

Vertical Resultant (kip) = 14.1

Horizontal Resultant (kip) = 94.1

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.90

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.90

Vertical Resultant (kip) = 14.1

Horizontal Resultant (kip) = 94.1

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 96.00 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 22

Member No: 31

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.90$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.90$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{10}$

P_u (kip) = 30.05 ϕF_{yscAsc} (kip) = 35.10 $P_u / \phi F_{yscAsc} = 0.86$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 35.10

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 11.70 \leq P_{ysc} \leq 1.2 P_{ysc} = 46.80$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.517 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 0.834%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.560%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 78.90

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Brace Parameters

Story: Roof

Frame No: 22

Member No: 32

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.90$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.90$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E11

P_u (kip) = 30.22 ϕF_{yscAsc} (kip) = 35.10 $P_u / \phi F_{yscAsc} = 0.86$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 35.10

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 11.70 \leq P_{ysc} \leq 1.2 P_{ysc} = 46.80$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.517 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 0.834%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.560%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 78.90

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 22

Member No: 135

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 121.31 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 50.94

Max Pu/φPn = 0.28 OK

Tension: Max Pu (kip) = 24.95 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -50.94

Max Pu/φPn = 0.04 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	121.31
Tension:	0.699 D + 1.000 Ecl	24.95

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 121.31 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 59.47 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 22

Member No: 136

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 107.12 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 50.94

Max Pu/φPn = 0.25 OK

Tension: Max Pu (kip) = 29.23 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -50.94

Max Pu/φPn = 0.05 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	107.12
Tension:	0.699 D + 1.000 Ecl	29.23

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 107.12 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 59.93 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 22

Member No: 72

Fy (ksi): 50.00

Size: W24X55

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 186.15

Right Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 186.15

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 215.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 186.15

Right Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 186.15

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 215.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 134.27 **OK**

Required strength of lateral brace (A-6-7) = 6.38 kip

Required stiffness of bracing (A-6-8) = 4254.89 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 6.94 Limit = 9.18 **OK**

Web h/tw = 54.63 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 22

Member No: 83

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

Cmax = $\beta\omega R_y P_{yscmax} = 186.15$ kip

Tmax = $\omega R_y P_{yscmax} = 167.70$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

Cmax = $\beta\omega R_y P_{yscmax} = 186.15$ kip

Tmax = $\omega R_y P_{yscmax} = 167.70$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E28

Pu (kip) = 89.87 ϕF_{yscAsc} (kip) = 105.30 $P_u / \phi F_{yscAsc} = 0.85$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.656 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.094%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.265%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 186.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 22

Member No: 84

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 186.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 167.70$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 186.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 167.70$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E25

P_u (kip) = 89.66 ϕF_{yscAsc} (kip) = 105.30 $P_u / \phi F_{yscAsc} = 0.85$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.656 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.094%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.265%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 186.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 23

Member No: 80

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 18.81 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.04 OK

Tension: Max Pu (kip) = 9.99 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -18.23

Max Pu/φPn = 0.02 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	18.81
Tension:	0.699 D + 1.000 Ecl	9.99

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 18.81 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 86.12 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 23

Member No: 81

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 29.26 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.06 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	29.26
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 29.26 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 82.98 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 23

Member No: 176

Fy (ksi): 46.00

Size: HSS6X6X3/8

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 28.98 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.16 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	28.98
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf = 14.19 Limit = 25.04 OK

Web h/tw = 14.19 Limit = 25.04 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	350.97	192.87

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: Roof Frame No: 23 Member No: 263

Fy (ksi): 36.00 Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 104.60

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 104.60

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Vertical Resultant (kip) = 24.6

Horizontal Resultant (kip) = 164.6

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 104.60

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 104.60

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 138.07

Vertical Resultant (kip) = 24.6

Horizontal Resultant (kip) = 164.6

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof Frame No: 23 Member No: 274

Fy (ksi): 36.00 Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Vertical Resultant (kip) = 11.9

Horizontal Resultant (kip) = 179.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Vertical Resultant (kip) = 11.9

Horizontal Resultant (kip) = 179.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 23

Member No: 29

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 104.60$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 104.60$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.200 Sp + 1.000 E25$

P_u (kip) = 46.60 ϕF_{yscAsc} (kip) = 61.42 $P_u / \phi F_{yscAsc} = 0.76$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 61.42

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 20.48 \leq P_{ysc} \leq 1.2 P_{ysc} = 81.90$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.617 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 0.892%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.564%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 138.07

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 23

Member No: 30

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 104.60$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 104.60$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{28}$

P_u (kip) = 47.21 ϕF_{yscAsc} (kip) = 61.42 $P_u / \phi F_{yscAsc} = 0.77$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 61.42

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 20.48 \leq P_{ysc} \leq 1.2 P_{ysc} = 81.90$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.617 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 0.892%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.564%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 138.07

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 23

Member No: 63

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 140.35$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 140.35$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E10

P_u (kip) = 48.57 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.71$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.598 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 0.817%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.450%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 140.35

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



RAM Structural System 24.00.01.18

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 23

Member No: 64

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 140.35$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 140.35$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E11

P_u (kip) = 51.48 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.75$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.598 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 0.817%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.450%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 140.35

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 23

Member No: 104

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 192.98 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 89.15

Max Pu/φPn = 0.45 OK

Tension: Max Pu (kip) = 53.25 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -89.15

Max Pu/φPn = 0.09 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	192.98
Tension:	0.699 D + 1.000 Ecl	53.25

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 192.98 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 57.16 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 23

Member No: 105

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 172.34 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 95.66

Max Pu/φPn = 0.40 OK

Tension: Max Pu (kip) = 67.80 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -95.66

Max Pu/φPn = 0.11 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	172.34
<i>Tension:</i>	0.699 D + 1.000 Ecl	67.80

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 172.34 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 57.83 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 23

Member No: 106

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 199.86 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 97.19

Max Pu/φPn = 0.46 OK

Tension: Max Pu (kip) = 61.74 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -97.19

Max Pu/φPn = 0.10 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	199.86
Tension:	0.699 D + 1.000 Ecl	61.74

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 199.86 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 56.94 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 23

Member No: 357

Fy (ksi): 50.00

Size: W27X84

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Vertical Resultant (kip) = 50.0

Horizontal Resultant (kip) = 465.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Vertical Resultant (kip) = 50.0

Horizontal Resultant (kip) = 465.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 207.12 **OK**

Required strength of lateral brace (A-6-7) = 10.30 kip

Required stiffness of bracing (A-6-8) = 6866.21 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.81 Limit = 9.18 **OK**

Web h/tw = 52.65 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 23

Member No: 358

Fy (ksi): 50.00

Size: W27X84

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 413.61

Right Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 413.61

Vertical Resultant (kip) = 50.0

Horizontal Resultant (kip) = 465.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 413.61

Right Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 413.61

Vertical Resultant (kip) = 50.0

Horizontal Resultant (kip) = 465.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 207.12 **OK**

Required strength of lateral brace (A-6-7) = 10.30 kip

Required stiffness of bracing (A-6-8) = 6866.21 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.81 Limit = 9.18 **OK**

Web h/tw = 52.65 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 23

Member No: 77

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_6.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

Cmax = $\beta\omega R_y P_{yscmax} = 413.61$ kip

Tmax = $\omega R_y P_{yscmax} = 350.52$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

Cmax = $\beta\omega R_y P_{yscmax} = 413.61$ kip

Tmax = $\omega R_y P_{yscmax} = 350.52$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E25

Pu (kip) = 177.09 ϕF_{yscAsc} (kip) = 205.20 Pu/ ϕF_{yscAsc} = 0.86

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 205.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 68.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 273.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.626 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2x Design Story Drift: 1.241%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 413.61

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 23

Member No: 78

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_6.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

$C_{max} = \beta \omega R_y P_{yscmax} = 413.61$ kip

$T_{max} = \omega R_y P_{yscmax} = 350.52$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

$C_{max} = \beta \omega R_y P_{yscmax} = 413.61$ kip

$T_{max} = \omega R_y P_{yscmax} = 350.52$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{10}$

P_u (kip) = 177.77 ϕF_{yscAsc} (kip) = 205.20 $P_u / \phi F_{yscAsc} = 0.87$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 205.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 68.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 273.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.634 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 1.245%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 413.61

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 23

Member No: 97

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_6.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

$C_{max} = \beta\omega R_y P_{yscmax} = 413.61$ kip

$T_{max} = \omega R_y P_{yscmax} = 350.52$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

$C_{max} = \beta\omega R_y P_{yscmax} = 413.61$ kip

$T_{max} = \omega R_y P_{yscmax} = 350.52$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E10

P_u (kip) = 177.20 ϕF_{yscAsc} (kip) = 205.20 $P_u/\phi F_{yscAsc} = 0.86$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 205.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 68.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 273.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.626 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.241%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 413.61

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 23

Member No: 98

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_6.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

Cmax = $\beta\omega R_y P_{yscmax} = 413.61$ kip

Tmax = $\omega R_y P_{yscmax} = 350.52$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

Cmax = $\beta\omega R_y P_{yscmax} = 413.61$ kip

Tmax = $\omega R_y P_{yscmax} = 350.52$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E25

Pu (kip) = 177.67 ϕF_{yscAsc} (kip) = 205.20 Pu/ ϕF_{yscAsc} = 0.87

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 205.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 68.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 273.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.634 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2x Design Story Drift: 1.245%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 413.61

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 24

Member No: 10

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 33.27 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.07 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	33.27
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 33.27 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 81.77 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Roof

Frame No: 24

Member No: 11

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 34.63 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.07 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	34.63
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 34.63 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 81.36 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 24

Member No: 556

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 60.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 79.46

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.30

Vertical Resultant (kip) = 14.5

Horizontal Resultant (kip) = 94.4

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 60.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 79.46

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.30

Vertical Resultant (kip) = 14.5

Horizontal Resultant (kip) = 94.4

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 24

Member No: 20

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.40$

$C_{max} = \beta\omega R_y P_{yscmax} = 79.46$ kip

$T_{max} = \omega R_y P_{yscmax} = 60.20$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.40$

$C_{max} = \beta\omega R_y P_{yscmax} = 79.46$ kip

$T_{max} = \omega R_y P_{yscmax} = 60.20$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.200 Sp + 1.000 E20$

P_u (kip) = 33.38 ϕF_{yscAsc} (kip) = 35.10 $P_u / \phi F_{yscAsc} = 0.95$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 35.10

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 11.70 \leq P_{ysc} \leq 1.2 P_{ysc} = 46.80$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.640 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 0.902%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.560%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 79.46

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 24

Member No: 21

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.31$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.30$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.31$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.30$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E17$

P_u (kip) = 33.50 ϕF_{yscAsc} (kip) = 35.10 $P_u / \phi F_{yscAsc} = 0.95$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 35.10

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 11.70 \leq P_{ysc} \leq 1.2 P_{ysc} = 46.80$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.687 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 0.928%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.560%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 78.30

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 24

Member No: 1

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 127.66 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 50.88

Max Pu/φPn = 0.30 OK

Tension: Max Pu (kip) = 22.35 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -51.15

Max Pu/φPn = 0.04 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	127.66
Tension:	0.699 D + 1.000 Ecl	22.35

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 127.66 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 59.27 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 24

Member No: 2

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 126.36 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 51.15

Max Pu/φPn = 0.29 OK

Tension: Max Pu (kip) = 22.78 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -50.88

Max Pu/φPn = 0.04 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	126.36
Tension:	0.699 D + 1.000 Ecl	22.78

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 126.36 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 59.31 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 24

Member No: 326

Fy (ksi): 50.00

Size: W24X55

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 186.15

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 186.15

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 215.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 186.15

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 186.15

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 215.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 134.27 **OK**

Required strength of lateral brace (A-6-7) = 6.38 kip

Required stiffness of bracing (A-6-8) = 4254.89 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 6.94 Limit = 9.18 **OK**

Web h/tw = 54.63 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 24

Member No: 65

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 186.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 167.70$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 186.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 167.70$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E34

P_u (kip) = 95.18 ϕF_{yscAsc} (kip) = 105.30 $P_u/\phi F_{yscAsc} = 0.90$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.763 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.138%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.265%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 186.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 24

Member No: 66

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 186.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 167.70$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 186.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 167.70$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E35

P_u (kip) = 95.61 ϕF_{yscAsc} (kip) = 105.30 $P_u/\phi F_{yscAsc} = 0.91$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.763 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.138%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.265%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 186.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Column Parameters

Story: Roof

Frame No: 25

Member No: 177

Fy (ksi): 50.00

Size: HSS6X6X3/8

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 20.35 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.11 OK

Tension: Max Pu (kip) = 1.93 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -10.77

Max Pu/φPn = 0.01 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	20.35
<i>Tension:</i>	0.699 D + 1.000 Ecl	1.93

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf = 14.19 Limit = 24.02 OK

Web h/tw = 14.19 Limit = 24.02 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	356.23	198.13

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Beam Parameters

Story: Roof

Frame No: 25

Member No: 897

Fy (ksi): 50.00

Size: W21X50

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 172.50

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 201.83

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 172.50

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 201.83

Vertical Resultant (kip) = 21.5

Horizontal Resultant (kip) = 253.9

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 172.50

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 201.83

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 172.50

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 201.83

Vertical Resultant (kip) = 21.5

Horizontal Resultant (kip) = 253.9

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 130.39 **OK**

Required strength of lateral brace (A-6-7) = 5.97 kip

Required stiffness of bracing (A-6-8) = 3980.59 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 6.10 Limit = 9.18 **OK**

Web h/tw = 49.26 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 25

Member No: 77

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.25$

$C_{max} = \beta\omega R_y P_{yscmax} = 201.82$ kip

$T_{max} = \omega R_y P_{yscmax} = 172.50$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.25$

$C_{max} = \beta\omega R_y P_{yscmax} = 201.82$ kip

$T_{max} = \omega R_y P_{yscmax} = 172.50$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E34

P_u (kip) = 69.31 ϕF_{yscAsc} (kip) = 102.60 $P_u/\phi F_{yscAsc} = 0.68$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 102.60

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 34.20 \leq P_{ysc} \leq 1.2 P_{ysc} = 136.80$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.625 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 0.838%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.463%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 201.82

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 25

Member No: 78

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.25$

$C_{max} = \beta\omega R_y P_{yscmax} = 201.82$ kip

$T_{max} = \omega R_y P_{yscmax} = 172.50$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.17$ $\omega = 1.25$

$C_{max} = \beta\omega R_y P_{yscmax} = 201.82$ kip

$T_{max} = \omega R_y P_{yscmax} = 172.50$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{35}$

P_u (kip) = 72.28 ϕF_{yscAsc} (kip) = 102.60 $P_u / \phi F_{yscAsc} = 0.70$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 102.60

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 34.20 \leq P_{ysc} \leq 1.2 P_{ysc} = 136.80$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.625 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 0.838%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.463%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 201.82

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: 2nd Floor

Frame No: 25

Member No: 338

Fy (ksi): 50.00

Size: W27X84

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Vertical Resultant (kip) = 50.0

Horizontal Resultant (kip) = 465.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Vertical Resultant (kip) = 50.0

Horizontal Resultant (kip) = 465.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 207.12 **OK**

Required strength of lateral brace (A-6-7) = 10.30 kip

Required stiffness of bracing (A-6-8) = 6866.21 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.81 Limit = 9.18 **OK**

Web h/tw = 52.65 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 25

Member No: 69

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_6.00

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

$C_{max} = \beta\omega R_y P_{yscmax} = 413.61$ kip

$T_{max} = \omega R_y P_{yscmax} = 350.52$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

$C_{max} = \beta\omega R_y P_{yscmax} = 413.61$ kip

$T_{max} = \omega R_y P_{yscmax} = 350.52$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{20}$

P_u (kip) = 180.63 ϕF_{yscAsc} (kip) = 205.20 $P_u/\phi F_{yscAsc} = 0.88$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 205.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 68.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 273.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.676 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 1.265%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 413.61

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 25

Member No: 70

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_6.00

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

Cmax = $\beta\omega R_y P_{yscmax} = 413.61$ kip

Tmax = $\omega R_y P_{yscmax} = 350.52$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

Cmax = $\beta\omega R_y P_{yscmax} = 413.61$ kip

Tmax = $\omega R_y P_{yscmax} = 350.52$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E35

Pu (kip) = 181.14 ϕF_{yscAsc} (kip) = 205.20 Pu/ ϕF_{yscAsc} = 0.88

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 205.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 68.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 273.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.677 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2x Design Story Drift: 1.265%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 413.61

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 26

Member No: 2

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 34.64 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.07 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	34.64
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 34.64 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 81.36 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 26

Member No: 3

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 33.28 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.07 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	33.28
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 33.28 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 81.77 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Beam Parameters

Story: Roof

Frame No: 26

Member No: 619

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.90

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.90

Vertical Resultant (kip) = 14.1

Horizontal Resultant (kip) = 94.1

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.90

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 59.77

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 78.90

Vertical Resultant (kip) = 14.1

Horizontal Resultant (kip) = 94.1

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 26

Member No: 18

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.90$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.90$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp - 1.000 E17

P_u (kip) = 32.32 ϕF_{yscAsc} (kip) = 35.10 $P_u / \phi F_{yscAsc} = 0.92$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 35.10

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 11.70 \leq P_{ysc} \leq 1.2 P_{ysc} = 46.80$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.636 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 0.900%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.560%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 78.90

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 26

Member No: 19

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.90$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 78.90$ kip

$T_{max} = \omega R_y P_{yscmax} = 59.77$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E17$

P_u (kip) = 32.82 ϕF_{yscAsc} (kip) = 35.10 $P_u / \phi F_{yscAsc} = 0.94$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 35.10

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 11.70 \leq P_{ysc} \leq 1.2 P_{ysc} = 46.80$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.636 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 0.900%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.560%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 78.90

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



RAM Structural System 24.00.01.18

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 26

Member No: 3

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 126.46 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 50.94

Max Pu/φPn = 0.29 OK

Tension: Max Pu (kip) = 22.73 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -50.94

Max Pu/φPn = 0.04 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	126.46
<i>Tension:</i>	0.699 D + 1.000 Ecl	22.73

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 126.46 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 59.30 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 26

Member No: 4

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 127.41 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 50.94

Max Pu/φPn = 0.30 OK

Tension: Max Pu (kip) = 22.25 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -50.94

Max Pu/φPn = 0.04 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	127.41
Tension:	0.699 D + 1.000 Ecl	22.25

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 127.41 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 59.27 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Beam Parameters

Story: 2nd Floor

Frame No: 26

Member No: 1126

Fy (ksi): 50.00

Size: W24X55

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 186.15

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 186.15

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 215.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 186.15

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 167.70

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 186.15

Vertical Resultant (kip) = 14.6

Horizontal Resultant (kip) = 215.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 134.27 **OK**

Required strength of lateral brace (A-6-7) = 6.38 kip

Required stiffness of bracing (A-6-8) = 4254.89 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 6.94 Limit = 9.18 **OK**

Web h/tw = 54.63 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 26

Member No: 63

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 186.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 167.70$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 186.15$ kip

$T_{max} = \omega R_y P_{yscmax} = 167.70$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E34

P_u (kip) = 95.18 ϕF_{yscAsc} (kip) = 105.30 $P_u/\phi F_{yscAsc} = 0.90$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.763 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.138%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.265%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 186.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 26

Member No: 64

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

Cmax = $\beta\omega R_y P_{yscmax} = 186.15$ kip

Tmax = $\omega R_y P_{yscmax} = 167.70$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

Cmax = $\beta\omega R_y P_{yscmax} = 186.15$ kip

Tmax = $\omega R_y P_{yscmax} = 167.70$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E35

Pu (kip) = 95.61 ϕF_{yscAsc} (kip) = 105.30 $P_u / \phi F_{yscAsc} = 0.91$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 105.30

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 35.10 \leq P_{ysc} \leq 1.2 P_{ysc} = 140.40$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.763 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.138%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.265%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 186.15

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 27

Member No: 39

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 50.68 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.11 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	50.68
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 50.68 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 76.54 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 27

Member No: 40

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 50.66 --- Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$

Ecl (kips) = 0.00

Max Pu/ ϕP_n = 0.11 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	$1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{cl}$	50.66
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 50.66 ϕP_y (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 76.54 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣM_{pc} (kip-ft)	580.80	264.60

Column Splice Shear Force Required $\Sigma M_{pc} / H$

Where ΣM_{pc} is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof Frame No: 27 Member No: 407

Fy (ksi): 36.00 Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 227.04

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 254.28

Right Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 227.04

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 252.01

Vertical Resultant (kip) = 20.0

Horizontal Resultant (kip) = 326.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 227.04

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 254.28

Right Brace: Tens = $\omega R_y P_{ysc-max}$ (kip) = 227.04

Cmax = $\beta \omega R_y P_{ysc-max}$ (kip) = 252.01

Vertical Resultant (kip) = 20.0

Horizontal Resultant (kip) = 326.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 128.76 **OK**

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 **OK**

Web h/tw = 53.71 Limit = 36.38 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 27

Member No: 22

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_4.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.32$

Cmax = $\beta\omega R_y P_{yscmax} = 254.28$ kip

Tmax = $\omega R_y P_{yscmax} = 227.04$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.12$ $\omega = 1.32$

Cmax = $\beta\omega R_y P_{yscmax} = 254.28$ kip

Tmax = $\omega R_y P_{yscmax} = 227.04$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E20

Pu (kip) = 94.59 ϕF_{yscAsc} (kip) = 140.40 $Pu/\phi F_{yscAsc} = 0.67$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 140.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 46.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 187.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.546 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 0.864%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.586%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 254.28

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 27

Member No: 23

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_4.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.32$

Cmax = $\beta\omega R_y P_{yscmax} = 252.01$ kip

Tmax = $\omega R_y P_{yscmax} = 227.04$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.32$

Cmax = $\beta\omega R_y P_{yscmax} = 252.01$ kip

Tmax = $\omega R_y P_{yscmax} = 227.04$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E17

Pu (kip) = 95.87 ϕF_{yscAsc} (kip) = 140.40 Pu/ ϕF_{yscAsc} = 0.68

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 140.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 46.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 187.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.546 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2x Design Story Drift: 0.864%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2% Design Story Drift: 1.586%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 252.01

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 27

Member No: 40

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 306.94 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 177.66

Max Pu/φPn = 0.71 OK

Tension: Max Pu (kip) = 128.23 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -176.82

Max Pu/φPn = 0.21 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	306.94
Tension:	0.699 D + 1.000 Ecl	128.23

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 306.94 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 53.49 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 27

Member No: 41

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 305.53 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 175.16

Max Pu/φPn = 0.71 OK

Tension: Max Pu (kip) = 126.99 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -175.99

Max Pu/φPn = 0.21 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	305.53
<i>Tension:</i>	0.699 D + 1.000 Ecl	126.99

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 305.53 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 53.54 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 27

Member No: 1138

Fy (ksi): 50.00

Size: W27X84

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Vertical Resultant (kip) = 50.0

Horizontal Resultant (kip) = 465.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Vertical Resultant (kip) = 50.0

Horizontal Resultant (kip) = 465.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 207.12 **OK**

Required strength of lateral brace (A-6-7) = 10.30 kip

Required stiffness of bracing (A-6-8) = 6866.21 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.81 Limit = 9.18 **OK**

Web h/tw = 52.65 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 27

Member No: 67

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_6.00

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

Cmax = $\beta\omega R_y P_{yscmax} = 413.61$ kip

Tmax = $\omega R_y P_{yscmax} = 350.52$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

Cmax = $\beta\omega R_y P_{yscmax} = 413.61$ kip

Tmax = $\omega R_y P_{yscmax} = 350.52$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E34

Pu (kip) = 181.10 ϕF_{yscAsc} (kip) = 205.20 Pu/ ϕF_{yscAsc} = 0.88

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 205.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 68.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 273.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.676 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2x Design Story Drift: 1.265%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 413.61

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 27

Member No: 68

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_6.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

$C_{max} = \beta\omega R_y P_{yscmax} = 413.61$ kip

$T_{max} = \omega R_y P_{yscmax} = 350.52$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

$C_{max} = \beta\omega R_y P_{yscmax} = 413.61$ kip

$T_{max} = \omega R_y P_{yscmax} = 350.52$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E35

P_u (kip) = 181.55 ϕF_{yscAsc} (kip) = 205.20 $P_u/\phi F_{yscAsc} = 0.88$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 205.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 68.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 273.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.676 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.265%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 413.61

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof Frame No: 28 Member No: 174

Fy (ksi): 50.00 Size: HSS6X6X3/8

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 21.12 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.11 OK

Tension: Max Pu (kip) = 0.55 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -9.77

Max Pu/φPn = 0.00 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	21.12
Tension:	0.699 D + 1.000 Ecl	0.55

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf = 14.19 Limit = 24.02 OK

Web h/tw = 14.19 Limit = 24.02 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	356.23	198.13

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 28

Member No: 175

Fy (ksi): 50.00

Size: HSS6X6X3/8

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 28.97 --- Combination: 1.401 D + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 0.00

Max Pu/φPn = 0.16 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.200 Sp + 1.000 Ecl	28.97
Tension:	No tension on column	

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf = 14.19 Limit = 24.02 OK

Web h/tw = 14.19 Limit = 24.02 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	356.23	198.13

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Roof

Frame No: 28

Member No: 892

Fy (ksi): 50.00

Size: W21X50

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Vertical Resultant (kip) = 11.9

Horizontal Resultant (kip) = 179.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 124.20

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 140.35

Vertical Resultant (kip) = 11.9

Horizontal Resultant (kip) = 179.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 130.39 **OK**

Required strength of lateral brace (A-6-7) = 5.97 kip

Required stiffness of bracing (A-6-8) = 3980.59 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 6.10 Limit = 9.18 **OK**

Web h/tw = 49.26 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 28

Member No: 81

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 140.35$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 140.35$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.200 Sp + 1.000 E10

P_u (kip) = 47.21 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.69$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.531 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 0.783%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 140.35

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 28

Member No: 82

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_2.00 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 140.35$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.13$ $\omega = 1.35$

$C_{max} = \beta\omega R_y P_{yscmax} = 140.35$ kip

$T_{max} = \omega R_y P_{yscmax} = 124.20$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E11

P_u (kip) = 47.94 ϕF_{yscAsc} (kip) = 68.40 $P_u/\phi F_{yscAsc} = 0.70$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 68.40

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 22.80 \leq P_{ysc} \leq 1.2 P_{ysc} = 91.20$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.531 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 0.783%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 140.35

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 28

Member No: 99

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 200.77 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 97.19

Max Pu/φPn = 0.47 OK

Tension: Max Pu (kip) = 61.42 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -97.19

Max Pu/φPn = 0.10 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	200.77
Tension:	0.699 D + 1.000 Ecl	61.42

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 200.77 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 56.91 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 28

Member No: 100

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 252.99 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 149.30

Max Pu/φPn = 0.59 OK

Tension: Max Pu (kip) = 113.36 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -149.30

Max Pu/φPn = 0.18 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	252.99
Tension:	0.699 D + 1.000 Ecl	113.36

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 252.99 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 55.23 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: 2nd Floor

Frame No: 28

Member No: 1156

Fy (ksi): 50.00

Size: W27X84

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Vertical Resultant (kip) = 50.0

Horizontal Resultant (kip) = 465.5

β and ω Factors at 2% Story Drift

Left Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Right Brace: Tens = $\omega R_y P_{ySC-max}$ (kip) = 350.52

Cmax = $\beta \omega R_y P_{ySC-max}$ (kip) = 413.61

Vertical Resultant (kip) = 50.0

Horizontal Resultant (kip) = 465.5

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 207.12 **OK**

Required strength of lateral brace (A-6-7) = 10.30 kip

Required stiffness of bracing (A-6-8) = 6866.21 kip / Lb

$C_d = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($C_d=2$)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.81 Limit = 9.18 **OK**

Web h/tw = 52.65 Limit = 36.05 **NG**

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 28

Member No: 79

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_6.00

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

Cmax = $\beta\omega R_y P_{yscmax} = 413.61$ kip

Tmax = $\omega R_y P_{yscmax} = 350.52$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

Cmax = $\beta\omega R_y P_{yscmax} = 413.61$ kip

Tmax = $\omega R_y P_{yscmax} = 350.52$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E28

Pu (kip) = 177.82 ϕF_{yscAsc} (kip) = 205.20 Pu/ ϕF_{yscAsc} = 0.87

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 205.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 68.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 273.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift (2 Δ_{bm}): 2.630 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2x Design Story Drift: 1.243%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 413.61

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 28

Member No: 80

Fymin (ksi): 38.00

Fymax (ksi): 46.00

Size: CoreBRB_6.00

Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

$C_{max} = \beta\omega R_y P_{yscmax} = 413.61$ kip

$T_{max} = \omega R_y P_{yscmax} = 350.52$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.18$ $\omega = 1.27$

$C_{max} = \beta\omega R_y P_{yscmax} = 413.61$ kip

$T_{max} = \omega R_y P_{yscmax} = 350.52$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 0.200 S_p + 1.000 E_{11}$

P_u (kip) = 177.58 ϕF_{yscAsc} (kip) = 205.20 $P_u/\phi F_{yscAsc} = 0.87$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 205.20

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 68.40 \leq P_{ysc} \leq 1.2 P_{ysc} = 273.60$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.630 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 1.243%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.451%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 413.61

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof Frame No: 29 Member No: 108

Fy (ksi): 36.00 Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 103.39 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 76.85

Max Pu/φPn = 0.21 OK

Tension: Max Pu (kip) = 90.13 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -101.44

Max Pu/φPn = 0.15 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	103.39
Tension:	0.699 D + 1.000 Ecl	90.13

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 103.39 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 60.70 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: Roof

Frame No: 29

Member No: 109

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 103.50 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 76.85

Max Pu/φPn = 0.21 OK

Tension: Max Pu (kip) = 89.35 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -100.68

Max Pu/φPn = 0.14 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	103.50
Tension:	0.699 D + 1.000 Ecl	89.35

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 103.50 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 60.66 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	580.80	264.60

Column Splice Shear Force Required ΣMpc / H

Where ΣMpc is the sum of column plastic strengths above and below splice.

See code for more information on the Required Shear Strength

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: Roof

Frame No: 29

Member No: 405

Fy (ksi): 36.00

Size: W21X44

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 128.76 OK

Required strength of lateral brace (A-6-7) = 5.09 kip

Required stiffness of bracing (A-6-8) = 3392.00 kip / Lb

$Cd = 1.0$ assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values ($Cd=2$)

Design Check Error

Only one of two braces framing into beam has a BRB size.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

Flange b/tf = 7.22 Limit = 9.27 OK

Web h/tw = 53.71 Limit = 36.38 NG

Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/RyFy}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 29

Member No: 33

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.31$ $\omega = 1.39$

Cmax = $\beta\omega R_y P_{yscmax} = 137.02$ kip

Tmax = $\omega R_y P_{yscmax} = 104.60$ kip

β and ω Factors at 2% Story Drift

Ry = 1.0 for CoreBrace Braces. User Assigned $\beta = 1.31$ $\omega = 1.39$

Cmax = $\beta\omega R_y P_{yscmax} = 137.02$ kip

Tmax = $\omega R_y P_{yscmax} = 104.60$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 1.000 E11

Pu (kip) = 44.29 ϕF_{yscAsc} (kip) = 61.42 Pu/ ϕF_{yscAsc} = 0.72

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum Pysc_s $\geq 0.9 P_{ysc}$ (kip) = 61.42

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 20.48 \leq P_{ysc} \leq 1.2 P_{ysc} = 81.90$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.452 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2x Design Story Drift: 0.791%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 Fpx_min Y EQ_User

Required core strain at 2% Design Story Drift: 1.546%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : Cmax (kip) = 137.02

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Building Code: IBC

Brace Parameters

Story: Roof

Frame No: 29

Member No: 34

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_1.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 104.60$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.32$ $\omega = 1.39$

$C_{max} = \beta\omega R_y P_{yscmax} = 138.07$ kip

$T_{max} = \omega R_y P_{yscmax} = 104.60$ kip

Warning: $\beta > 1.3$. Lateral Force Distribution requirements not met. See Section F4.4c.

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: $1.401 D + 0.500 L_p + 1.000 E_{10}$

P_u (kip) = 42.85 ϕF_{yscAsc} (kip) = 61.42 $P_u / \phi F_{yscAsc} = 0.70$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 61.42

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 20.48 \leq P_{ysc} \leq 1.2 P_{ysc} = 81.90$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 1.389 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2x Design Story Drift: 0.757%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 2.838 in

Load Case: E38 $F_{px_min Y}$ EQ_User

Required core strain at 2% Design Story Drift: 1.546%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 138.07

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 29

Member No: 133

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 153.94 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 79.62

Max Pu/φPn = 0.36 OK

Tension: Max Pu (kip) = 70.70 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -97.52

Max Pu/φPn = 0.11 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
Compression:	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	153.94
Tension:	0.699 D + 1.000 Ecl	70.70

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 153.94 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 58.42 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Column Parameters

Story: 2nd Floor

Frame No: 29

Member No: 134

Fy (ksi): 36.00

Size: W12X65

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

D1.4a Required Strength --- OK

Adjusted Brace Strength Criteria Applied.

D1.4a(a) – Ecl based on Adjusted Brace Strength:

Compression: Max Pu (kip) = 146.47 --- Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl

Ecl (kips) = 80.00

Max Pu/φPn = 0.34 OK

Tension: Max Pu (kip) = 74.84 --- Combination: 0.699 D + 1.000 Ecl

Ecl (kips) = -98.68

Max Pu/φPn = 0.12 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:

	Load Combination	Force(kip)
<i>Compression:</i>	1.401 D + 0.500 Lp + 0.200 Sp + 1.000 Ecl	146.47
<i>Tension:</i>	0.699 D + 1.000 Ecl	74.84

Ecl - Capacity limited seismic load from adjusted brace strengths in compression or in tension.

Required shear for column splice is max result from D2.5b and D2.5c

Refer to AISC 341 section D2.5b for additional detailing requirements.

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pu (kip) = 146.47 φPy (kip) = 928.26

Flange b/tf = 9.92 Limit = 9.27 NG

Web h/tw = 24.87 Limit = 58.66 OK

F4.6d Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Beam Parameters

Story: 2nd Floor

Frame No: 29

Member No: 1164

Fy (ksi): 50.00

Size: W24X55

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.4a V- and Inverted V- Braced Frames --- Additional Check Required

Beam must be continuous between columns **OK**

1. Beam must be designed for unbalanced horizontal brace forces on the beam ends in addition to the unbalanced forces below:

β and ω Factors at 2x Design Story Drift

Top Left Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 104.60

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 137.02

Top Right Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 104.60

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 138.07

Bottom Left Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 209.62

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 232.68

Bottom Right Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 224.25

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 248.92

Vertical Resultant (kip) = 17.9

Horizontal Resultant (kip) = 115.4

β and ω Factors at 2% Story Drift

Top Left Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 104.60

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 137.02

Top Right Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 104.60

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 138.07

Bottom Left Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 209.62

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 232.68

Bottom Right Brace: Tens = $\omega R_y P_{ysec-max}$ (kip) = 224.25

Cmax = $\beta \omega R_y P_{ysec-max}$ (kip) = 248.92

Vertical Resultant (kip) = 17.9

Horizontal Resultant (kip) = 115.4

Beam must be capable of carrying gravity loads without braces

2. Lateral Bracing Requirements along Beam (D1.2a Moderately Ductile)

Max Lu (in) = 128.04 Lu Limit = 134.27 **OK**

Required strength of lateral brace (A-6-7) = 6.38 kip

Required stiffness of bracing (A-6-8) = 4254.89 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)

Lb = distance between braces (in)

The brace closest to the point of inflection should be designed for two times these values (Cd=2)

F4.5a Basic Requirements (D1.1 Moderately Ductile) --- Additional Check Required

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Flange b/tf =	6.94	Limit =	9.18	OK
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Web h/tw =	54.63	Limit =	36.05	NG
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Note: Beam axial load from F4.3 Analysis not determined; Web h/tw limit assumed to be $1.56 \sqrt{E/R_y F_y}$.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 29

Member No: 81

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 232.68$ kip

$T_{max} = \omega R_y P_{yscmax} = 209.63$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 232.68$ kip

$T_{max} = \omega R_y P_{yscmax} = 209.63$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E10

P_u (kip) = 113.50 ϕF_{yscAsc} (kip) = 131.63 $P_u/\phi F_{yscAsc} = 0.86$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 131.63

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 43.88 \leq P_{ysc} \leq 1.2 P_{ysc} = 175.50$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.648 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.101%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.277%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 232.68

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.

Seismic Provisions Member Code Check



RAM Structural System 24.00.01.18

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Steel Code: AISC341-16 - LRFD

Bentley Building Code: IBC

Brace Parameters

Story: 2nd Floor

Frame No: 29

Member No: 82

Fymin (ksi): 39.00

Fymax (ksi): 43.00

Size: CoreBRB_3.75 Connection Type: BOLTED

Frame Type: Buckling Restrained Braced Frame - Chevron

Criteria

Apply one and two story exceptions where applicable

F4.2 Basis of Design --- OK

F4.2a Adjusted Brace Strength

β and ω Factors at 2x Design Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 232.68$ kip

$T_{max} = \omega R_y P_{yscmax} = 209.63$ kip

β and ω Factors at 2% Story Drift

$R_y = 1.0$ for CoreBrace Braces. User Assigned $\beta = 1.11$ $\omega = 1.30$

$C_{max} = \beta\omega R_y P_{yscmax} = 232.68$ kip

$T_{max} = \omega R_y P_{yscmax} = 209.63$ kip

F4.5 Bracing Members

F4.5b(2) Available Strength - Steel Core --- OK

Load Combination: 1.401 D + 0.500 Lp + 0.200 Sp + 1.000 E11

P_u (kip) = 111.26 ϕF_{yscAsc} (kip) = 131.63 $P_u/\phi F_{yscAsc} = 0.85$

F4.5b(3) Conformance Demonstration

K3.2(b) Subassembly Test: Test with minimum $P_{ysc_s} \geq 0.9 P_{ysc}$ (kip) = 131.63

K3.3c(b) Brace Test: Test with core yield capacity within $0.3 P_{ysc} = 43.88 \leq P_{ysc} \leq 1.2 P_{ysc} = 175.50$

2x Design Story Drift

K3.4c Required Elongation at 2 times design story drift ($2\Delta_{bm}$): 2.709 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2x Design Story Drift: 1.126%

2% Story Drift

K3.4c Required Elongation at 2% story drift: 3.070 in

Load Case: E38 F_{px_min} Y EQ_User

Required core strain at 2% Design Story Drift: 1.277%

F4.6 Diagonal Brace Connections

F4.6c(1) --- Required Strength

Strength of the bracing connection : C_{max} (kip) = 232.68

F4.6c(2) --- Gusset Plates

Lateral bracing shall be consistent with that used in testing.

Verify brace properties with the brace manufacturer prior to finalizing structural design.



QUANTUM | CONSULTING ENGINEERS

**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

1015 39th AVENUE SE
PUYALLUP, WA 98374

QUANTUM JOB NUMBER: 19305.04

RAM BUILDING ANALYSIS

CORE BRACE SUMMARY

CoreBrace Summary



Bentley Building Code: IBC

RAM Frame 24.00.01.18
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Cd/Ie: 3.33

Frame #1:

Level: Roof

Brace #	Size	Type	Pyse Min (kips)	Pyse Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
39	CoreBRB_2.00	BOLTED	78.0	86.0	1.35	1.30	220	0.871
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)
	2Δbm 2%	2Δbm	2Δbm 2%	2Δbm 2%				
39	2.171 3.380	0.846 1.318	1.31 1.31	1.11 1.11	112.7	125.1	123.9	137.6

Frame #2:

Level: 2nd Floor

Brace #	Size	Type	Pyse Min (kips)	Pyse Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
37	CoreBRB_7.50	BOLTED	292.5	345.0	1.45	1.33	1082	0.965
38	CoreBRB_7.50	BOLTED	292.5	345.0	1.45	1.33	1082	0.979
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)
	2Δbm 2%	2Δbm	2Δbm 2%	2Δbm 2%				
37	2.604 2.604	1.314 1.313	1.29 1.29	1.12 1.12	445.0	498.5	489.6	548.3
38	2.604 2.604	1.314 1.313	1.29 1.29	1.12 1.12	445.0	498.5	489.6	548.3

CoreBrace Summary



Bentley Building Code: IBC

RAM Frame 24.00.01.18
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Frame #3:

Level: Roof

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR			
95	CoreBRB_2.25	BOLTED	85.5	103.5	1.35	1.39	344	0.832			
96	CoreBRB_2.25	BOLTED	85.5	103.5	1.35	1.39	344	0.771			
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)			
	2Δbm	2%	2Δbm	2%							
95	1.306	2.455	0.807	1.518	1.35	1.35	1.12	139.7	156.5	153.7	172.1
96	1.306	2.455	0.807	1.518	1.35	1.35	1.12	139.7	156.5	153.7	172.1

Level: 2nd Floor

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR			
39	CoreBRB_3.75	BOLTED	146.3	161.3	1.45	1.29	541	0.969			
40	CoreBRB_3.75	BOLTED	146.3	161.3	1.40	1.23	522	0.974			
Brace #	Elong (in)	ε (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)			
	2Δbm	2%	2Δbm	2%	2Δbm	2%					
39	2.759	2.604	1.334	1.258	1.31	1.31	1.11	211.2	234.5	232.4	257.9
40	2.853	2.604	1.305	1.191	1.29	1.29	1.11	208.0	230.9	228.8	254.0

Frame #4:

Level: Roof

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
91	CoreBRB_2.25	BOLTED	85.5	103.5	1.35	1.39	344	0.809
92	CoreBRB_2.25	BOLTED	85.5	103.5	1.35	1.39	344	0.863
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)
	2Δbm	2%	2Δbm 2%	2Δbm 2%				

CoreBrace Summary



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91	1.355	2.455	0.838	1.518	1.35	1.35	1.12	1.12	139.7	156.5	153.7	172.1
92	1.356	2.455	0.838	1.518	1.35	1.35	1.12	1.12	139.7	156.5	153.7	172.1

Level: 2nd Floor

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR				
41	CoreBRB_3.25	BOLTED	126.8	139.8	1.40	1.22	453	0.982				
42	CoreBRB_3.25	BOLTED	126.8	139.8	1.40	1.22	453	0.992				
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)				
	2Δbm	2%	2Δbm	2%	2Δbm	2%						
41	2.828	2.604	1.285	1.183	1.29	1.29	1.11	1.11	180.3	200.1	198.3	220.1
42	2.828	2.604	1.285	1.183	1.29	1.29	1.11	1.11	180.3	200.1	198.3	220.1

Frame #6:

Level: Roof

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
6	CoreBRB_2.75	BOLTED	107.3	118.3	1.35	1.35	420	0.666
7	CoreBRB_2.75	BOLTED	107.3	118.3	1.35	1.35	420	0.684
41	CoreBRB_2.00	BOLTED	76.0	92.0	1.35	1.32	305	0.755
42	CoreBRB_2.00	BOLTED	76.0	92.0	1.35	1.32	305	0.766

Brace #	Elong (in)		ϵ (%)		ω		β		Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)
	2Δbm	2%	2Δbm	2%	2Δbm	2%	2Δbm	2%				
6	1.200	2.455	0.736	1.506	1.33	1.33	1.12	1.12	157.3	176.1	173.0	193.8
7	1.200	2.455	0.736	1.506	1.33	1.33	1.12	1.12	157.3	176.1	173.0	193.8
41	1.259	2.455	0.704	1.373	1.35	1.35	1.12	1.12	124.2	139.1	136.6	153.0
42	1.259	2.455	0.704	1.373	1.35	1.35	1.12	1.12	124.2	139.1	136.6	153.0

Level: 2nd Floor

CoreBrace Summary



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Brace #	Size	Type	Pyse Min (kips)	Pyse Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
45	CoreBRB_3.00	BOLTED	117.0	129.0	1.30	1.21	388	0.740
46	CoreBRB_3.00	BOLTED	117.0	129.0	1.30	1.21	388	0.752
93	CoreBRB_3.00	BOLTED	117.0	129.0	1.30	1.21	388	0.743
94	CoreBRB_3.00	BOLTED	117.0	129.0	1.30	1.21	388	0.750
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm 2%	2Δbm 2%	2Δbm 2%	2Δbm 2%	(kips)	(kips)	(kips)	(kips)
45	2.805	2.604	1.260	1.169	1.21	1.21	1.17	1.17
					1.17	1.17	1.17	1.17
					1.17	1.17	1.17	1.17
46	2.812	2.604	1.263	1.169	1.21	1.21	1.17	1.17
					1.17	1.17	1.17	1.17
					1.17	1.17	1.17	1.17
93	2.811	2.604	1.263	1.169	1.21	1.21	1.17	1.17
					1.17	1.17	1.17	1.17
					1.17	1.17	1.17	1.17
94	2.806	2.604	1.260	1.169	1.21	1.21	1.17	1.17
					1.17	1.17	1.17	1.17
					1.17	1.17	1.17	1.17

Frame #7:

Level: 2nd Floor

Brace #	Size	Type	Pyse Min (kips)	Pyse Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
47	CoreBRB_2.75	BOLTED	107.3	118.3	1.40	1.25	383	0.767
48	CoreBRB_2.75	BOLTED	107.3	118.3	1.40	1.25	383	0.776
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm 2%	2Δbm 2%	2Δbm 2%	2Δbm 2%	(kips)	(kips)	(kips)	(kips)
47	2.805	2.604	1.346	1.249	1.30	1.30	1.11	1.11
					1.11	1.11	1.11	1.11
					1.11	1.11	1.11	1.11
48	2.805	2.604	1.346	1.249	1.30	1.30	1.11	1.11
					1.11	1.11	1.11	1.11
					1.11	1.11	1.11	1.11

Frame #8:

Level: 2nd Floor

Brace #	Size	Type	Pyse Min (kips)	Pyse Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
49	CoreBRB_2.75	BOLTED	107.3	118.3	1.40	1.25	383	0.743

CoreBrace Summary



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50	CoreBRB_2.75	BOLTED	107.3	118.3	1.40	1.25	383	0.753
Brace #	Elong (in)	ϵ (%)	ω	β			Tmax	1.1Tmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)
49	2.801	2.604	1.344	1.249	1.31	1.31	1.11	1.11
							154.9	171.9
50	2.801	2.604	1.344	1.249	1.31	1.31	1.11	1.11
							154.9	171.9
							170.4	189.1
							170.4	189.1

Frame #9:

Level: Roof

Brace #	Size	Type	PySc Min (kips)	PySc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
8	CoreBRB_2.00	BOLTED	78.0	86.0	1.30	1.32	294	0.667
9	CoreBRB_2.00	BOLTED	78.0	86.0	1.30	1.32	294	0.752

Brace #	Elong (in)	ϵ (%)	ω	β			Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)	(kips)	(kips)
8	1.375	2.455	0.769	1.373	1.30	1.30	1.11	1.11	111.8	124.1
9	1.374	2.455	0.769	1.373	1.30	1.30	1.11	1.11	111.8	124.1
							123.0	123.0	123.0	136.5
							123.0	123.0	123.0	136.5

Level: 2nd Floor

Brace #	Size	Type	PySc Min (kips)	PySc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
113	CoreBRB_2.75	BOLTED	104.5	126.5	1.30	1.25	356	0.713
114	CoreBRB_2.75	BOLTED	104.5	126.5	1.30	1.25	356	0.721

Brace #	Elong (in)	ϵ (%)	ω	β			Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)	(kips)	(kips)
113	2.811	2.604	1.348	1.249	1.34	1.34	1.12	1.12	169.5	189.9
114	2.810	2.604	1.348	1.249	1.34	1.34	1.12	1.12	169.5	189.9
							186.5	186.5	186.5	208.8
							186.5	186.5	186.5	208.8

Frame #10:

Level: Roof

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Level: 2nd Floor

Frame #11:

Brace #	Type	Size	Pysc Min	Pysc Max	Stiff Mult	Stiff Mult	Keff	Core max
1	1	1	1	1	1	1	1	1
2	1	1	1	1	1	1	1	1
3	1	1	1	1	1	1	1	1
4	1	1	1	1	1	1	1	1
5	1	1	1	1	1	1	1	1
6	1	1	1	1	1	1	1	1
7	1	1	1	1	1	1	1	1
8	1	1	1	1	1	1	1	1
9	1	1	1	1	1	1	1	1
10	1	1	1	1	1	1	1	1
11	1	1	1	1	1	1	1	1
12	1	1	1	1	1	1	1	1
13	1	1	1	1	1	1	1	1
14	1	1	1	1	1	1	1	1
15	1	1	1	1	1	1	1	1
16	1	1	1	1	1	1	1	1
17	1	1	1	1	1	1	1	1
18	1	1	1	1	1	1	1	1
19	1	1	1	1	1	1	1	1
20	1	1	1	1	1	1	1	1
21	1	1	1	1	1	1	1	1
22	1	1	1	1	1	1	1	1
23	1	1	1	1	1	1	1	1
24	1	1	1	1	1	1	1	1
25	1	1	1	1	1	1	1	1
26	1	1	1	1	1	1	1	1
27	1	1	1	1	1	1	1	1
28	1	1	1	1	1	1	1	1
29	1	1	1	1	1	1	1	1
30	1	1	1	1	1	1	1	1
31	1	1	1	1	1	1	1	1
32	1	1	1	1	1	1	1	1
33	1	1	1	1	1	1	1	1
34	1	1	1	1	1	1	1	1
35	1	1	1	1	1	1	1	1
36	1	1	1	1	1	1	1	1
37	1	1	1	1	1	1	1	1
38	1	1	1	1	1	1	1	1
39	1	1	1	1	1	1	1	1
40	1	1	1	1	1	1	1	1
41	1	1	1	1	1	1	1	1
42	1	1	1	1	1	1	1	1
43	1	1	1	1	1	1	1	1
44	1	1	1	1	1	1	1	1
45	1	1	1	1	1	1	1	1
46	1	1	1	1	1	1	1	1
47	1	1	1	1	1	1	1	1
48	1	1	1	1	1	1	1	1
49	1	1	1	1	1	1	1	1
50	1	1	1	1	1	1	1	1
51	1	1	1	1	1	1	1	1
52	1	1	1	1	1	1	1	1
53	1	1	1	1	1	1	1	1
54	1	1	1	1	1	1	1	1
55	1	1	1	1	1	1	1	1
56	1	1	1	1	1	1	1	1
57	1	1	1					

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CoreBrace Summary



Bentley® Building Code: IBC

Level: 2nd Floor

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	K _{eff} (kips/in)	Core max DCR				
109	CoreBRB_2.00	BOLTED	76.0	92.0	1.30	1.22	259	0.788				
110	CoreBRB_2.00	BOLTED	76.0	92.0	1.30	1.22	259	0.794				
Brace #	Elong (in)	ε (%)	ω		β		T _{max}	C _{max}	1.1T _{max}	1.1C _{max}		
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)	(kips)	(kips)		
	2.831	2.604	1.262	1.161	1.34	1.34	1.12	1.12	123.3	138.1	135.6	151.9
	2.831	2.604	1.262	1.161	1.34	1.34	1.12	1.12	123.3	138.1	135.6	151.9

Frame #12:

Level: 2nd Floor

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	K _{eff} (kips/in)	Core max DCR
91	CoreBRB_2.75	BOLTED	107.3	118.3	1.40	1.25	383	0.798
92	CoreBRB_2.75	BOLTED	107.3	118.3	1.40	1.25	383	0.804
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)
91	2Δbm	2%	2Δbm	2%	2Δbm	2%	2Δbm	2%
92	2.786	2.604	1.337	1.249	1.31	1.31	1.11	1.11
91	2.786	2.604	1.337	1.249	1.31	1.31	1.11	1.11
92	2.786	2.604	1.337	1.249	1.31	1.31	1.11	1.11

Frame #13:

Level: Roof

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	K _{eff} (kips/in)	Core max DCR
37	CoreBRB_2.25	BOLTED	87.8	96.8	1.40	1.38	356	0.727
38	CoreBRB_2.25	BOLTED	87.8	96.8	1.40	1.38	356	0.738
Brace #	Elong (in)	ϵ (%)	ω	β	T _{max} (kips)	C _{max} (kips)	1.1T _{max} (kips)	1.1C _{max} (kips)
	2Δbm	2%	2Δbm	2%				

CoreBrace Summary



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37	1.266	2.455	0.774	1.501	1.34	1.34	1.12	1.12	129.6	145.2	142.6	159.7
38	1.266	2.455	0.774	1.501	1.34	1.34	1.12	1.12	129.6	145.2	142.6	159.7

Level: 2nd Floor

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
111	CoreBRB_2.00	BOLTED	76.0	92.0	1.30	1.22	259	0.787
112	CoreBRB_2.00	BOLTED	76.0	92.0	1.30	1.22	259	0.793
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)
111	2.831	2.604	1.262	1.161	1.34	1.12	123.3	138.1
112	2.831	2.604	1.262	1.161	1.34	1.12	123.3	138.1

Frame #14:

Level: Roof

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR				
59	CoreBRB_2.75	BOLTED	104.5	126.5	1.40	1.38	435	0.749				
60	CoreBRB_2.75	BOLTED	104.5	126.5	1.40	1.38	435	0.727				
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)				
	2Δbm	2%	2Δbm	2%	2Δbm	2%						
59	1.192	2.455	0.749	1.542	1.36	1.36	1.14	1.14	172.0	196.1	189.2	215.7
60	1.192	2.455	0.749	1.542	1.36	1.36	1.14	1.14	172.0	196.1	189.2	215.7

Level: 2nd Floor

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
53	CoreBRB_3.25	BOLTED	126.8	139.8	1.35	1.22	436	0.814
54	CoreBRB_3.25	BOLTED	126.8	139.8	1.35	1.22	436	0.818
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax

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	2Δbm	2%	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)	(kips)	(kips)
53	2.758	2.604	1.243	1.173	1.28	1.28	1.11	1.11	178.9	198.6	196.8	218.4
54	2.758	2.604	1.243	1.173	1.28	1.28	1.11	1.11	178.9	198.6	196.8	218.4

Frame #15:

Level: Roof

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	K _{eff} (kips/in)	Core max DCR
10	CoreBRB_3.00	BOLTED	117.0	129.0	1.35	1.31	458	0.612
11	CoreBRB_3.00	BOLTED	117.0	129.0	1.35	1.31	458	0.625

Brace #	Elong (in)	ε (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm	2%	2Δbm	2%	(kips)	(kips)	(kips)	(kips)
10	1.018	2.455	0.574	1.386	1.30	1.30	1.11	1.11
11	1.018	2.455	0.574	1.386	1.31	1.31	1.11	1.11

Level: 2nd Floor

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	K _{eff} (kips/in)	Core max DCR
55	CoreBRB_4.25	BOLTED	165.8	182.8	1.35	1.25	571	0.882
56	CoreBRB_4.25	BOLTED	165.8	182.8	1.35	1.25	571	0.885

Brace #	Elong (in)	ε (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm	2%	2Δbm	2%	(kips)	(kips)	(kips)	(kips)
55	2.703	2.604	1.247	1.201	1.28	1.28	1.11	1.11
					233.9	233.9	259.7	259.7
56	2.703	2.604	1.247	1.201	1.28	1.28	1.11	1.11
					233.9	233.9	259.7	259.7

Frame #16:

Level: Roof

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	K _{eff} (kips/in)	Core max DCR
12	CoreBRB 1.25	BOLTED	48.8	53.8	1.65	1.42	233	0.878

CoreBrace Summary



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13	CoreBRB_1.25	BOLTED	48.8	53.8	1.65	1.42	233	0.886
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)
12	2Δbm 2%	2Δbm 2%	2Δbm 2%	2Δbm 2%	76.3	102.3	84.0	112.5
13	1.146 2.455	0.701 1.501	1.42 1.42	1.34 1.34	75.8	101.6	83.4	111.7
	1.146 2.455	0.701 1.501	1.41 1.41	1.34 1.34				

Level: 2nd Floor

Brace #	Size	Type	PySc Min (kips)	PySc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
57	CoreBRB_4.25	BOLTED	165.8	182.8	1.35	1.25	571	0.844
58	CoreBRB_4.25	BOLTED	165.8	182.8	1.35	1.25	571	0.846
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)
57	2Δbm 2%	2Δbm 2%	2Δbm 2%	2Δbm 2%	233.9	259.7	257.3	285.6
58	2.704 2.604	1.248 1.201	1.28 1.28	1.11 1.11	233.9	259.7	257.3	285.6
	2.702 2.604	1.247 1.201	1.28 1.28	1.11 1.11				

Frame #17:

Level: Roof

Brace #	Size	Type	PySc Min (kips)	PySc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
16	CoreBRB_1.00	BOLTED	39.0	43.0	1.60	1.45	181	0.800
17	CoreBRB_1.00	BOLTED	39.0	43.0	1.60	1.45	181	0.795

Brace #	Elong (in)	ϵ (%)	ω	β	Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)
16	2Δbm 2%	2Δbm 2%	2Δbm 2%	2Δbm 2%	59.8	78.3	65.7	86.1
17	1.038 2.455	0.638 1.507	1.39 1.39	1.31 1.31	59.8	78.3	65.7	86.1
	1.038 2.455	0.638 1.507	1.39 1.39	1.31 1.31				

Level: 2nd Floor

Brace #	Size	Type	PySc Min (kips)	PySc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
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59	CoreBRB_3.75	BOLTED	146.3	161.3	1.35	1.22	504	0.985
60	CoreBRB_3.75	BOLTED	146.3	161.3	1.35	1.22	504	0.987
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm 2%	2Δbm	2Δbm 2%	2Δbm 2%	(kips)	(kips)	(kips)	(kips)
59	2.722	2.604	1.235	1.181	1.28	1.28	1.11	1.11
60	2.722	2.604	1.235	1.181	1.28	1.28	1.11	1.11

Frame #18:

Level: Roof

Brace #	Size	Type	Pyse Min (kips)	Pyse Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
26	CoreBRB_2.00	BOLTED	76.0	92.0	1.35	1.32	305	0.714
27	CoreBRB_2.00	BOLTED	76.0	92.0	1.35	1.32	305	0.673

Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm 2%	2Δbm	2Δbm 2%	2Δbm 2%	(kips)	(kips)	(kips)	(kips)
26	1.361	2.455	0.761	1.373	1.35	1.35	1.13	1.13
27	1.361	2.455	0.761	1.373	1.35	1.35	1.13	1.13

Level: 2nd Floor

Brace #	Size	Type	Pyse Min (kips)	Pyse Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
73	CoreBRB_2.75	BOLTED	107.3	118.3	1.45	1.25	397	0.720
74	CoreBRB_2.75	BOLTED	107.3	118.3	1.40	1.25	383	0.718

Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm 2%	2Δbm	2Δbm 2%	2Δbm 2%	(kips)	(kips)	(kips)	(kips)
73	2.275	2.604	1.092	1.249	1.31	1.31	1.11	1.11
74	2.353	2.604	1.129	1.249	1.31	1.31	1.11	1.11

Frame #19:

CoreBrace Summary



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Level: 2nd Floor

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
75	CoreBRB_2.50	BOLTED	97.5	107.5	1.45	1.30	361	0.783
76	CoreBRB_2.50	BOLTED	97.5	107.5	1.45	1.30	361	0.782
Brace #	Elong (in)	ϵ (%)	ω	β			Tmax	Cmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)
75	2.265	2.604	1.137	1.307	1.12	1.12	141.9	158.9
76	2.265	2.604	1.137	1.307	1.12	1.12	141.9	158.9

Frame #20:

Level: Roof

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
40	CoreBRB_4.50	BOLTED	175.5	193.5	1.30	1.30	478	0.551
Brace #	Elong (in)	ϵ (%)	ω	β			Tmax	Cmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)
40	1.713	3.380	0.676	1.334	1.30	1.30	1.11	1.11

Frame #21:

Level: 2nd Floor

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
71	CoreBRB_2.75	BOLTED	107.3	118.3	1.40	1.24	354	0.940
72	CoreBRB_2.75	BOLTED	107.3	118.3	1.40	1.24	354	0.942
Brace #	Elong (in)	ϵ (%)	ω	β			Tmax	Cmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)
71	2.734	3.070	1.197	1.343	1.12	1.12	157.3	176.1
72	2.734	3.070	1.197	1.343	1.12	1.12	157.3	176.1

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Frame #22:

Level: Roof

Brace #	Size	Type	Pysec Min (kips)	Pysec Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
31	CoreBRB_1.00	BOLTED	39.0	43.0	1.55	1.44	159	0.856
32	CoreBRB_1.00	BOLTED	39.0	43.0	1.55	1.44	159	0.861
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm 2%	2Δbm 2%	2Δbm 2%	2Δbm 2%	(kips)	(kips)	(kips)	(kips)
31	1.517	2.838	0.834	1.560	1.39	1.39	1.32	1.32
					59.8	59.8	65.7	65.7
32	1.517	2.838	0.834	1.560	1.39	1.39	1.32	1.32
					59.8	59.8	65.7	65.7

Level: 2nd Floor

Brace #	Size	Type	Pysec Min (kips)	Pysec Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
83	CoreBRB_3.00	BOLTED	117.0	129.0	1.35	1.21	373	0.853
84	CoreBRB_3.00	BOLTED	117.0	129.0	1.35	1.21	373	0.852
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm 2%	2Δbm 2%	2Δbm 2%	2Δbm 2%	(kips)	(kips)	(kips)	(kips)
83	2.656	3.070	1.094	1.265	1.30	1.30	1.11	1.11
					167.7	167.7	184.5	184.5
84	2.656	3.070	1.094	1.265	1.30	1.30	1.11	1.11
					167.7	167.7	184.5	184.5

Frame #23:

Level: Roof

Brace #	Size	Type	Pysec Min (kips)	Pysec Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
29	CoreBRB_1.75	BOLTED	68.3	75.3	1.45	1.38	260	0.759
30	CoreBRB_1.75	BOLTED	68.3	75.3	1.45	1.38	260	0.769
63	CoreBRB_2.00	BOLTED	76.0	92.0	1.35	1.34	277	0.710
64	CoreBRB_2.00	BOLTED	76.0	92.0	1.35	1.34	277	0.753

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Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm 2%	2Δbm 2%	2Δbm 2%	2Δbm 2%	(kips)	(kips)	(kips)	(kips)
29	1.617	2.838	0.892	1.564	1.39	1.39	1.32	1.32
					1.39	1.39	1.32	1.32
30	1.617	2.838	0.892	1.564	1.39	1.39	1.32	1.32
					1.39	1.39	1.32	1.32
63	1.598	2.838	0.817	1.450	1.35	1.35	1.13	1.13
					1.35	1.35	1.13	1.13
64	1.598	2.838	0.817	1.450	1.35	1.35	1.13	1.13
					1.35	1.35	1.13	1.13

Level: 2nd Floor

Brace #	Size	Type	Pysc Min	Pysc Max	Stiff Mult	Stiff Mult	Keff	Core max
			(kips)	(kips)	Used	Calc	(kips/in)	DCR
77	CoreBRB_6.00	BOLTED	228.0	276.0	1.40	1.36	773	0.863
78	CoreBRB_6.00	BOLTED	228.0	276.0	1.40	1.36	773	0.866
97	CoreBRB_6.00	BOLTED	228.0	276.0	1.40	1.36	773	0.864
98	CoreBRB_6.00	BOLTED	228.0	276.0	1.40	1.36	773	0.866

Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm 2%	2Δbm 2%	2Δbm 2%	2Δbm 2%	(kips)	(kips)	(kips)	(kips)
77	2.626	3.070	1.241	1.451	1.27	1.27	1.18	1.18
					1.27	1.27	1.18	1.18
78	2.634	3.070	1.245	1.451	1.27	1.27	1.18	1.18
					1.27	1.27	1.18	1.18
97	2.626	3.070	1.241	1.451	1.27	1.27	1.18	1.18
					1.27	1.27	1.18	1.18
98	2.634	3.070	1.245	1.451	1.27	1.27	1.18	1.18
					1.27	1.27	1.18	1.18

Frame #24:

Level: Roof

Brace #	Size	Type	Pysc Min	Pysc Max	Stiff Mult	Stiff Mult	Keff	Core max
			(kips)	(kips)	Used	Calc	(kips/in)	DCR
20	CoreBRB_1.00	BOLTED	39.0	43.0	1.60	1.44	164	0.951
21	CoreBRB_1.00	BOLTED	39.0	43.0	1.55	1.44	159	0.954
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm 2%	2Δbm 2%	2Δbm 2%	2Δbm 2%	(kips)	(kips)	(kips)	(kips)

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20	1.640	2.838	0.902	1.560	1.40	1.40	1.32	1.32	60.2	79.5	66.2	87.4
21	1.687	2.838	0.928	1.560	1.39	1.39	1.31	1.31	59.8	78.3	65.7	86.1

Level: 2nd Floor

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
65	CoreBRB_3.00	BOLTED	117.0	129.0	1.35	1.21	373	0.904
66	CoreBRB_3.00	BOLTED	117.0	129.0	1.35	1.21	373	0.908
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)
65	2.763	3.070	1.138	1.265	1.30	1.30	1.11	1.11
					167.7	186.1	184.5	204.8
66	2.763	3.070	1.138	1.265	1.30	1.30	1.11	1.11
					167.7	186.1	184.5	204.8

Frame #25:

Level: Roof

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
77	CoreBRB_3.00	BOLTED	114.0	138.0	1.30	1.33	400	0.676
78	CoreBRB_3.00	BOLTED	114.0	138.0	1.30	1.33	400	0.705
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)
77	1.625	2.838	0.838	1.463	1.17	1.17	172.5	201.8
					172.5	201.8	189.8	222.0
78	1.625	2.838	0.838	1.463	1.17	1.17	172.5	201.8
					172.5	201.8	189.8	222.0

Level: 2nd Floor

Brace #	Size	Type	Pyssc Min (kips)	Pyssc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
69	CoreBRB_6.00	BOLTED	228.0	276.0	1.40	1.36	773	0.880
70	CoreBRB_6.00	BOLTED	228.0	276.0	1.40	1.36	773	0.883
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax

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69	2Δbm	2%	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)	(kips)
70	2.676	3.070	1.265	1.451	1.27	1.27	1.18	1.18	350.5	413.6	385.6
	2.677	3.070	1.265	1.451	1.27	1.27	1.18	1.18	350.5	413.6	385.6
											455.0

Frame #26:

Level: Roof

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
18	CoreBRB_1.00	BOLTED	39.0	43.0	1.55	1.44	159	0.921
19	CoreBRB_1.00	BOLTED	39.0	43.0	1.55	1.44	159	0.935
Brace #	Elong (in)	ε (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)
18	1.636	2.838	0.900	1.560	1.39	1.32	59.8	78.9
19	1.636	2.838	0.900	1.560	1.39	1.32	59.8	78.9
							65.7	86.8

Level: 2nd Floor

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
63	CoreBRB_3.00	BOLTED	117.0	129.0	1.35	1.21	373	0.904
64	CoreBRB_3.00	BOLTED	117.0	129.0	1.35	1.21	373	0.908
Brace #	Elong (in)	ε (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)
63	2.763	3.070	1.138	1.265	1.30	1.30	167.7	186.1
64	2.763	3.070	1.138	1.265	1.30	1.30	167.7	186.1
							184.5	204.8

Frame #27:

Level: Roof

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
22	CoreBRB_4.00	BOLTED	156.0	172.0	1.30	1.42	533	0.674

CoreBrace Summary



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23	CoreBRB_4.00	BOLTED	156.0	172.0	1.30	1.42	533	0.683				
Brace #	Elong (in)	ϵ (%)	ω	β		Tmax	Cmax	1.1Tmax	1.1Cmax			
	2Δbm	2%	2Δbm	2%	2Δbm	2%	(kips)	(kips)	(kips)			
22	1.546	2.838	0.864	1.586	1.32	1.32	1.12	1.12	227.0	254.3	249.7	279.7
23	1.546	2.838	0.864	1.586	1.32	1.32	1.11	1.11	227.0	252.0	249.7	277.2

Level: 2nd Floor

Brace #	Size	Type	Pysc Min (kips)	Pysc Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR				
67	CoreBRB_6.00	BOLTED	228.0	276.0	1.40	1.36	773	0.883				
68	CoreBRB_6.00	BOLTED	228.0	276.0	1.40	1.36	773	0.885				
Brace #	Elong (in)	ϵ (%)	ω	β		Tmax (kips)	Cmax (kips)	1.1Tmax (kips)	1.1Cmax (kips)			
	2Δbm	2%	2Δbm	2%	2Δbm	2%						
67	2.676	3.070	1.265	1.451	1.27	1.27	1.18	1.18	350.5	413.6	385.6	455.0
68	2.676	3.070	1.265	1.451	1.27	1.27	1.18	1.18	350.5	413.6	385.6	455.0

Frame #28:

Level: Roof

Brace #	Size	Type	Pyse Min (kips)	Pyse Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
81	CoreBRB_2.00	BOLTED	76.0	92.0	1.35	1.34	277	0.690
82	CoreBRB_2.00	BOLTED	76.0	92.0	1.35	1.34	277	0.701

Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Δbm	2%	2Δbm	2%	(kips)	(kips)	(kips)	(kips)
81	1.531	2.838	0.783	1.451	1.35	1.35	1.13	1.13
82	1.531	2.838	0.783	1.451	1.35	1.35	1.13	1.13

Level: 2nd Floor

Brace #	Size	Type	Pyse Min (kips)	Pyse Max (kips)	Stiff Mult Used	Stiff Mult Calc	Keff (kips/in)	Core max DCR
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CoreBrace Summary



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79	CoreBRB_6.00	BOLTED	228.0	276.0	1.40	1.36	773	0.867
80	CoreBRB_6.00	BOLTED	228.0	276.0	1.40	1.36	773	0.865
Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Abm 2%	2Abm	2Abm 2%	2Abm 2%	(kips)	(kips)	(kips)	(kips)
79	2.630	3.070	1.243	1.451	1.27	1.27	1.18	1.18
80	2.630	3.070	1.243	1.451	1.27	1.27	1.18	1.18
					350.5	350.5	413.6	413.6
					385.6	385.6	455.0	455.0

Frame #29:

Level: Roof

Brace #	Size	Type	Pyse Min	Pyse Max	Stiff Mult	Stiff Mult	Keff	Core max
			(kips)	(kips)	Used	Calc	(kips/in)	DCR
33	CoreBRB_1.75	BOLTED	68.3	75.3	1.50	1.37	269	0.721
34	CoreBRB_1.75	BOLTED	68.3	75.3	1.50	1.37	269	0.698

Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Abm 2%	2Abm	2Abm 2%	2Abm 2%	(kips)	(kips)	(kips)	(kips)
33	1.452	2.838	0.791	1.546	1.39	1.39	1.31	1.31
34	1.389	2.838	0.757	1.546	1.39	1.39	1.32	1.32
					104.6	104.6	138.1	138.1
					115.1	115.1	151.9	151.9

Level: 2nd Floor

Brace #	Size	Type	Pyse Min	Pyse Max	Stiff Mult	Stiff Mult	Keff	Core max
			(kips)	(kips)	Used	Calc	(kips/in)	DCR
81	CoreBRB_3.75	BOLTED	146.3	161.3	1.40	1.22	483	0.862
82	CoreBRB_3.75	BOLTED	146.3	161.3	1.35	1.22	466	0.845

Brace #	Elong (in)	ϵ (%)	ω	β	Tmax	Cmax	1.1Tmax	1.1Cmax
	2Abm 2%	2Abm	2Abm 2%	2Abm 2%	(kips)	(kips)	(kips)	(kips)
81	2.648	3.070	1.101	1.277	1.30	1.30	1.11	1.11
82	2.709	3.070	1.126	1.277	1.30	1.30	1.11	1.11
					209.6	209.6	232.7	232.7
					230.6	230.6	256.0	256.0

BOLTED: Bolted Connection Brace

CoreBrace Summary



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WELDED: Welded Connection Brace
PINNED: Pinned Connection Brace
SPLICE PLATE: Splice Plate Connection Brace
CUSTOM: Custom Connection Brace

Verify brace properties with the brace manufacturer prior to finalizing structural design.



QUANTUM | CONSULTING ENGINEERS

**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

1015 39th AVENUE SE
PUYALLUP, WA 98374

QUANTUM JOB NUMBER: 19305.04

BRACED FRAMES

CHEVRON BEAM DUCTILITY AND STRENGTH

Chevron Beam AISC 341 Ductility Checks

Roof		EQ D1-3				Beam		BRB Loads				
Frame #	Grids	Beam Size	Span (ft)	ry	Lb (in)	Lu Limit (in)		BRB SIZE	% Axial Force	Vert Load (kips)	Horiz. Load (kips)	Collector Load (kips)
1	2/D-E	W24X68	25	1.87	100	191	OK	2	100%	NA	95	123.4
2	2/F-H											
3	6/D-E	W21x44	25	1.26	75	129	OK	2.25	75%	12	150.8	158.2
4	6/G-H	W21x44	25	1.26	75	129	OK	2.25	75%	12	150.8	165.5
5	N/A											
6A	8/G-H	W21x44	25	1.26	75	129	OK	2.75	75%	15.3	195	142.5
6B	8/H-J	W21x44	25	1.26	75	129	OK	2	75%	9.2	129.6	142.5
7	9/B-C											
8	10/C-D											
9	10/G-H	W21x44	25	1.26	75	129	OK	2	75%	10	138	115.9
10	11/D-E	W27X94	25	2.12	150	212	OK	2.75	75%	15	185.4	163.3
11	13/C-D	W21x44	25	1.26	75	129	OK	2.5	75%	14.9	178	156.3
12	13/E-F											
13	13/H-J	W21x44	25	1.26	75	129	OK	2.25	75%	12.6	160.8	140.6
14	15/H-J	W27X94	25	2.12	150	212	OK	2.75	75%	15	185.4	173.7
15	17/C-D	W21x44	25	1.26	75	129	OK	3	50%	16.1	207.8	163.7
16A	17/G-H											
16B	17/H-J	W21x44	25	1.26	75	129	OK	1.25	75%	21.5	104.2	95.7
17	18/D-F	W21x44	25	1.26	75	129	OK	1	75%	15	80.8	67.35
18	D/3-4	W21x44	25	1.26	100	129	OK	2	75%	31	162.2	125.88
19	H/3-4											
20	H/5-6	W24X68	25	1.87	100	191	OK	4.5	100%	NA	218	170.9
21	D/7-8											
22	K/7-8	W21x44	32	1.26	96	129	OK	1	75%	14.1	94.1	90.2
23A	H/9-10	W21x44	32	1.26	128	129	OK	1.75	75%	24.6	164.6	151.0
23B	H/10-11	W21X44	32	1.26	128	129	OK	2	75%	9	153	151.0
24	A/10-11	W21x44	32	1.26	128	129	OK	1	75%	14.5	94.4	101.0
25	D/10-11	W21x62	32	1.77	128	177	OK	3	75%	13.7	230.2	225.7
26	A/15-16	W21x44	32	1.26	128	129	OK	1	75%	14.1	94.1	98.1
27	D/15-16	W21x44	32	1.26	96	129	OK	4	50%	20	326.5	288.0
28	H/15-16	W21x62	32	1.77	128	177	OK	2	75%	9	153	147.9
29	K/15-16	W21x44	32	1.26	128	129	OK	1.75	75%	NA	150.9	137.6

Chevron Beam AISC 341 Ductility Checks

Roof		Beam Properties					Py = Ry Ag Fy						
Frame					Area								
#	Grids	d (in)	k_des (in)	tw (in)	(sqin)	Fy (ksi)	Ry	Py (k)	Ω	Ca	h/tw	h/tw (max)	
1	2/D-E	23.7	1.09	0.415	20.1	36	1.5	1085.4	1.67	0.19	51.86	57.70	OK
2	2/F-H												
3	6/D-E	20.7	0.95	0.35	13	36	1.5	702	1.67	0.28	53.71	54.94	OK
4	6/G-H	20.7	0.95	0.35	13	36	1.5	702	1.67	0.30	53.71	54.55	OK
5	N/A												
6A	8/G-H	20.7	0.95	0.35	13	36	1.5	702	1.67	0.35	53.71	52.98	NG
6B	8/H-J	20.7	0.95	0.35	13	36	1.5	702	1.67	0.25	53.71	55.78	OK
7	9/B-C												
8	10/C-D												
9	10/G-H	20.7	0.95	0.35	13	36	1.5	702	1.67	0.25	53.71	56.02	OK
10	11/D-E	26.9	1.34	0.49	27.6	50	1.1	1518	1.67	0.15	49.43	58.27	OK
11	13/C-D	20.7	0.95	0.35	13	36	1.5	702	1.67	0.32	53.71	53.88	OK
12	13/E-F												
13	13/H-J	20.7	0.95	0.35	13	36	1.5	702	1.67	0.29	53.71	54.80	OK
14	15/H-J	26.9	1.34	0.49	27.6	50	1.1	1518	1.67	0.15	49.43	58.27	OK
15	17/C-D	20.7	0.95	0.35	13	36	1.5	702	1.67	0.25	53.71	55.99	OK
16A	17/G-H												
16B	17/H-J	20.7	0.95	0.35	13	36	1.5	702	1.67	0.19	53.71	57.82	OK
17	18/D-F	20.7	0.95	0.35	13	36	1.5	702	1.67	0.14	53.71	59.07	OK
18	D/3-4	20.7	0.95	0.35	13	36	1.5	702	1.67	0.29	53.71	54.73	OK
19	H/3-4												
20	H/5-6	23.7	1.09	0.415	20.1	36	1.5	1085.4	1.67	0.34	51.86	53.35	OK
21	D/7-8												
22	K/7-8	20.7	0.95	0.35	13	36	1.5	702	1.67	0.17	53.71	58.36	OK
23A	H/9-10	20.7	0.95	0.35	13	36	1.5	702	1.67	0.29	53.71	54.60	OK
23B	H/10-11	20.7	0.95	0.35	13	36	1.5	702	1.67	0.27	53.71	55.22	OK
24	A/10-11	20.7	0.95	0.35	13	36	1.5	702	1.67	0.18	53.71	57.99	OK
25	D/10-11	21	1.12	0.4	18.3	50	1.1	1006.5	1.67	0.29	46.90	54.31	OK
26	A/15-16	20.7	0.95	0.35	13	36	1.5	702	1.67	0.17	53.71	58.15	OK
27	D/15-16	20.7	0.95	0.35	13	36	1.5	702	1.67	0.39	53.71	51.77	NG
28	H/15-16	21	1.12	0.4	18.3	50	1.1	1006.5	1.67	0.19	46.90	57.16	OK
29	K/15-16	20.7	0.95	0.35	13	36	1.5	702	1.67	0.27	53.71	55.33	OK

Chevron Beam AISC 341 Ductility Checks

Roof									
Frame									
#	Grids	bf (in)	tf (in)	b/t	b/t (max)			Notes	rev h/tw
1	2/D-E	8.97	0.585	7.67	9.27	OK	Mod. Ductile		
2	2/F-H								
3	6/D-E	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
4	6/G-H	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
5	N/A								
6A	8/G-H	6.5	0.45	7.22	9.27	OK	Not Mod. Ductile	Retrofit	38 57.468 OK
6B	8/H-J	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
7	9/B-C								
8	10/C-D								
9	10/G-H	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
10	11/D-E	10	0.745	6.71	9.18	OK	Mod. Ductile		
11	13/C-D	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
12	13/E-F								
13	13/H-J	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
14	15/H-J	10	0.745	6.71	9.18	OK	Mod. Ductile		
15	17/C-D	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
16A	17/G-H								
16B	17/H-J	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
17	18/D-F	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
18	D/3-4	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
19	H/3-4								
20	H/5-6	8.97	0.585	7.67	9.27	OK	Mod. Ductile		
21	D/7-8								
22	K/7-8	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
23A	H/9-10	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
23B	H/10-11	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
24	A/10-11	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
25	D/10-11	8.24	0.615	6.70	9.18	OK	Mod. Ductile		
26	A/15-16	6.5	0.45	7.22	9.27	OK	Mod. Ductile		
27	D/15-16	6.5	0.45	7.22	9.27	OK	Not Mod. Ductile	Retrofit	38 55.413 OK
28	H/15-16	8.24	0.615	6.70	9.18	OK	Mod. Ductile		
29	K/15-16	6.5	0.45	7.22	9.27	OK	Mod. Ductile		

Chevron Beam AISC 341 Ductility Checks

Level 2		EQ D1-3								BRB Loads			Ratio'd Force 2- story frames
Frame #	Grids	Beam Size	Span (ft)	ry	Lb (in)	Lu Limit (in)		BRB SIZE	% Axial Force	Vert Load (kips) NA	Horiz. Load (kips)	Collector Load (kips)	
2	2/F-H	W24x55	25	1.34	75	134	OK	7.5	75%	35.1	423.7	587.23	317.8
3	6/D-E	W21x44	25	1.26	100	126	OK	3.75	75%	22.7	227.7	282.78	133.1
4	6/G-H	W21x44	25	1.26	100	126	OK	3.25	50%	17	195.7	242.80	97.9
5	8/D-E	W18x35	N/A	1.22									
6A	8/G-H	W18x35	25	1.22	100	122	OK	3	75%	11.5	162.6	193.55	73.2
6B	8/H-J	W18x35	25	1.22	100	122	OK	3	75%	15.6	179.3	193.55	102.1
7	9/B-C	W18x35	25	1.22	100	122	OK	2.75	50%	14.5	166.9	208.70	83.5
8	10/C-D	W18x35	25	1.22	100	122	OK	2.75	50%	14.6	168.2	207.91	84.1
9	10/G-H	W18x35	25	1.22	100	122	OK	2.75	50%	11.6	152.1	165.23	76.1
10	11/D-E	W18x35	25	1.22	100	122	OK	2.75	50%	14.6	168.2	176.43	84.1
11	13/C-D	W18x35	25	1.22	100	122	OK	2	60%	7.6	108.2	130.55	47.1
12	13/E-F	W18x35	25	1.22	100	122	OK	2.75	60%	15.6	167.6	206.30	100.6
13	13/H-J	W18x35	25	1.22	100	122	OK	2	50%	11.6	152.1	130.55	76.1
14	15/H-J	W18x35	25	1.22	100	122	OK	3.25	50%	16.9	194.2	231.18	97.1
15	17/C-D	W18x35	25	1.22	100	122	OK	4.25	50%	22.1	254	317.68	127.0
16A	17/G-H	W18x35	25	1.22	75	122	OK	4.25	75%	22.1	254	239.40	190.5
16B	17/H-J												
17	18/D-F	W18x35	25	1.22	100	122	OK	3.75	50%	19.5	224.1	289.78	112.1
18	D/3-4	W18x35	25	1.22	100	122	OK	2.75	50%	14.6	168.2	159.55	84.1
19	H/3-4	W14x30	25	1.49	75	149	OK	2.5	75%	14.6	154.8	157.79	116.1
20	H/5-6												
21	D/7-8	W24x55	32	1.34	96	134	OK	2.75	50%	15	203.1	221.94	101.6
22	K/7-8	W24x55	32	1.34	128	134	OK	3	50%	14.6	215.5	423.35	107.8
23A	H/9-10	W27x84	32	2.07	128	207	OK	6	75%	29.3	412.3	423.35	268.1
23B	H/10-11	W27x84	32	2.07	128	207	OK	6	75%	32.4	439.8	228.83	291.6
24	A/10-11	W24x55	32	1.34	128	134	OK	3	50%	14.6	215.5	228.83	107.8
25	D/10-11	W27x84	32	2.07	128	207	OK	6	50%	32.4	439.8	433.20	219.9
26	A/15-16	W24x55	32	1.34	128	134	OK	3	50%	14.6	215.5	228.83	107.8
27	D/15-16	W27x84	32	2.07	128	207	OK	6	50%	32.4	439.8	433.20	219.9
28	H/15-16	W27x84	32	2.07	128	207	OK	6	50%	32.4	439.8	433.20	219.9
29	K/15-16	W24x55	32	1.34	128	134	OK	3.75	75%	6.3	105.5	278.23	41.4

Chevron Beam AISC 341 Ductility Checks

Level 2		Beam Properties					Py = Ry Ag Fy						
Frame #	Grids	d (in)	k_des (in)	tw (in)	Area (sqin)	Fy (ksi)	Ry	Py (k)	Ω	Ca	h/tw	h/tw (max)	
2	2/F-H	23.6	1.01	0.395	16.2	50	1.1	891	1.67	0.83	54.63	38.35	NG
3	6/D-E	20.7	0.95	0.35	13	50	1.1	715	1.67	0.50	53.71	48.12	NG
4	6/G-H	20.7	0.95	0.35	13	50	1.1	715	1.67	0.28	53.71	54.40	OK
5	8/D-E	17.7	0.827	0.3	10.3								
6A	8/G-H	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.43	53.49	50.12	NG
6B	8/H-J	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.43	53.49	50.12	NG
7	9/B-C	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.31	53.49	53.69	OK
8	10/C-D	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.31	53.49	53.72	OK
9	10/G-H	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.24	53.49	55.58	OK
10	11/D-E	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.26	53.49	55.09	OK
11	13/C-D	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.23	53.49	55.96	OK
12	13/E-F	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.36	53.49	51.99	NG
13	13/H-J	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.22	53.49	56.16	OK
14	15/H-J	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.34	53.49	52.70	NG
15	17/C-D	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.47	53.49	48.93	NG
16A	17/G-H	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.56	53.49	46.16	NG
16B	17/H-J												
17	18/D-F	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.43	53.49	50.15	NG
18	D/3-4	17.7	0.827	0.3	10.3	50	1.1	566.5	1.67	0.25	53.49	55.45	OK
19	H/3-4	13.8	0.785	0.27	8.85	50	1.1	486.75	1.67	0.41	45.30	50.77	OK
20	H/5-6												
21	D/7-8	23.6	1.01	0.395	16.2	50	1.1	891	1.67	0.21	54.63	56.64	OK
22	K/7-8	23.6	1.01	0.395	16.2	50	1.1	891	1.67	0.40	54.63	51.05	NG
23A	H/9-10	26.7	1.24	0.46	24.7	50	1.1	1359	1.67	0.39	52.65	51.24	NG
23B	H/10-11	26.7	1.24	0.46	24.7	50	1.1	1359	1.67	0.36	52.65	52.18	NG
24	A/10-11	23.6	1.01	0.395	16.2	50	1.1	891	1.67	0.21	54.63	56.45	OK
25	D/10-11	26.7	1.24	0.46	24.7	50	1.1	1359	1.67	0.27	52.65	54.79	OK
26	A/15-16	23.6	1.01	0.395	16.2	50	1.1	891	1.67	0.21	54.63	56.45	OK
27	D/15-16	26.7	1.24	0.46	24.7	50	1.1	1359	1.67	0.27	52.65	54.79	OK
28	H/15-16	26.7	1.24	0.46	24.7	50	1.1	1359	1.67	0.27	52.65	54.79	OK
29	K/15-16	23.6	1.01	0.395	16.2	50	1.1	891	1.67	0.39	54.63	51.21	NG

Chevron Beam AISC 341 Ductility Checks

Level 2

Frame #	Grids	bf (in)	tf (in)	b/t	b/t (max)			Notes	rev h/tw	rev h/tw (max)		
2	2/F-H	7.01	0.505	6.94	9.18	OK	Not Mod. Ductile	Retrofit	40.7	42.81	OK	
3	6/D-E	6.5	0.45	7.22	9.18	OK	Not Mod. Ductile	Retrofit	38.0	51.389	OK	
4	6/G-H	6.5	0.45	7.22	9.18	OK	Mod. Ductile					
5	8/D-E	6	0.425									
6A	8/G-H	6	0.425	7.06	9.18	OK	Not Mod. Ductile	Retrofit	34.8	53.535	OK	
6B	8/H-J	6	0.425	7.06	9.18	OK	Not Mod. Ductile	Retrofit	34.8	53.535	OK	
7	9/B-C	6	0.425	7.06	9.18	OK	Mod. Ductile					
8	10/C-D	6	0.425	7.06	9.18	OK	Mod. Ductile					
9	10/G-H	6	0.425	7.06	9.18	OK	Mod. Ductile					
10	11/D-E	6	0.425	7.06	9.18	OK	Mod. Ductile					
11	13/C-D	6	0.425	7.06	9.18	OK	Mod. Ductile					
12	13/E-F	6	0.425	7.06	9.18	OK	Not Mod. Ductile	Retrofit	34.8	54.899	OK	
13	13/H-J	6	0.425	7.06	9.18	OK	Mod. Ductile					
14	15/H-J	6	0.425	7.06	9.18	OK	Not Mod. Ductile	Retrofit	34.8	55.422	OK	
15	17/C-D	6	0.425	7.06	9.18	OK	Not Mod. Ductile	Retrofit	34.8	52.662	OK	
16A	17/G-H	6	0.425	7.06	9.18	OK	Not Mod. Ductile	Retrofit	34.8	50.642	OK	
16B	17/H-J											
17	18/D-F	6	0.425	7.06	9.18	OK	Not Mod. Ductile	Retrofit	34.8	53.552	OK	
18	D/3-4	6	0.425	7.06	9.18	OK	Mod. Ductile					
19	H/3-4	6.73	0.385	8.74	9.18	OK	Mod. Ductile					
20	H/5-6											
21	D/7-8	7.01	0.505	6.94	9.18	OK	Mod. Ductile					
22	K/7-8	7.01	0.505	6.94	9.18	OK	Not Mod. Ductile	Retrofit	40.7	53.191	OK	
23A	H/9-10	10	0.64	7.81	9.18	OK	Not Mod. Ductile	Retrofit	40.9	52.754	OK	
23B	H/10-11	10	0.64	7.81	9.18	OK	Not Mod. Ductile	Retrofit	40.9	53.573	OK	
24	A/10-11	7.01	0.505	6.94	9.18	OK	Mod. Ductile					
25	D/10-11	10	0.64	7.81	9.18	OK	Mod. Ductile					
26	A/15-16	7.01	0.505	6.94	9.18	OK	Mod. Ductile					
27	D/15-16	10	0.64	7.81	9.18	OK	Mod. Ductile					
28	H/15-16	10	0.64	7.81	9.18	OK	Mod. Ductile					
29	K/15-16	7.01	0.505	6.94	9.18	OK	Not Mod. Ductile	Retrofit	40.722	53.327	OK	

Seattle, WA 98101

F. 206.957.3901

ProjectSouth Hill Bldg C

Job #14079.01

Pageof

ClientBenereya

ByAMJ

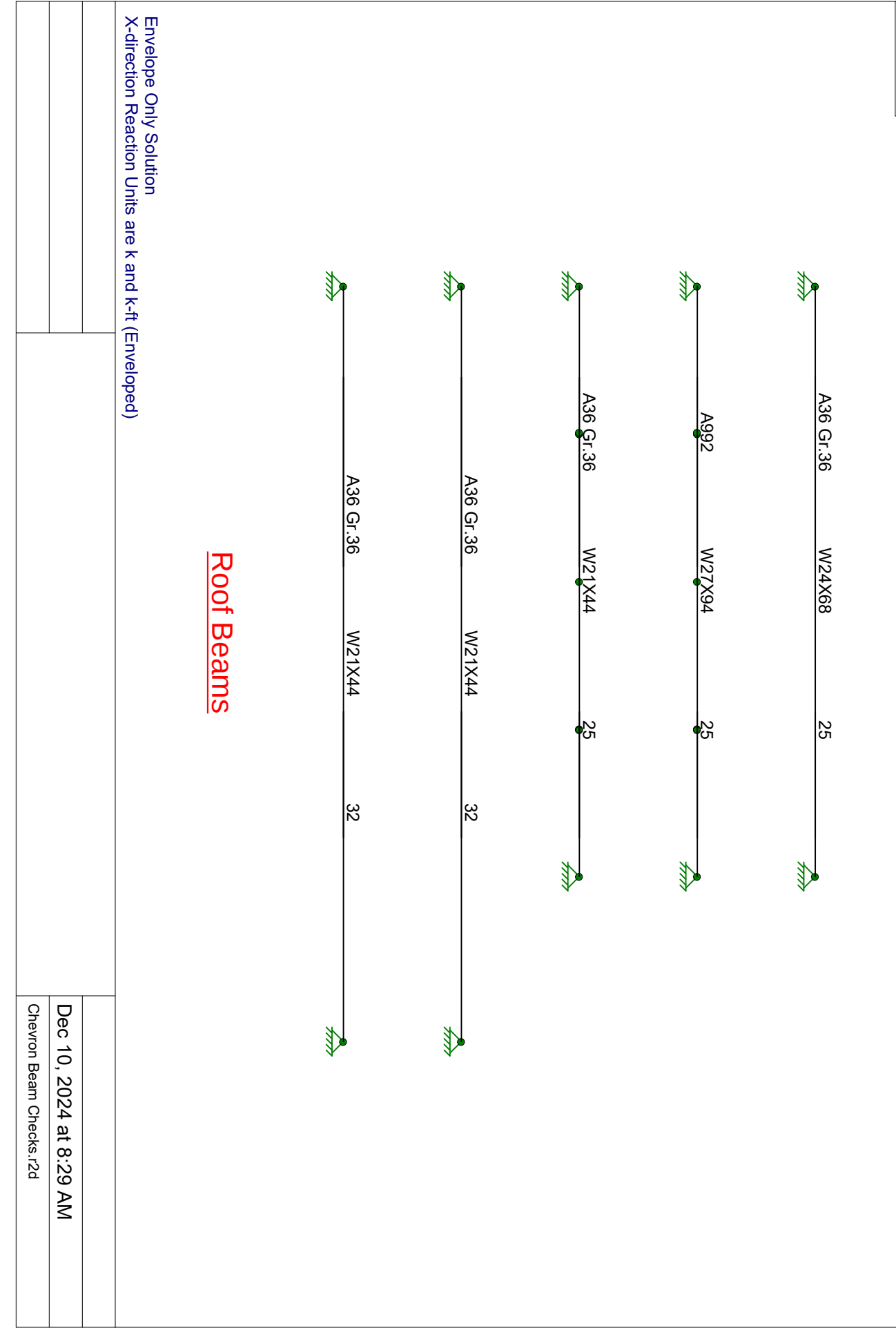
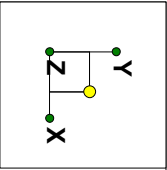
Date12/10/24

Chevron Beam Strength Design

Level		Roof	(@ beam midspan)												ASD	
Frame	Beam	Fy	Area	r _y	L	L _b	K/r	F _{cr} /Ω	P _n /Ω	0.75E	P _r /P _c	M _a	M _n /Ω	Unity Eqn		
1	W24X68	36	20.1	1.87	25	8.3333	53.5	18.5	371.85	86.40	0.23	20.1	318.0	0.29		
2																
3	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	83.07	0.36	84.9	171.4	0.80		
4	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	86.87	0.38	84.9	171.4	0.82		
6A	W21x44R	36	16.72	1.421	25	6.25	52.8	18.6	310.99	102.38	0.33	84.9	193.5	0.72		
6B	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	74.79	0.32	84.9	171.4	0.76		
7																
8																
9	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	72.45	0.31	84.9	171.4	0.75		
10	W27X94	50	27.6	2.12	50	12.5	70.8	20.7	571.32	97.34	0.17	48.7	608.8	0.17		
11	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	93.45	0.40	84.9	171.4	0.84		
12																
13	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	84.42	0.36	84.9	171.4	0.81		
14	W27X94	50	27.6	2.12	50	12.5	70.8	20.7	571.32	97.34	0.17	48.7	608.8	0.17		
15	W21x44	51	13	1.26	25	6.25	59.5	17.8	231.40	72.73	0.31	84.9	171.4	0.75		
16B	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	54.71	0.24	84.9	171.4	0.68		
17	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	42.42	0.18	84.9	171.4	0.59		
18	W21x44	36	13	1.26	25	8.3333	79.4	15.4	200.20	85.16	0.43	84.9	171.4	0.87		
19																
20	W24X68	36	20.1	1.87	25	8.3333	53.5	18.5	371.85	152.60	0.41	20.1	318.0	0.47		
21		36														
22	W21x44	36	13	1.26	32	8	76.2	15.8	205.40	49.40	0.24	122	171.4	0.87		
23A	W21x44R	36	16.72	1.421	32	10.667	90.1	13.9	232.41	86.42	0.37	122	193.5	0.93		
23B	W21x44R	36	16.72	1.421	33	10.667	90.1	13.9	232.41	80.33	0.35	122	193.5	0.91		
24	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	53.01	0.33	122	171.4	0.96		
25	W21x62	36	18.3	1.77	32	10.667	72.3	16.3	298.29	120.86	0.41	122	258.7	0.82		
26	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	51.49	0.32	122	171.4	0.95		
27	W21x44R	36	16.72	1.421	32	8	67.6	16.9	282.57	114.28	0.40	122	193.5	0.96		
28	W21x62	46	18.3	1.77	32	10.667	72.3	16.3	298.29	80.33	0.27	122	258.7	0.69		
29	W21x44R	36	16.72	1.421	32	10.667	90.1	13.9	232.41	79.22	0.34	32.9	193.5	0.49		

Chevron Beam Strength Design

Level second floor			(@ beam midspan)											
Frame	Beam	Fy	Area	I _y	L	L _b	K/r	F _{cr} /Ω	P _m /Ω	0.75E	P _r /P _c	M _a	M _n /Ω	Unity Eqn
2	W24x55R	50	19.818	1.473	25	6.25	50.9	24.8	491.49	308.29	0.63	125.1	484.0	0.86
3	W21x44R	50	16.72	1.421	25	8.33	70.4	20.7	346.10	148.46	0.43	90.9	318.6	0.68
4	W21x44	50	13	1.26	25	8.33	79.4	18.8	244.40	84.98	0.35	90.9	318.6	0.60
6A	W18x35R	50	14.095	1.421	25	8.33	70.4	20.7	291.77	101.61	0.35	89.4	189.3	0.77
6B	W18x35R	50	14.095	1.421	25	8.33	70.4	20.7	291.77	101.61	0.35	89.4	189.3	0.77
7	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	73.05	0.39	89.4	165.9	0.87
8	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	72.77	0.39	89.4	165.9	0.87
9	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	57.83	0.31	89.4	165.9	0.79
10	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	61.75	0.33	89.4	165.9	0.81
11	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	54.83	0.29	89.4	165.9	0.77
12	W18x35R	50	14.095	1.421	25	8.33	70.4	20.7	291.77	86.65	0.30	89.4	189.3	0.72
13	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	53.24	0.28	89.4	165.9	0.76
14	W18x35R	50	14.095	1.421	25	8.33	70.4	20.7	291.77	80.91	0.28	89.4	189.3	0.70
15	W18x35R	50	14.095	1.421	25	8.33	70.4	20.7	291.77	111.19	0.38	89.4	189.3	0.80
16A	W18x35R	50	14.095	1.421	25	8.33	70.3	20.7	291.77	133.35	0.46	89.4	189.3	0.88
16B														
17	W18x35R	50	14.095	1.421	25	8.33	70.4	20.7	291.77	101.42	0.35	89.4	189.3	0.77
18	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	58.87	0.31	89.4	210.0	0.69
19	W14x30	50	8.85	1.49	25	6.25	50.3	24.8	219.48	82.84	0.38	55.4	118.0	0.79
20														
21	W24x55	50	16.2	1.34	32	8.00	71.6	20.5	332.10	77.68	0.23	144	334.3	0.62
22	W24x55R	50	19.818	1.473	32	10.67	86.9	17.2	340.87	148.17	0.43	144	379.5	0.77
23A	W27x84R	50	28.433	10.433	32	10.67	12.3	29.6	841.62	222.26	0.26	277	608.8	0.67
23B	W27x84R	50	28.433	10.433	33	10.67	12.3	29.6	841.62	204.12	0.24	277	608.8	0.65
24	W24x55	50	16.2	1.34	32	10.67	95.5	15.3	247.86	80.09	0.32	144	334.3	0.71
25	W27x84	50	24.7	2.07	32	10.67	61.8	22.6	558.22	153.93	0.28	277	608.8	0.68
26	W24x55	50	16.2	1.34	32	10.67	95.5	15.3	247.86	80.09	0.32	144	334.3	0.71
27	W27x84	50	24.7	2.07	32	10.67	61.8	22.6	558.22	153.93	0.28	277	608.8	0.68
28	W27x84	50	24.7	2.07	32	10.67	61.8	22.6	558.22	153.93	0.28	277	608.8	0.68
29	W24x55R	50	19.818	1.473	32	10.67	86.9	17.2	340.87	146.07	0.43	144	379.5	0.77

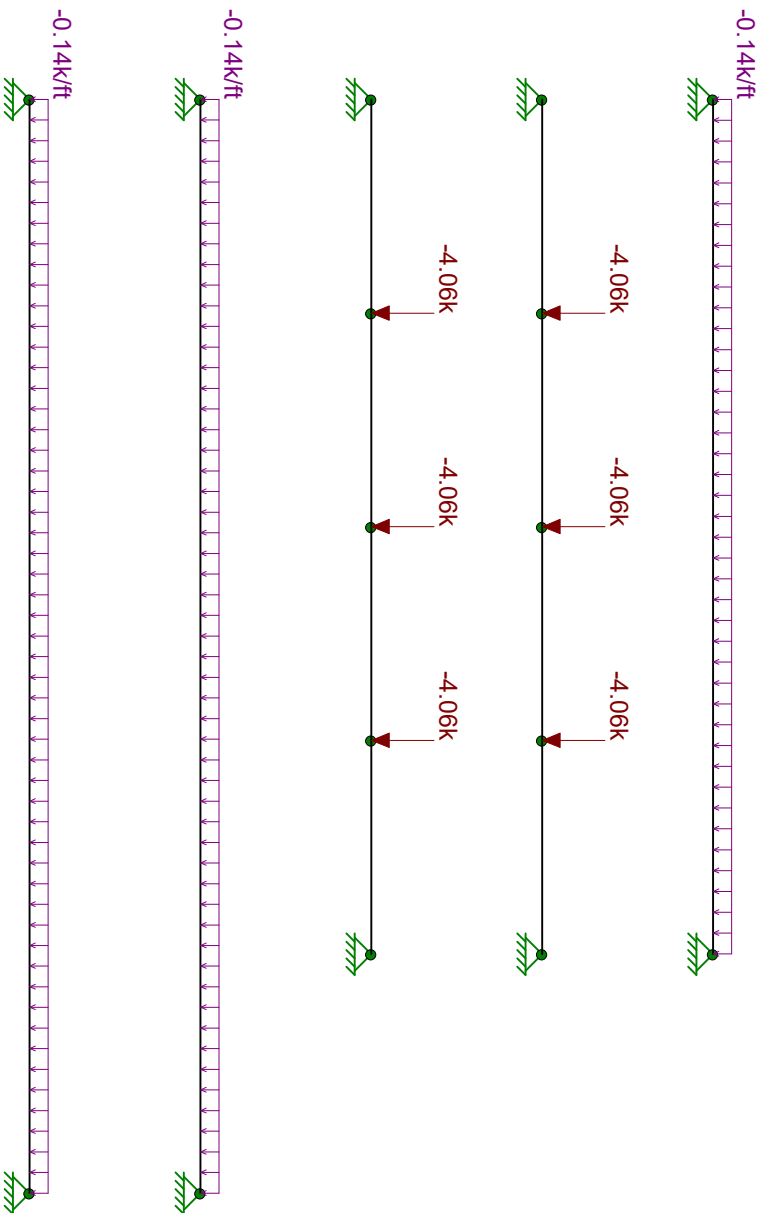
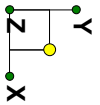


Roof Beams

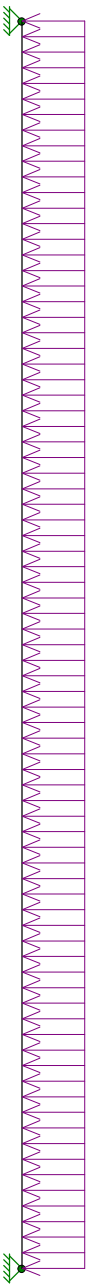
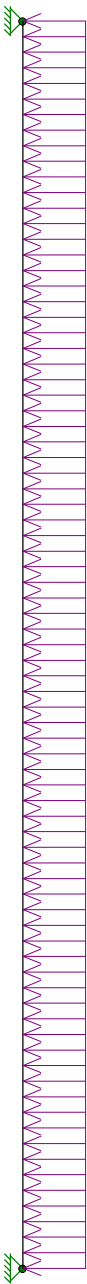
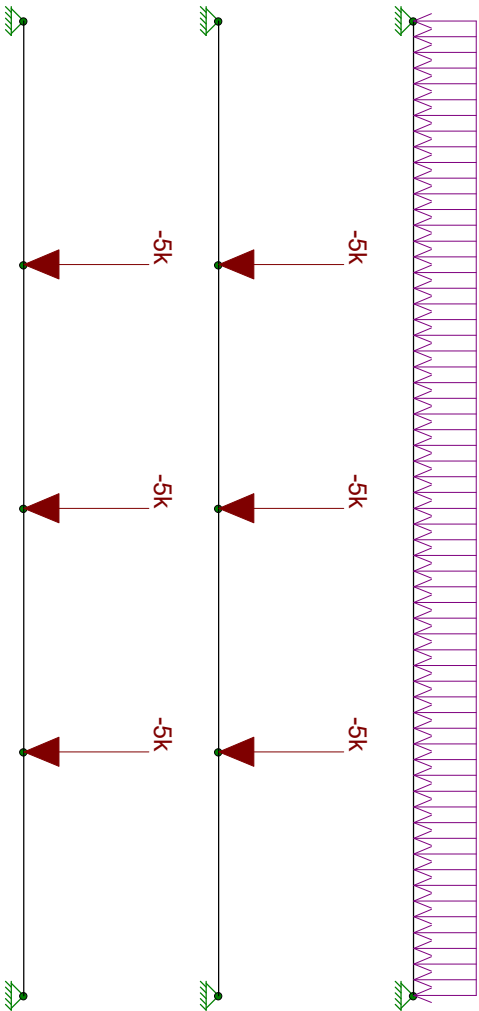
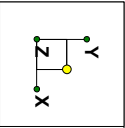
Envelope Only Solution
X-direction Reaction Units are k and k-ft (Enveloped)

Dec 10, 2024 at 8:29 AM

Chevron Beam Checks.r2d



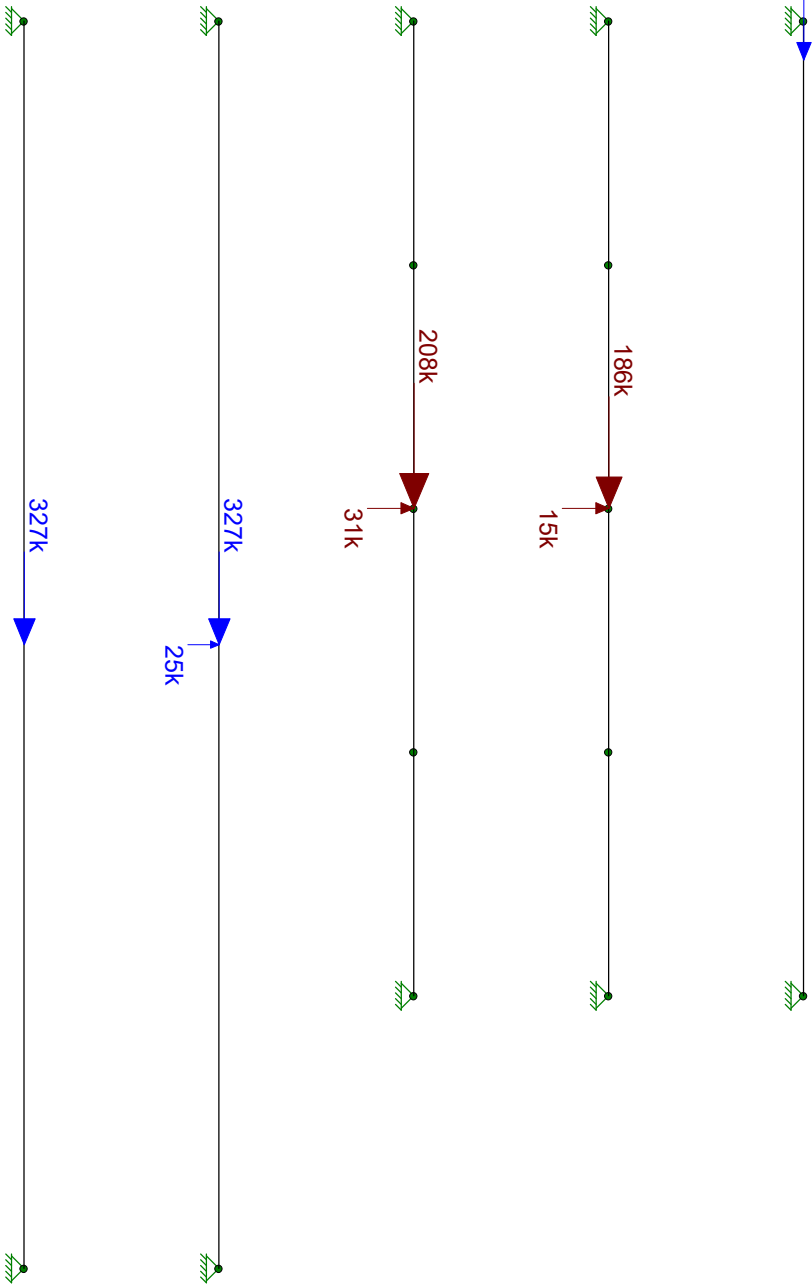
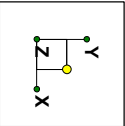
Loads: BLC 1, Dead
Envelope Only Solution
X-direction Reaction Units are k and k-ft (Enveloped)



Loads: BLC 3, Snow
Envelope Only Solution
X-direction Reaction Units are k and k-ft (Enveloped)

Dec 10, 2024 at 8:33 AM

Chevron Beam Checks.r2d



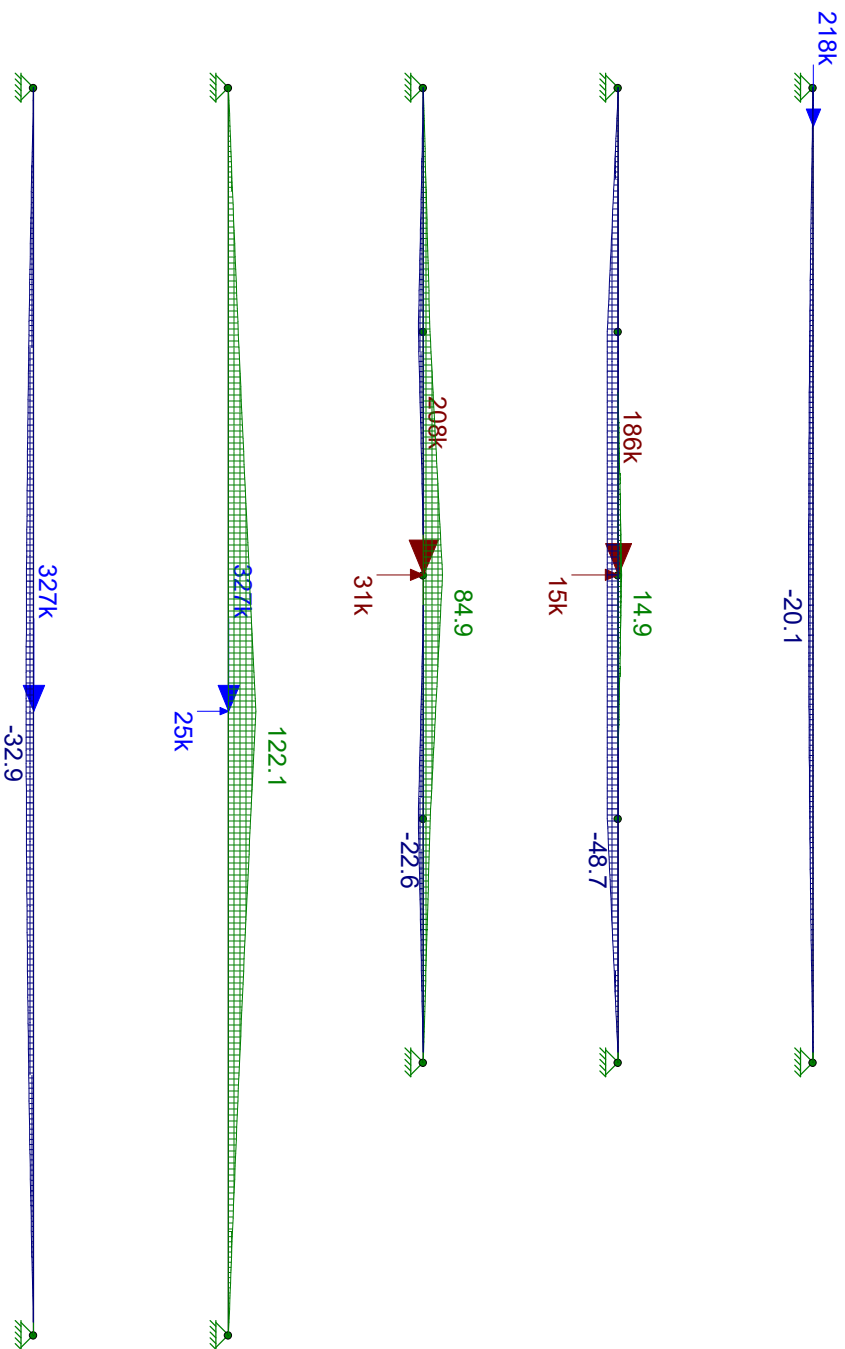
Loads: BLC 4, Seismic
Envelope Only Solution
X-direction Reaction Units are k and k-ft (Enveloped)

Dec 10, 2024 at 8:34 AM

Chevron Beam Checks.r2d

Load Combinations

	Description	S...	P...	S...	BLC	Fa...	BLC	Fa...	BLC	Fa...	BLC	Factor	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...
1	IBC 16-12 (b) (...)	Yes	Y		DL	1	EL	0.7																	
2	IBC 16-14 (a)	Yes	Y		DL	1	EL	0.5...	LL	0.75	LLS	0.75	SL	0.75	S...	0.75									
3	IBC 16-16 (a)		Y		DL	0.6	EL	0.7																	
4	IBC 16-9		Y		DL	1	LL	1	LLS	1															
5	IBC 16-10 (b)		Y		DL	1	SL	1	SLN	1															
6	IBC 16-11 (b)		Y		DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75											

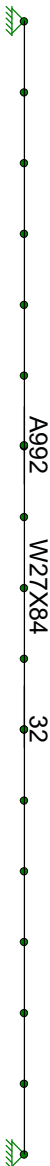
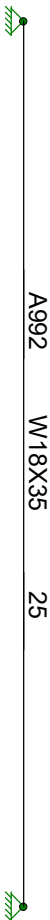
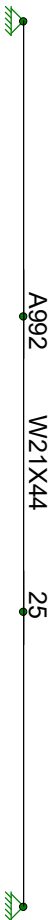
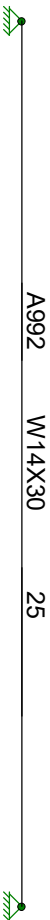


Loads: BLC 4, Seismic
Envelope Only Solution
Member Bending Moments (k-ft) (Enveloped)
X-direction Reaction Units are k and k-ft (Enveloped)

These moments are used to design
chevron beams for combined axial
effects. See spreadsheet output.

Dec 10, 2024 at 8:35 AM

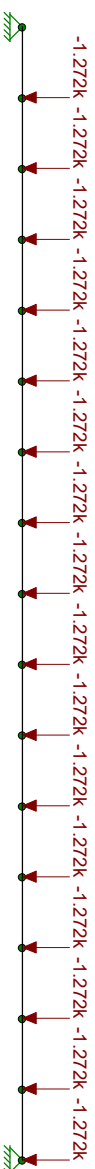
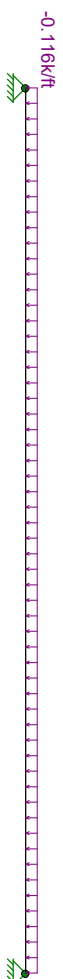
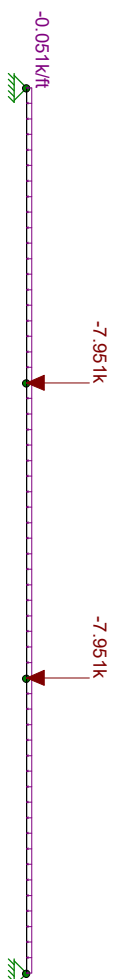
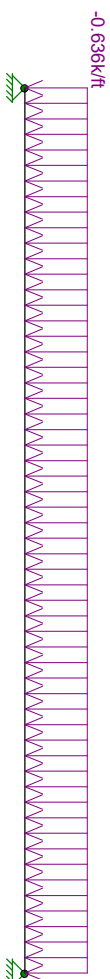
Chevron Beam Checks.r2d

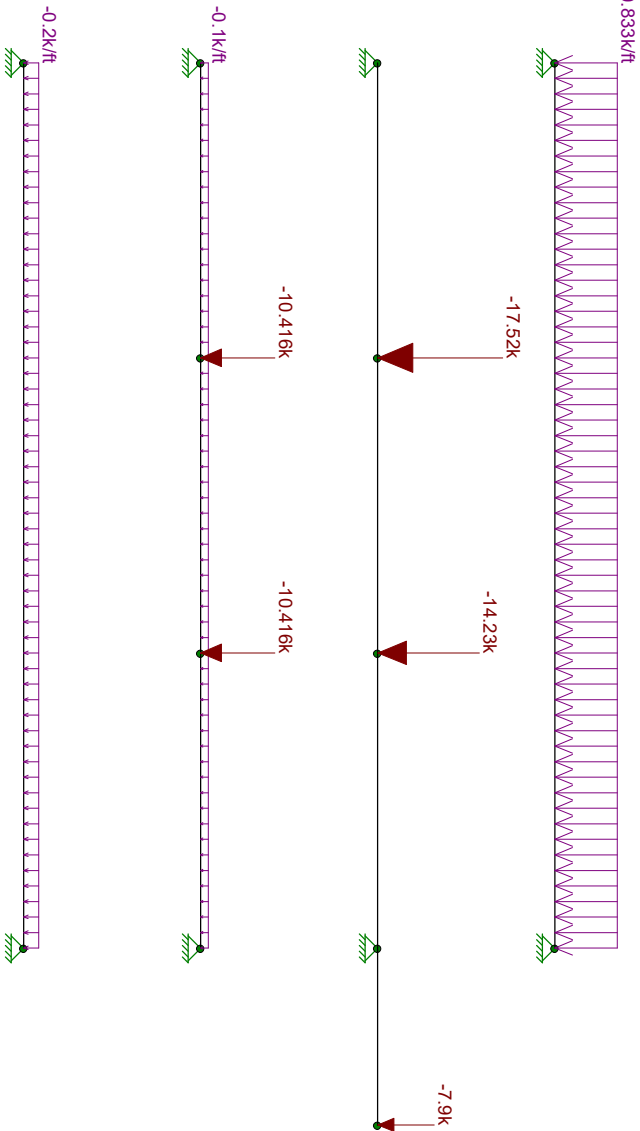
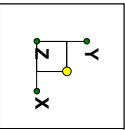


Level 2 Beams

Envelope Only Solution
X-direction Reaction Units are k and k-ft (Enveloped)

		Dec 10, 2024 at 8:37 AM
		Chevron Beam Checks.r2d

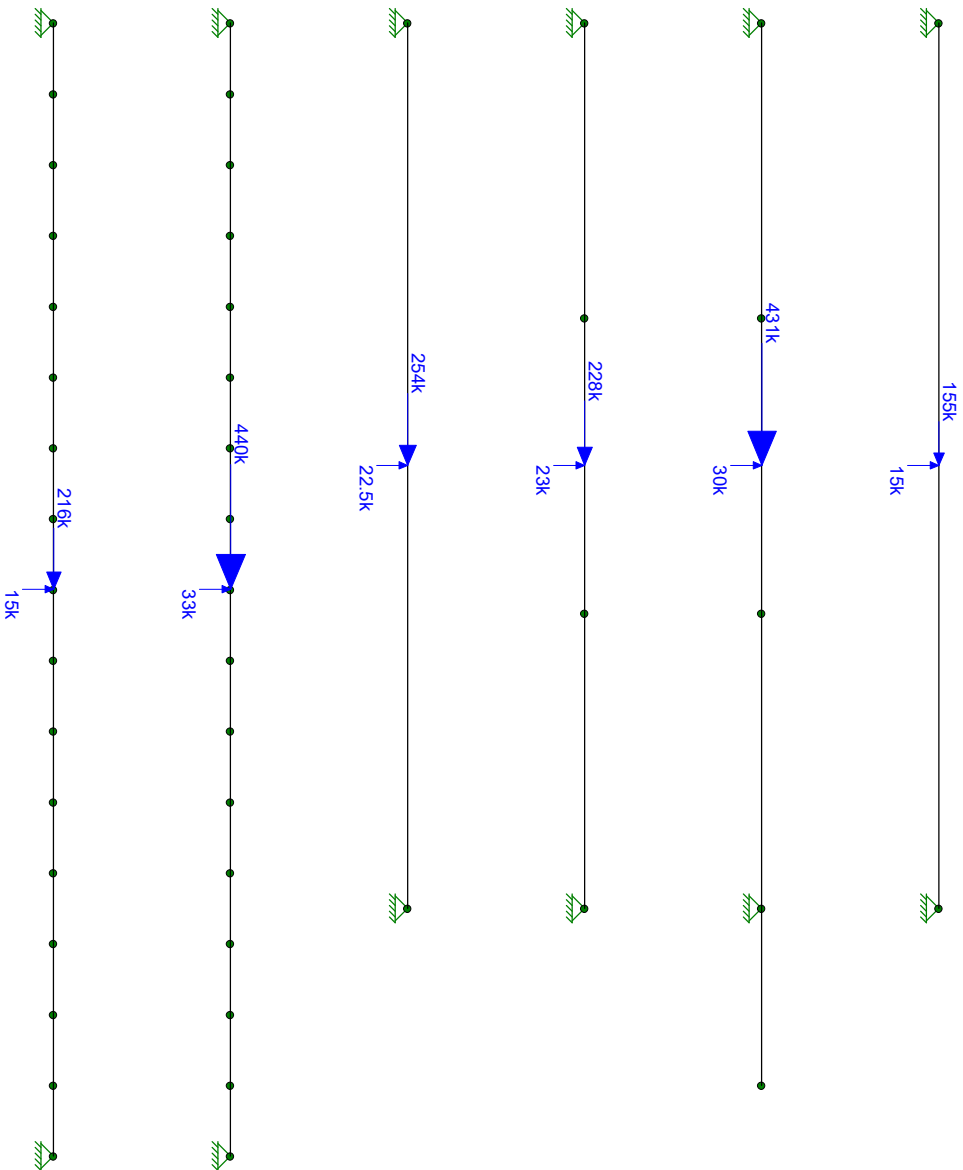
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Loads: BLC 2, Live
Envelope Only Solution
X-direction Reaction Units are k and k-ft (Enveloped)

Dec 10, 2024 at 8:40 AM

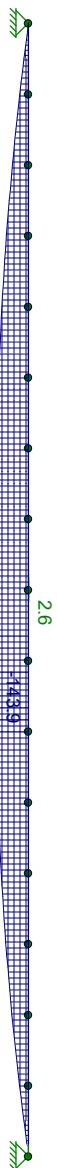
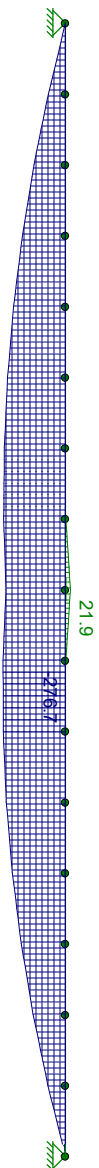
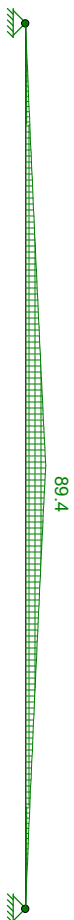
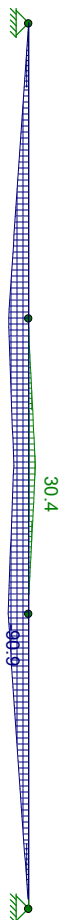
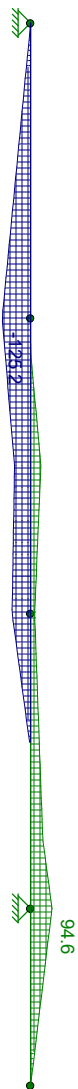
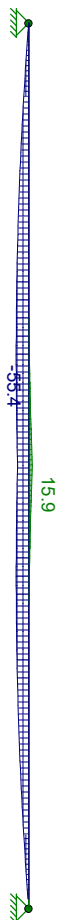
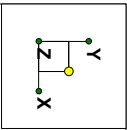
Chevron Beam Checks.r2d



Loads: BLC 4, Seismic
Envelope Only Solution
X-direction Reaction Units are k and k-ft (Enveloped)

Load Combinations

	Description	S...	P...	S...	BLC	Fa...	BLC	Fa...	BLC	Fa...	BLC	Factor	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...
1	IBC 16-12 (b) (...)	Yes	Y		DL	1	EL	0.7																	
2	IBC 16-14 (a)	Yes	Y		DL	1	EL	0.5...	LL	0.75	LLS	0.75	SL	0.75	S...	0.75									
3	IBC 16-16 (a)		Y		DL	0.6	EL	0.7																	
4	IBC 16-9		Y		DL	1	LL	1	LLS	1															
5	IBC 16-10 (b)		Y		DL	1	SL	1	SLN	1															
6	IBC 16-11 (b)		Y		DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75											



Envelope Only Solution
Member Bending Moments (k-ft) (Envelopes)
X-direction Reaction Units are k and k-ft (Envelopes)

Dec 10, 2024 at 8:41 AM

Chevron Beam Checks.r2d



QUANTUM | CONSULTING ENGINEERS

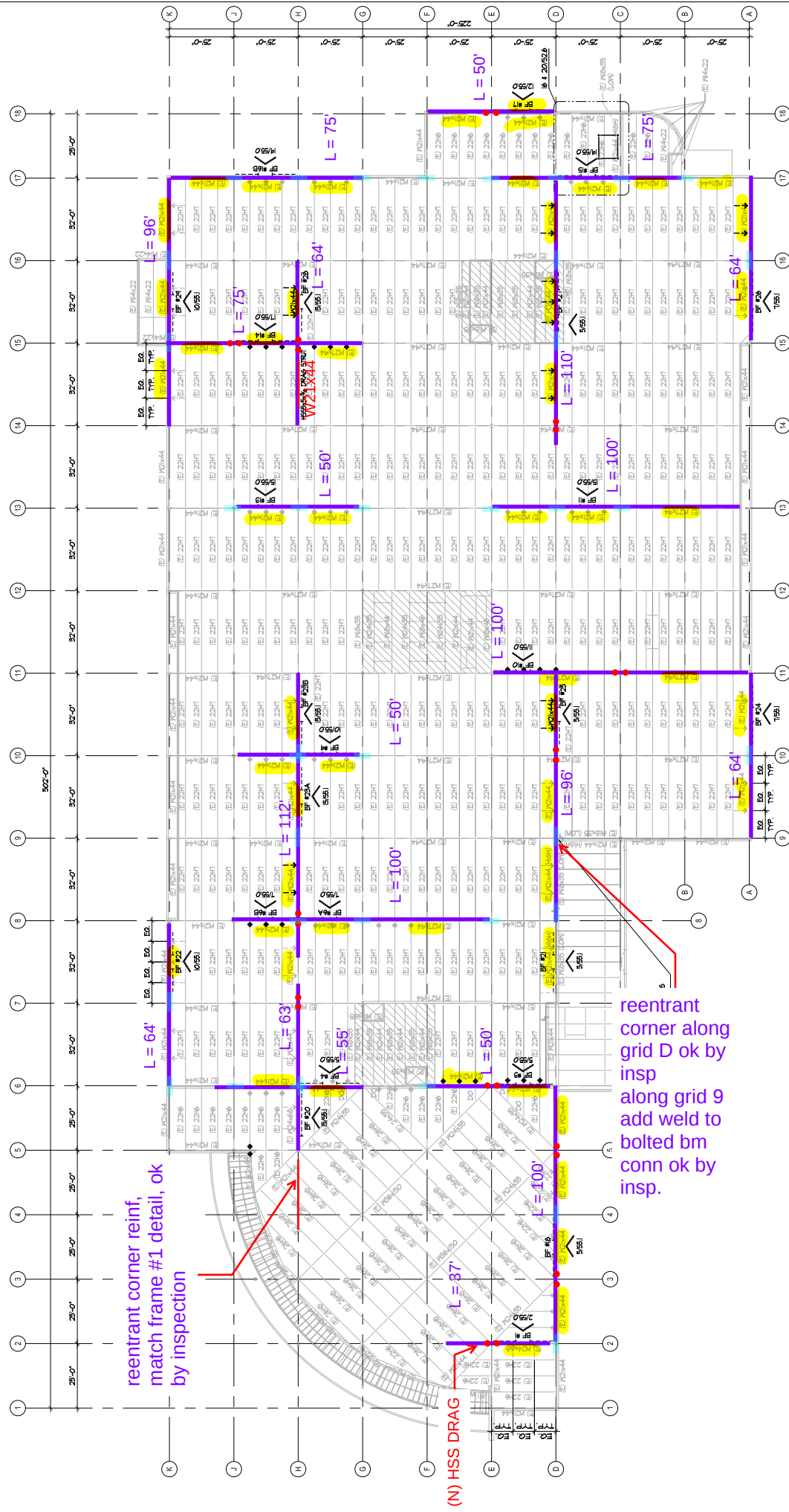
**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

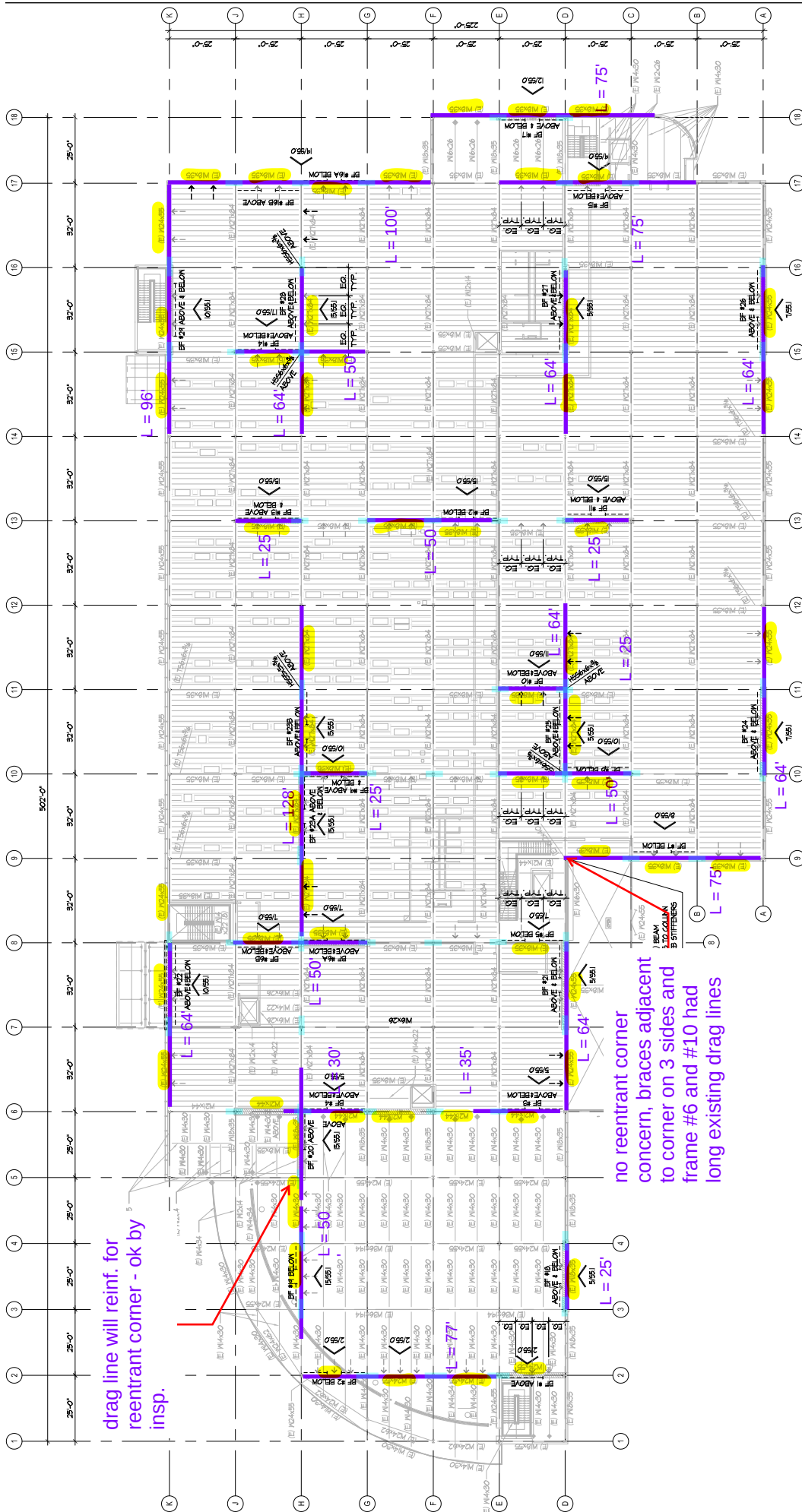
1015 39th AVENUE SE
PUYALLUP, WA 98374

QUANTUM JOB NUMBER: 19305.04

BRACED FRAMES

DIAPHRAGM AND DRAG BEAM ANALYSIS





no reentrant corner concern, braces adjacent to corner on 3 sides and frame #6 and #10 had long existing drag lines

KEY

- EXISTING DRAG CONNECTIONS
- RETROFIT DESIGN DRAG LOCATIONS


SECOND FLOOR FRAMING PLAN
 SCALE: 1/8" = 1'-0"

1511 Third Avenue, Suite 323
Seattle, WA 98101

T. 206.957.3900
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Project	South Hill Bldg C	Job #	14079.01	Page	of
Client	Beneroya	By	AMJ	Date	12/19/24
Subject	Checked				
	Date				

Diaphragm Design

Structure Information:

Roof Level

Decking:

Span = 7.0 ft

X-Direction

Diaphragm Design Shear

Diaph. Length
(ft)

Diaph. Design Force
SD (kips)

Diaph. Design Force
ASD (kips)

Diaph. Shear
(plf)

Deck Attachment

Sidelap Capacity
(plf)

Frame Line	Diaph. Length (ft)	Diaph. Design Force SD (kips)	Diaph. Design Force ASD (kips)	Diaph. Shear (plf)	Deck Attachment	Sidelap Capacity (plf)
Frame #1	37	72.7	50.9	1375	4W -VSC @ 12" OC	1400
Frame #2						OK 2-sided
Frame #3	50	98.4	68.9	1378	4W -VSC @ 12" OC	1400
Frame #4	55	101.8	71.3	1295	4W -VSC @ 12" OC	1400
Frame #5						OK 2-sided
Frame #6	100	193.9	135.7	1357	4W -VSC @ 12" OC	1400
Frame #7						OK 2-sided
Frame #8						
Frame #9	50	88.4	61.8	1237	4W -VSC @ 12" OC	1400
Frame #10	100	124.6	87.2	872	4W -VSC @ 12" OC	1400
Frame #11	100	110.0	77.0	770	4W -VSC @ 12" OC	1400
Frame #12						OK 2-sided
Frame #13	50	98.9	69.2	1385	4W -VSC @ 12" OC	1400
Frame #14	75	113.6	79.5	1060	4W -VSC @ 12" OC	1400
Frame #15	75	101.5	71.1	948	4W -VSC @ 12" OC	1400
Frame #16	75	58.6	41.0	547	4W -VSC @ 12" OC	700
Frame #17	50	40.4	28.3	566	4W -VSC @ 12" OC	700
						OK
						OK

Drag Design

Frame	Frame Length (ft)	Diaph. Len (ft)	Design Force SD (kips)	Design Force ASD (kips)	Drag Ends	Drag Force (kips)
1	25	37	123.4	86.4	1.0 Side	28.0
2						
3	25	50	158.2	110.8	1.0 Side	55.4
4	25	55	165.5	115.8	1.0 Side	63.2
5						
6	50	100	284.9	199.4	1.0 Side	99.7
7						
8						
9	25	50	115.9	81.1	1.0 Side	40.6
10	50	100	163.3	114.3	1.5 Side	38.1
11	25	100	156.3	109.4	2.0 Side	41.0
12						
13	25	50	140.6	98.4	1.0 Side	49.2
14	50	75	173.7	121.6	2.0 Side	20.3
15	25	75	163.7	114.6	2.0 Side	38.2
16	25	75	95.7	67.0	2.0 Side	22.3
17	25	50	67.4	47.1	1.0 Side	23.6

Structure Information:

Roof Level

Decking:

Span = 7.0 ft

Y-Direction

Diaphragm Design Shear

Diaph.

Diaph.

Diaph. Length

Design Force

SD (kips)

Design Force

ASD (kips)

Frame Line	(ft)	Diaph. Length	Design Force	SD (kips)	Diaph.	Design Force	ASD (kips)
Frame #18	100			91.0		63.7	
Frame #19							
Frame #20	63			124.4		87.1	
Frame #21							
Frame #22	62			61.2		42.9	
Frame #23	112			218.3		152.8	
Frame #24	69			68.2		47.7	
Frame #25	96			165.2		115.7	
Frame #26	66			66.0		46.2	
Frame #27	110			208.4		145.8	
Frame #28	64			107.7		75.4	
Frame #29	98			97.2		68.0	

Diaph. Shear

(plf)

637	4W -VSC @ 12" OC	
1382	4W -VSC @ 12" OC	
691	4W -VSC @ 12" OC	
1364	4W -VSC @ 12" OC	
692	4W -VSC @ 12" OC	
1205	4W -VSC @ 12" OC	
700	4W -VSC @ 12" OC	
1326	4W -VSC @ 12" OC	
1178	4W -VSC @ 12" OC	
694	4W -VSC @ 12" OC	

Sidelap

Capacity

(plf)

700		OK
1400		OK 2 Sided
700		OK
1400		OK 2 Sided
700		OK
1400		OK 2 Sided
700		OK
1400		OK 2 Sided
1400		OK 2 Sided
700		OK

Drag Design

Frame	Frame Length (ft)	Diaph. Len (ft)	Design Force SD (kips)
18	25	100	125.9
19	0	0	0.0
20	25	63	170.9
21	0	0	0.0
22	32	62	90.2
23	64	112	302.1
24	32	69	101.0
25	32	96	225.7
26	32	66	98.1
27	32	110	288.0
28	32	64	147.9
29	32	98	137.6

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Subject					

Diaphragm Design

Structure Information: Level 2

Decking: Span = 8.0 ft

X-Direction

Diaphragm Design Shear

Frame Line	Diaph. Length (ft)	Diaph. Design Force		Diaph. Shear (plf)	Deck Attachment	Sidelap Capacity	
		SD (kips)	ASD (kips)			(plf)	
Frame #1	0.01	0.0	0.0	0	W3-6.5"(4 welds)	3720	OK Drags to Frame 2
Frame #2	77	544.2	380.9	4947	W3-6.5"(4 welds)	4960	OK
Frame #3	35	189.7	132.8	3793	Shallow Vercor (3.5")	3980	OK 2-sided
Frame #4	30	145.8	102.0	3401	Shallow Vercor (3.5")	3980	OK 2-sided
Frame #5							
Frame #6	50	227.9	159.6	3191	Shallow Vercor (3.5")	3980	OK 2-sided
Frame #7	75	208.7	146.1	1948	Shallow Vercor (3.5")	1990	OK
Frame #8	50	207.9	145.5	2911	Shallow Vercor (3.5")	3980	OK 2-sided
Frame #9	25	105.0	73.5	2939	Shallow Vercor (3.5")	3980	OK 2-sided
Frame #10	25	82.3	57.6	2304	Shallow Vercor (3.5")	3980	OK 2-sided
Frame #11	25	31.4	22.0	880	Shallow Vercor (3.5")	3980	OK 2-sided
Frame #12	50	206.3	144.4	2888	Shallow Vercor (3.5")	3980	OK 2-sided
Frame #13	25	42.5	29.8	1191	Shallow Vercor (3.5")	3980	OK 2-sided
Frame #14	50	119.2	83.5	1669	Shallow Vercor (3.5")	3980	OK 2-sided
Frame #15	75	197.1	138.0	1840	Shallow Vercor (3.5")	1990	OK 1-sided @ Stair
Frame #16	100	239.4	167.6	1676	Shallow Vercor (3.5")	1990	OK
Frame #17	75	225.0	157.5	2100	W3-6.5"(4 welds)	2480	OK

Drag Design

Frame	Frame Length (ft)	Diaph. Len (ft)	Design Force SD (kips)	Diaph. Design Force ASD (kips)	Drag Ends	Drag Force (kips)	to #2	amplified frame shear
1	0	25	123.4	86.4	1.0 Side	86.4		
2	29	104	587.2	411.1	1.0 Side	296.4		
3	25	35	189.7	132.8	1.0 Side	37.9		
4	25	30	145.8	102.0	1.0 Side	17.0		
5	25							
6	50	50	227.9	159.6	1.0 Side	0.0		
7	25	75	208.7	146.1	2.0 Side	48.7		
8	25	50	207.9	145.5	1.0 Side	72.8		
9	25	25	105.0	73.5	1.0 Side	0.0		
10	25	25	82.3	57.6	1.0 Side	0.0		
11	25	25	31.4	22.0	1.0 Side	0.0		
12	25	50	206.3	144.4	1.0 Side	72.2		
13	25	25	42.5	29.8	1.0 Side	0.0		
14	25	50	119.2	83.5	1.0 Side	41.7		
15	25	75	197.1	138.0	2.0 Side	46.0		
16	25	100	239.4	167.6	1.0 Side	125.7		
17	25	75	225.0	157.5	2.0 Side	52.5		

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Project	South Hill Bldg C	Job #	14079.01	Page	of
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Drag Beam Design

Level Roof			ASD										Unity Eqn	
Frame	Beam	Fy	Area	r _y	L	L _b	k/r	F _{cr} /Ω	P _n /Ω	0.7OE	P _t /P _c	M _a	M _n /Ω	
1	L5x5x3/8	36	3.65	0.986	8	8	97.4	13	47.45	28.02	0.59	0	0.01	0.59
3	W21x44R	36	16.72	1.421	25	6.25	52.8	18.6	310.99	55.38	0.18	166	193.4731	0.947
4	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	63.18	0.27	105.29	171.3772	0.82
6	W27x94	50	27.6	2.12	50	12.5	70.8	20.7	571.32	99.72	0.17	485	693.6128	0.79
9	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	40.57	0.18	117.86	171.3772	0.78
10	W27x94	50	27.6	2.12	50	12.5	70.8	20.7	571.32	38.11	0.07	485	693.6128	0.73
11 (E-D)	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	41.03	0.18	117.86	171.3772	0.78
11 (A-C)	W27x94	50	27.6	2.12	50	12.5	70.8	20.7	571.32	41.03	0.07	485	693.6128	0.74
13	W21x44	36	13	1.26	25	12.5	119.0	10.1	131.30	49.20	0.37	117.86	171.3772	0.99
14 (G-H)	W27x94	50	27.6	2.12	25	12.5	70.8	20.7	571.32	20.26	0.04	485	693.6128	0.72
14 (J-K)	W21x44	36	13	1.26	25	12.5	119.0	10.1	131.30	20.26	0.15	117.86	171.3772	0.76
15	W21x44	36	13	1.26	25	6.25	59.5	17.8	231.40	38.19	0.17	105.71	171.3772	0.70
16	W21x44	36	13	1.26	25	12.5	119.0	10.1	131.30	22.33	0.17	67.857	171.3772	0.48
17	W21x44	36	13	1.26	25	12.5	119.0	10.1	131.30	23.57	0.18	55	171.3772	0.41
18	W21x44	36	13	1.26	25	8.333	79.4	15.4	200.20	33.04	0.17	32.143	171.3772	0.27
18 (5-6)	W21x45	36	16.72	1.421	25	8.333	70.4	16.5	275.88	7.93	0.03	32.143	193.4731	0.18
20 (6-7)	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	72.16	0.44	42.143	171.3772	0.66
20 (7-8)	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	6.87	0.04	97.143	171.3772	0.59
22	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	30.55	0.19	35.714	171.3772	0.30
23 (8-9)	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	90.62	0.56	42.143	171.3772	0.78
23 (7-8)	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	12.95	0.08	42.143	171.3772	0.29
24	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	37.90	0.23	35.714	171.3772	0.42
25	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	105.33	0.65	42.143	171.3772	0.87
26	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	35.37	0.22	35.714	171.3772	0.40
27 (14-15)	W21x44	36	13	1.26	32	10.667	101.6	12.5	162.50	71.48	0.44	42.143	171.3772	0.66
28	W21x44	46	13	1.26	32	10.667	101.6	12.5	162.50	51.75	0.32	42.143	171.3772	0.54
29	W21x44	36	13	1.26	32	10.67	101.6	12.5	162.50	32.42	0.20	35.714	171.3772	0.31

Drag Beam Design

Level second floor																		
Frame	Beam	Fy	Area	r _y	L	L _b	kl/r	F _{cr} /Ω	P _n /Ω	0.7QE	P _r /P _c	M _a	M _n /Ω	Unity Eqn				
1	W24x55	50	16.2	1.34	25	6.25	56.0	23.8	385.56	86.38	0.22	371.43	484	0.91				
2 (F-G)	W24x55R	50	28.003	1.836	25	6.25	40.8	26.5	742.08	296.44	0.40	321.43	484	0.99				
2(E-F)	W24x55R	50	28.003	1.836	25	6.25	40.8	26.5	742.08	197.62	0.27	371.43	484	0.95				
3	W21x44	50	13	1.26	25	8.33	79.4	18.8	244.40	37.93	0.16	171.43	318.6	0.62				
4	W21x44	50	13	1.26	25	8.33	79.4	18.8	244.40	17.01	0.07	202.14	318.6	0.67				
5	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	0.00	0.00	27.143	119.4611	0.23				
6	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	0.00	0.00	27	165.9182	0.16				
7	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	48.70	0.26	28.714	165.9182	0.41				
8	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	72.77	0.39	27	165.9182	0.53				
9	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	0.00	0.00	27	165.9182	0.16				
10	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	0.00	0.00	27	165.9182	0.16				
11	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	0.00	0.00	27	165.9182	0.16				
12	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	72.21	0.38	27	165.9182	0.53				
13	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	0.00	0.00	27	165.9182	0.16				
14	W18x35	50	10.3	1.22	25	8.33	82.0	18.3	188.49	41.73	0.22	27	165.9182	0.37				
15	W18x35R	50	14.095	1.421	25	6.25	52.8	24.4	343.92	45.99	0.13	178.57	189.3214	1.01				
16	W18x35	50	10.3	1.22	25	8.33	81.9	18.3	188.49	125.69	0.67	21.1	165.9182	0.78				
17	W18x35	50	10.3	1.22	25	6.25	61.5	22.6	232.78	52.51	0.23	167.14	210	0.93				
18	W18x35	50	10.3	1.22	25	25	159.8	5.87	60.46	0.00	0.00	69.286	210	0.33				
19	W14x30	50	8.85	1.49	25	6.25	50.3	24.8	219.48	55.23	0.25	117	182.9674	0.82				
20 (4-5)	W14x30	50	8.85	1.49	25	6.25	50.3	24.8	219.48	59.82	0.27	117	182.9674	0.84				
20 (6-7)	W27x84	50	24.7	2.07	32	10.667	61.8	22.6	558.22	77.68	0.14	514.29	608.7824	0.91				
21	W24x55	50	16.2	1.34	32	8	71.6	20.5	332.10	77.68	0.23	272.14	333.9987	0.96				
22	W24x55	50	16.2	1.34	32	10.67	95.5	15.3	247.86	57.79	0.23	261.43	333.9987	0.93				
23	W27x84	50	24.7	2.07	32	10.67	61.8	22.6	558.22	124.65	0.22	514.29	608.7824	0.97				
24	W24x55	50	16.2	1.34	32	10.67	95.5	15.3	247.86	58.53	0.24	272.21	334.3313	0.96				
25	W27x84	50	24.7	2.07	32	10.67	61.8	22.6	558.22	107.71	0.19	514.29	608.7824	0.94				
26	W24x55	50	16.2	1.34	32	10.67	95.5	15.3	247.86	59.29	0.24	272.21	334.3313	0.96				
27	W27x84	50	24.7	2.07	32	10.67	61.8	22.6	558.22	92.95	0.17	514.29	608.7824	0.93				
28	W27x84	50	24.7	2.07	32	10.67	61.9	22.6	558.22	124.98	0.22	514.29	608.7824	0.97				
29	W24x55	50	16.2	1.34	32	10.67	95.6	15.3	247.86	45.11	0.18	272.21	334.3313	0.91				

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Project	South Hill Bldg C	Job #	14079.01	Page	of
Client	Beneroya	By	AMJ	Date	12/19/24

Drag Beam End Connection

Level	Roof										
Frame	Beam	Fy	Existing Connection	Shear D+L	Shear Capacity	Tension 0.75E	Tension Capacity	Weld Capacity	Weld Check		
1	W24x68	36	Shear Tab	5	5	28.02	35 OK				
3	W21x44R	36	Shear Tab	14	21	55.38	27 NG	89.08	OK	WELD (E) TAB	
4	W21x44	36	End Plate	13	136.407	63.18	184.958 OK				
6	W27x94	50	End Plate	34	175.38	99.72	235.401 OK				
9	W21x44	36	End Plate	14	136.407	40.57	184.958 OK				
10	W27x94	50	Beam Seat	0	0	38.11	23.5 NG	74.00	OK	NEW WEB PLATE	
11 (E-D)	W21x44	36	End Plate	14	136.407	41.03	184.958 OK				
11 (A-C)	W27x94	50	End Plate	34	175.38	41.03	235.401 OK				
13	W21x44	36	End Plate	14	136.407	49.20	184.958 OK				
14 (G-H)	W27x94	50		34	5	20.26	35 OK				
14 (J-K)	W21x44	36	Beam Seat	0	0	20.26	20.5 OK			NEW WEB PLATE	
15	W21x44	36	End Plate	13	136.407	38.19	184.958 OK				
16	W21x44	36	End Plate	9	136.407	22.33	184.958 OK				
17	W21x44	36	Shear Tab	7	7	23.57	27 OK			WELD (E) TAB	
18	W21x44	36	Shear Tab	4	7	33.04	27 NG	89.08	OK	WELD (E) TAB	
18 (5-6)	W21x45	36	Shear Tab	4	21	7.93	27 OK				
20 (6-7)	W21x44	36	End Plate	6	136.407	72.16	184.958 OK				
20 (7-8)	W21x44	36	Shear Tab	6	7	6.87	27 OK				
21	W21x44	36	End Plate	11	136.407	0.00	184.958 OK				
22	W21x44	36	End Plate	5	136.407	30.55	184.958 OK				
23 (8-9)	W21x44	36	End Plate	6	136.407	90.62	184.958 OK				
23 (7-8)	W21x44	36	Shear Tab	6	7	12.95	27 OK				
24	W21x44	36	End Plate	5	136.407	37.90	184.958 OK				
25	W21x44	36		6	0	105.33	0 NG	0.00	NG		
26	W21x44	36	End Plate	5	136.407	35.37	184.958 OK				
27 (14-15)	W21x44	36	End Plate	6	136.407	71.48	184.958 OK				
27 (13-14)	Rod	46		6	0	9.10	10.84 OK			Joist Seat Weld to beam OK	
28	W21x44	46	Shear Tab	6	7	51.75	27 NG			New welded Shear Pl's	
29	W21x44	36	End Plate	5	136.407	32.42	184.958 OK				

Drag Beam End Connection

Level		second floor									
Frame	Beam	Fy	Existing Connection	Shear D+L	Shear Capacity	Tension 0.75E	Tension Capacity	Weld			
								Capacity	Weld Check		
1	W24x55	50	End Plate	43	154.038	86.38	210.18	OK			
2 (F-G)	W24x55R	50	End Plate	39	154.038	296.44	210.18	NG	296.94	OK	PREV. WELDED
2(E-F)	W24x55R	50	End Plate	43	154.038	197.62	210.18	OK			
3	W21x44	50	End Plate	21	136.407	37.93	184.958	OK			
4	W21x44	50	End Plate	12	136.407	17.01	184.958	OK			
5	W18x35	50	End Plate		115.064	0.00	201.772	OK			
6	W18x35	50	End Plate		115.064	0.00	201.772	OK			
7	W18x35	50	End Plate	5	115.064	48.70	201.772	OK			
8	W18x35	50	End Plate	4	115.064	72.77	201.772	OK			
9	W18x35	50	End Plate	4	115.064	0.00	201.772	OK			
10	W18x35	50	End Plate	4	115.064	0.00	201.772	OK			
11	W18x35	50	End Plate	4	115.064	0.00	201.772	OK			
12	W18x35	50	End Plate	4	115.064	72.21	201.772	OK			
13	W18x35	50	End Plate	4	115.064	0.00	201.772	OK			
14	W18x35	50	End Plate	4	115.064	41.73	201.772	OK			
15	W18x35R	50	End Plate	22	115.064	45.99	201.772	OK			
16	W18x35	50	End Plate	4	115.064	125.69	201.772	OK			
17	W18x35	50	End Plate	20	115.064	52.51	201.772	OK			
18	W18x35	50	End Plate	11	115.064	0.00	201.772	OK			
19	W14x30	50	End Plate	24	86.2982	55.23	201.772	OK			
20 (4-5)	W14x30	50	End Plate	18	86.2982	59.82	201.772	OK			
20 (6-7)	W27x84	50	End Plate	58	175.38	77.68	235.401	OK			
21	W24x55	50	End Plate	31	154.038	77.68	210.18	OK			
22	W24x55	50	End Plate	30	154.038	57.79	210.18	OK			
23	W27x84	50	End Plate	58	175.38	124.65	235.401	OK			
24	W24x55	50	End Plate	31	154.038	58.53	210.18	OK			
25	W27x84	50	End Plate	58	175.38	107.71	235.401	OK			
26	W24x55	50	End Plate	31	154.038	59.29	210.18	OK			
27	W27x84	50	End Plate	58	175.38	92.95	235.401	OK			
28	W27x84	50	End Plate	58	175.38	124.98	235.401	OK			
29	W24x55	50	End Plate	30	154.038	45.11	210.18	OK			

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Client	Project	Job #	By	AMJ	Date	Page	of
	South Hill Bldg C	14079.01	Beneroya		12/19/24		

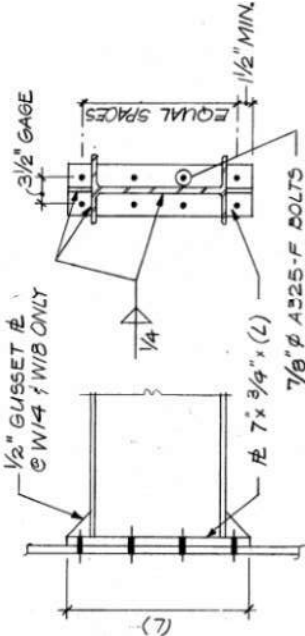
Bolted Frame Beam Capacity

Bolt Diam = 0.875
Bolt Grade = A325-F
Shear Rn/Ω = 12.6
Tension Rn/Ω = 26.5
Bolt Bearing = 16.5
Ab = 0.601
Tension Fnt = 92 ksi
Tension Fnt/Ω = 44 ksi
Tension Ω= 2.091

EQ J3-3-b 1.3 Fnt - (ΩFnt/Fnv)/rv
Modified: 1.3 Fnt - (Fnt *Uc V)

Weld size 0.3125

Beam			Column		End Plate					Reactions		Beam Web Weld Bolt Capacity				W12x65 Web Tension Capacity				End Plate Weld
Beam Size	Web T (in)	weld	Column Size	Fy	tf (in)	#Bolts	t	b	h	Fy	Va (max)	Ta	Rnv/Ω (kips)	Web Rnv/Ω (kips)	Bolt Rnv/Ω (kips)	Uc V	Fnt	Tn/Ω (kips)	Tn/Ω (kips)	End Plate Weld
W14x30	11.625	0.25	W12x65	36	0.605	8	0.75	7	24	36	24	201.8	86.30	100.8	100.8	0.24	92.0	212	201.8	287.6606
W18x35	15.5	0.25	W12x65	36	0.605	8	0.75	7	24	36	22	201.8	115.06	100.8	100.8	0.22	92.0	212	201.8	287.6606
W21x44	18.375	0.25	W12x65	36	0.605	8	0.75	7	22	36	21	185.0	136.41	100.8	100.8	0.21	92.0	212	185.0	269.1019
W24x55	20.75	0.25	W12x65	36	0.605	10	0.75	7	25	36	43	210.2	154.04	126	126	0.34	88.2	254.06349	210.2	296.94
W24x68	20.75	0.25	W12x65	36	0.605	10	0.75	7	25	36	5	210.2	154.04	126	126	0.04	92.0	265	210.2	296.94
W27x84	23.625	0.25	W12x65	36	0.605	12	0.75	7	28	36	58	235.4	175.38	151.2	151.2	0.38	84.3	291	235.4	324.7781
W27x94	23.625	0.25	W12x65	36	0.605	12	0.75	7	28	36	34	235.4	175.38	151.2	151.2	0.22	92.0	318	235.4	324.7781



BEAM DEPTH	27	24	21	18	14
NO. BOLTS	12	10	8	6	8
END PLATE LENGTH (L)	28	25	22	24	24

52-2 3/4" = 1'-0"

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Bolted Frame Beam Capacity

Bolt Diam = 0.75 Ab = 0.442

Bolt Grade = A325-F

Shear Rn/Ω = 18.6 AISC 9th Table I-D

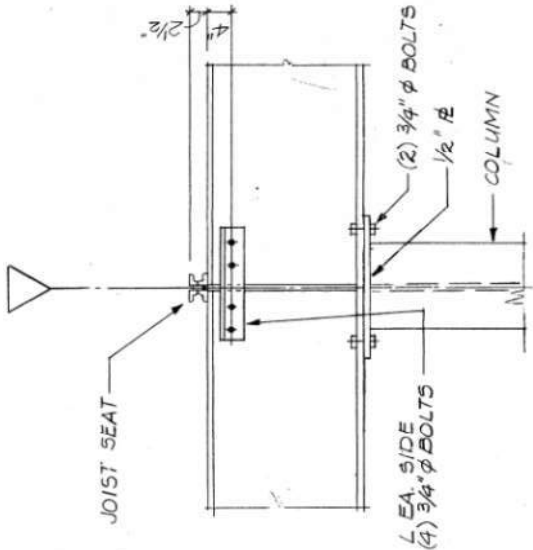
Bearing/Bolt Spacing 52.2 k/in

Bearing/Bolt Edge 29.4 k/in

Beam	Beam Size	Fy	tw (in)	Leh	# Bolts	s	Double Angle 3.5x3.5 Clips	Bolt Shear	Bolt shear Spacing (kips)	Bolt Shear Edge (kips)	Controlling	Weld Lengtl Thickness	Weld Retrofit capacity
	W21x44	36	0.35	1.5	2	3	0.3125	37.2	36.54	20.58	20.58	16	0.19
	W27x94	50	0.515	1.5	2	3	0.3125	37.2	53.766	30.282	30.28	16	0.19

assumed

assumed



Block Shear 3/8" Web per AISC J4.3
Reinf Plate is 6" deep and extends 5.75" in from end of member

L_T = 6 in
L_V = 11.5 in
F_y = 36 ksi
F_u = 58 ksi
U_{bs} = 1
Ant = 2.25 sqin = Agt
Anv = 4.31 sqin = Agv

R_n = 223.7 kips
R_n / Ω = 111.8 kips > 89.08 kips ok

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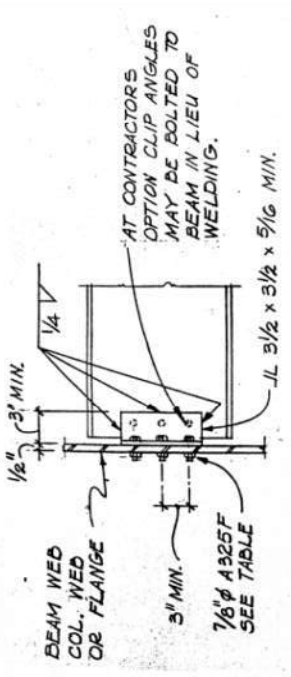
Bolted Frame Beam Capacity

Bolt Diam = 0.875
Bolt Grade = A325-F

Ab = 0.601

Beam		Column			Double Angle 3.5x3.5 Clips			Reactions	Allowable Tension	Weld Retrofit					
Beam Size	tw (in)	# Bolts	Column Size	Fy	tf (in)	# Bolts	t	gage	Fy	Va (max)	Rnt/Ω (kips)	Length Legs to col	Legs to Web	Thickness	capacity
W21x44	0.35	4	W12x65	36	0.605	8	0.313	4	36	7	27	24	24	0.25	89.082
W21x44R	0.35	4	W12x65	36	0.605	8	0.313	4	36	21	27	24	24	0.25	89.082
W24x68	0.415	5	W12x65	36	0.605	10	0.313	4	36	5	35	24	30	0.25	89.082

Per Clip Angle Spreadsheet



CONN. BOLT TABLE	
BEAM DEPTH	MIN. NO. BOLTS 1/2 EA. ANGLE
6", 8"	2
10", 12"	4
14", 16"	6
18", 21"	8
24", 27"	10
30", 33"	12
36"	14

TYPICAL BEAM CONNECTIONS

Retrofit Shear Tab
Beam W21x44
Tension Demand 105.33
in Weld Req'd (5/16 in Weld Req'd (1/2)
22.70 14.19
double side 15" ok



QUANTUM | CONSULTING ENGINEERS

**1015 BUILDING RISK CATEGORY IV
VOLUNTARY SEISMIC UPGRADE**

1015 39th AVENUE SE
PUYALLUP, WA 98374

QUANTUM JOB NUMBER: 19305.04

BRACED FRAMES

BASE PLATE AND ANCHORAGE

Braced Frame Base Plate Reactions

IBC 2021, ASCE 7-16

Sds 1.006

Ω (W) 2.5

Indicates new frame or frame expansion
(No existing B.F. base plate)

Column Loads

Frame	NODE	DEAD (k)	LIVE (k)	SNOW (k)	(+/-)EQ (k) Rz	(k) Rx
1	215	35.64	29.69	7.06	34.30	0.00
	225	63.60	52.89	22.76	34.30	0.00
2	239	80.49	50.10	36.66	195.71	117.45
	252	50.17	42.25	4.19	195.71	117.45
3	212	44.37	38.73	13.08	138.18	56.60
	224	70.36	70.04	23.17	138.16	56.50
4	238	90.42	70.00	22.20	126.89	48.58
	248	67.86	72.67	16.75	138.24	48.54
7	195	37.17	41.66	10.29	61.36	36.84
	201	54.94	42.12	18.73	61.36	36.84
8	200	74.46	79.99	29.69	59.07	35.44
	208	62.06	71.52	20.49	179.31	35.47
<u>9</u>	<u>234</u>	72.46	78.24	29.41	87.14	33.06
	<u>245</u>	67.80	83.09	20.60	85.47	33.06
10	207	67.34	80.29	20.10	187.63	35.25
	220	109.31	78.85	29.72	103.97	35.31
<u>11</u>	<u>199</u>	71.39	79.81	27.94	86.91	26.11
	<u>206</u>	64.85	80.18	20.46	86.91	26.11
12	219	64.60	79.99	19.78	63.41	38.05
	228	64.78	80.01	20.22	63.41	38.05
<u>13</u>	<u>243</u>	63.78	79.84	19.93	82.53	26.11
	<u>255</u>	64.02	80.16	20.07	82.53	26.11
14	242	65.89	81.20	19.82	157.58	46.22
	254	64.46	79.04	20.11	124.96	46.25
15	197	59.39	62.43	17.51	151.29	63.54
	203	58.03	60.61	17.91	151.29	63.54
17	202	33.38	25.77	8.36	115.27	57.95
	216	38.12	32.47	8.53	115.27	57.95



Project: South Hill 1015 Building

Date: 12/9/24

Job No: 19305.04

By: TVM

Sheet: 1

Client: Benaryoa

Braced Frame Base Plate Reactions

IBC 2021, ASCE 7-16

Sds 1.006

Ω 2.5

Column Loads

Frame	NODE	DEAD (k)	LIVE (k)	SNOW (k)	(+/-)EQ (k) Rz	(k) Ry
18	213	35.67	33.41	4.71	88.10	31.94
	214	71.16	63.67	14.13	88.10	31.88
19	250	83.68	93.70	5.87	47.89	28.74
	251	108.49	79.87	38.13	47.89	28.74
20	248	67.86	72.67	16.75	138.24	0.02
	249	102.03	92.60	23.26	47.43	0.00
21	210	56.23	46.35	23.47	52.46	40.55
	211	50.44	42.15	20.01	52.82	40.57
22	260	37.83	28.80	20.95	75.39	42.89
	261	43.86	39.93	21.06	75.39	42.89
24	193	47.13	42.02	20.29	81.50	45.77
	194	48.32	42.25	20.92	81.50	45.77
25	207	67.34	80.29	20.10	187.63	86.69
	208	62.06	71.52	20.49	179.31	86.72
26	191	47.97	42.03	20.68	80.87	45.77
	192	47.48	42.23	20.53	80.87	45.77
27	204	82.99	79.68	19.87	175.26	86.64
	205	83.77	80.31	20.13	175.26	86.64
28	241	64.50	79.76	20.07	142.27	84.75
	242	65.89	81.20	19.82	173.58	84.68
29	258	42.09	43.77	13.32	102.27	56.03
	259	45.97	46.18	15.56	102.27	55.25

Special MultiBay Frames

6	236	73.37	79.90	29.24	108.97	38.78 Rx
	247	66.81	82.01	20.48	10.58	77.27 Rx
	256	60.90	72.33	20.28	98.39	38.80 Rx
16	231	36.98	41.65	10.49	105.74	63.50 Rx
	240	37.10	41.56	10.80	79.16	63.41 Rx
	253	37.17	41.78	10.60	26.59	0.00 Rx
23	244	63.96	79.03	20.09	144.03	84.88 Ry
	245	67.80	83.09	20.60	25.54	169.00 Ry
	246	64.54	79.62	19.89	141.98	84.88 Ry



Project: South Hill 1015 Building

Date: 12/9/24

Job No: 19305.04

By: TVM

Sheet: 2

Client: Benaryoa

Braced Frame Base Plate Reactions - Tension

IBC 2021, ASCE 7-16

Earthquake Load Comparison

Frame	NODE	Controlling Loads					
		EQ*W (k)	(-)Ecl (k)	(+)Ecl (k)	E FND OT (k)	(-) EQ (k)	(+) EQ (k)
1	215	85.8	-64.2	71.3		-85.8 EQ*W	85.8 EQ*W
	225	85.8	-64.2	71.3		-85.8 EQ*W	85.8 EQ*W
2	239	489.3	-381.6	427.4	884.8	-489.3 EQ*W	489.3 EQ*W
	252	489.3	-381.6	427.4	884.8	-489.3 EQ*W	489.3 EQ*W
3	212	345.5	-294.4	328.0	438.2	-345.5 EQ*W	345.5 EQ*W
	224	345.4	-294.4	328.0	438.2	-345.4 EQ*W	345.4 EQ*W
4	238	317.2	-267.9	298.5	360.5	-317.2 EQ*W	317.2 EQ*W
	248	345.6	-267.9	298.5	360.5	-345.6 EQ*W	345.6 EQ*W
7	195	153.4	-131.8	146.3	394.8	-153.4 EQ*W	153.4 EQ*W
	201	153.4	-131.8	146.3	394.8	-153.4 EQ*W	153.4 EQ*W
8	200	147.7	-132.8	147.4	309.1	-147.7 EQ*W	147.7 EQ*W
	208	448.3	-132.8	147.4	309.1	-309.1 E FND OT	309.1 E FND OT
9	234	217.9	-236.0	263.5	424.3	-236.0 (-)Ecl	263.5 (-)Ecl
	245	213.7	-236.0	263.5	424.3	-236.0 (-)Ecl	263.5 (-)Ecl
10	207	469.1	-271.3	303.9	426.0	-426.0 E FND OT	426.0 E FND OT
	220	259.9	-271.3	303.9	426.0	-271.3 (-)Ecl	303.9 (-)Ecl
11	199	217.3	-221.7	248.3	377.2	-221.7 (-)Ecl	248.3 (-)Ecl
	206	217.3	-221.7	248.3	377.2	-221.7 (-)Ecl	248.3 (-)Ecl
12	219	158.5	-125.6	139.4	349.5	-158.5 EQ*W	158.5 EQ*W
	228	158.5	-125.6	139.4	349.5	-158.5 EQ*W	158.5 EQ*W
13	243	206.3	-210.8	236.2	347.9	-210.8 (-)Ecl	236.2 (-)Ecl
	255	206.3	-210.8	236.2	347.9	-210.8 (-)Ecl	236.2 (-)Ecl
14	242	394.0	-292.9	329.3		-394.0 EQ*W	394.0 EQ*W
	254	312.4	-292.9	329.3		-312.4 EQ*W	329.3 (-)Ecl
15	197	378.2	-336.6	373.6	387.9	-378.2 EQ*W	378.2 EQ*W
	203	378.2	-336.6	373.6	387.9	-378.2 EQ*W	378.2 EQ*W
17	202	288.2	-225.5	259.9	303.7	-288.2 EQ*W	288.2 EQ*W
	216	288.2	-225.5	259.9	303.7	-288.2 EQ*W	288.2 EQ*W



Project: South Hill 1015 Building

Date: 12/9/24

Job No: 19305.04

By: TVM

Sheet: 3

Client: Benaryoa

Braced Frame Base Plate Reactions - Tension

IBC 2021, ASCE 7-16

Earthquake Load Comparison

Frame	NODE	Controlling Loads					
		(+)EQ*Ω (k)	(-)Ecl (k)	(+)Ecl (k)	E FND OT (k)	(-) EQ (k)	(+) EQ (k)
18	213	220.3	-233.6	261.2	318.4	-233.6 (-)Ecl	261.2 (-)Ecl
	214	220.3	-233.6	261.2	318.4	-233.6 (-)Ecl	261.2 (-)Ecl
19	250	119.7	-121.7	136.2	370.0	-121.7 (-)Ecl	136.2 (-)Ecl
	251	119.7	-121.7	136.2	370.0	-121.7 (-)Ecl	136.2 (-)Ecl
20	248	345.6	-143.3	159.1		-345.6 EQ*W	345.6 EQ*W
	249	118.6	-143.3	159.1		-143.3 (-)Ecl	159.1 (-)Ecl
21	210	131.2	-124.7	139.7	419.7	-131.2 EQ*W	139.7 (-)Ecl
	211	132.1	-124.7	139.7	419.7	-132.1 EQ*W	139.7 (-)Ecl
22	260	188.5	-176.9	205.6	561.7	-188.5 EQ*W	205.6 (-)Ecl
	261	188.5	-176.9	205.6	561.7	-188.5 EQ*W	205.6 (-)Ecl
24	193	203.8	-177.2	206.0	389.7	-203.8 EQ*W	206.0 (-)Ecl
	194	203.8	-177.2	206.0	389.7	-203.8 EQ*W	206.0 (-)Ecl
25	207	469.1	-404.7	476.3	625.2	-469.1 EQ*W	476.3 (-)Ecl
	208	448.3	-404.7	476.3	625.2	-448.3 EQ*W	476.3 (-)Ecl
26	191	202.2	-176.9	205.6	388.6	-202.2 EQ*W	205.6 (-)Ecl
	192	202.2	-176.9	205.6	388.6	-202.2 EQ*W	205.6 (-)Ecl
27	204	438.2	-444.7	514.8	584.5	-444.7 (-)Ecl	514.8 (-)Ecl
	205	438.2	-444.7	514.8	584.5	-444.7 (-)Ecl	514.8 (-)Ecl
28	241	355.7	-369.2	431.1	461.6	-369.2 (-)Ecl	431.1 (-)Ecl
	242	434.0	-369.2	431.1	461.6	-434.0 EQ*W	434.0 EQ*W
29	258	255.7	-65.6	107.7	331.2	-255.7 EQ*W	255.7 EQ*W
	259	255.7	-65.6	107.7	331.2	-255.7 EQ*W	255.7 EQ*W

Special MultiBay Frames

6	236	272.4	-261.4	299.4	531.9	-272.4 EQ*W	299.4 (-)Ecl
	247	26.5	8.0	64.8		-26.5 EQ*W	64.8 (-)Ecl
	256	246.0	-234.6	269.4	531.9	-246.0 EQ*W	269.4 (-)Ecl
16	231	264.4	-200.6	222.7	749.2	-264.4 EQ*W	264.4 EQ*W
	240	197.9	160.8	-117.6		-197.9 EQ*W	197.9 EQ*W
	253	66.5	-61.9	83.0	749.2	-66.5 EQ*W	83.0 (-)Ecl
23	244	360.1	-354.8	429.5	684.0	-360.1 EQ*W	429.5 (-)Ecl
	245	63.9	76.3	60.3		-63.9 EQ*W	63.9 EQ*W
	246	355.0	-369.2	431.1	684.0	-369.2 (-)Ecl	431.1 (-)Ecl



Project: **South Hill 1015 Building**

Date: **12/9/24**

Job No: **19305.04**

By: **TVM**

Sheet: **4**

Client: **Benaryoa**

Braced Frame Base Plate Reactions - Tension

IBC 2021, ASCE 7-16

Existing Base Plate Anchorage is (6) 1 1/4"Ø AB.s

FTG. ϕT_n Footing = k
 G.B. ϕT_n Grade BM. = 457.9 k
 PL ϕT_n Plinth = 351.6 k

Frame	NODE	Tension Load Case (0.9 - 0.2 Sds) DL + (-)EQ	Anchor Uc (T)	Downward Load Case (1.2 + 0.2 Sds) DL + (+)EQ
1	215	-60.8		135.7
	225	-41.3		174.9
2	239	-433.0	PL 1.23	602.1
	252	-454.2	PL 1.29	559.6
3	212	-314.4	G.B. 0.69	407.6
	224	-296.2	G.B. 0.65	444.0
4	238	-254.0	PL 0.72	443.9
	248	-298.2	PL 0.85	440.7
7	195	-127.4	G.B. 0.28	205.5
	201	-115.0	G.B. 0.25	230.4
8	200	-95.6	G.B. 0.21	252.0
	208	-265.8	G.B. 0.58	396.1
<u>9</u>	<u>234</u>	-185.4		365.0
	<u>245</u>	-188.6		358.5
10	207	-378.9	G.B. 0.83	520.3
	220	-195.0	G.B. 0.43	457.1
<u>11</u>	<u>199</u>	-171.8		348.3
	<u>206</u>	-176.4		339.1
12	219	-113.4	G.B. 0.25	249.0
	228	-113.3	G.B. 0.25	249.3
<u>13</u>	<u>243</u>	-166.3		325.5
	<u>255</u>	-166.1		325.9
14	242	-347.9	PL 0.99	486.3
	254	-267.4	PL 0.76	419.7
15	197	-336.7	G.B. 0.74	461.4
	203	-337.7	G.B. 0.74	459.5
17	202	-264.8	G.B. 0.58	334.9
	216	-261.5	G.B. 0.57	341.6

	Project: South Hill 1015 Building	Date: 12/9/24	Job No: 19305.04
		By: TVM	Sheet: 6
	Client: Benaryoa		

Braced Frame Base Plate Reactions - Tension

IBC 2021, ASCE 7-16

Overstrength Load Combinations

Frame	NODE	Tension Load Case (0.9 - 0.2 Sds) DL + (-)EQ	Anchor Uc (T)	Downward Load Case (1.2 + 0.2 Sds) DL + (+)EQ
18	213	-208.6	G.B. ● 0.46	311.2
	214	-183.8	G.B. ● 0.40	360.9
19	250	-63.2	PL ● 0.18	253.5
	251	-45.9	PL ● 0.13	288.3
20	248	-284.5		440.7
	249	-72.0	PL ● 0.20	302.0
21	210	-91.9	G.B. ● 0.20	218.4
	211	-96.8	G.B. ● 0.21	210.3
22	260	-162.0	PL ● 0.46	258.6
	261	-157.8	PL ● 0.45	267.0
24	193	-170.8	G.B. ● 0.37	272.0
	194	-170.0	G.B. ● 0.37	273.7
25	207	-422.0	G.B. ● 0.92	570.6
	208	-404.9	G.B. ● 0.88	563.2
26	191	-168.7	G.B. ● 0.37	272.8
	192	-169.0	G.B. ● 0.37	272.1
27	204	-386.8	G.B. ● 0.84	631.1
	205	-386.2	G.B. ● 0.84	632.2
28	241	-324.1	PL ● 0.92	521.5
	242	-387.9	PL ● 1.10	526.3
29	258	-226.3	PL ● 0.64	314.7
	259	-223.6	PL ● 0.64	320.1
	0			
	0			
6	236	-221.2		393.0
	247	20.2	PL ● -0.06	150.1
	256	-203.4	PL ● 0.58	321.2
16	231	-238.5	PL ● 0.68	316.3
	240	-172.0	PL ● 0.49	250.0
	253	-40.5	PL	172.6
23	244	-315.4	PL ● 0.90	524.5
	245	-16.5	PL ● 0.05	154.3
	<u>246</u>	-324.1		431.1



Project: South Hill 1015 Building

Date: 12/9/24

Job No: 19305.04

By: TVM

Sheet: 6

Client: Benaryoa

Braced Frame Base Plate Reactions - Shear

IBC 2021, ASCE 7-16

Earthquake Load Comparison			Controlling Loads			
Frame	NODE	EQ*W (k)	(-)Ecl (k)	(+)Ecl (k)	(-) EQ (k)	(+) EQ (k)
1	215					
	225					
2	239	293.6	-229.0	256.5	-293.6 EQ*W	293.6 EQ*W
	252	293.6	-229.0	256.5	-293.6 EQ*W	293.6 EQ*W
3	212	141.5	-108.7	120.7	-141.5 EQ*W	141.5 EQ*W
	224	141.3	-108.7	120.7	-141.3 EQ*W	141.3 EQ*W
4	238	121.5	-92.8	103.0	-121.5 EQ*W	121.5 EQ*W
	248	121.4	-92.8	103.0	-121.4 EQ*W	121.4 EQ*W
7	195	92.1	-79.1	87.8	-92.1 EQ*W	92.1 EQ*W
	201	92.1	-79.1	87.8	-92.1 EQ*W	92.1 EQ*W
8	200	88.6	-79.7	88.5	-88.6 EQ*W	88.6 EQ*W
	208	88.7	-79.7	88.5	-88.7 EQ*W	88.7 EQ*W
9	234	82.7	-87.2	97.7	-87.2 (-)Ecl	97.7 (-)Ecl
	245	82.7	-87.2	97.7	-87.2 (-)Ecl	97.7 (-)Ecl
10	207	88.1	-79.7	88.5	-88.1 EQ*W	88.5 (-)Ecl
	220	88.3	-79.7	88.5	-88.3 EQ*W	88.5 (-)Ecl
11	199	65.3	-63.4	71.1	-65.3 EQ*W	71.1 (-)Ecl
	206	65.3	-63.4	71.1	-65.3 EQ*W	71.1 (-)Ecl
12	219	95.1	0.0	0.0	-95.1 EQ*W	95.1 EQ*W
	228	95.1	0.0	0.0	-95.1 EQ*W	95.1 EQ*W
13	243	65.3	-63.4	71.1	-65.3 EQ*W	71.1 (-)Ecl
	255	65.3	-63.4	71.1	-65.3 EQ*W	71.1 (-)Ecl
14	242	115.6	-92.1	102.2	-115.6 EQ*W	115.6 EQ*W
	254	115.6	-92.1	102.2	-115.6 EQ*W	115.6 EQ*W
15	197	158.9	-120.4	133.6	-158.9 EQ*W	158.9 EQ*W
	203	158.9	-120.4	133.6	-158.9 EQ*W	158.9 EQ*W
17	202	144.9	-106.2	117.9	-144.9 EQ*W	144.9 EQ*W
	216	144.9	-106.2	117.9	-144.9 EQ*W	144.9 EQ*W

	Project: South Hill 1015 Building	Date: 12/9/24	Job No: 19305.04
		By: TVM	Sheet: 7
	Client: Benaryoa		

Braced Frame Base Plate Reactions - Shear

IBC 2021, ASCE 7-16

Earthquake Load Comparison

Frame	NODE	Controlling Loads				
		(+)EQ* Ω (k)	(-)Ecl (k)	(+)Ecl (k)	(-) EQ (k)	(+) EQ (k)
18	213	79.9	-79.7	88.5	-79.9 EQ*W	88.5 (-)Ecl
	214	79.7	-79.7	88.5	-79.7 (-)Ecl	88.5 (-)Ecl
19	250	71.9	-73.0	81.8	-73.0 (-)Ecl	81.8 (-)Ecl
	251	71.9	-73.0	81.8	-73.0 (-)Ecl	81.8 (-)Ecl
20	248					
	249					
21	210	101.4	-95.8	107.3	-101.4 EQ*W	107.3 (-)Ecl
	211	101.4	-95.8	107.3	-101.4 EQ*W	107.3 (-)Ecl
22	260	107.2	-102.2	113.4	-107.2 EQ*W	113.4 (-)Ecl
	261	107.2	-102.2	113.4	-107.2 EQ*W	113.4 (-)Ecl
24	193	114.4	-102.2	113.4	-114.4 EQ*W	114.4 EQ*W
	194	114.4	-102.2	113.4	-114.4 EQ*W	114.4 EQ*W
25	207	216.7	-213.5	251.9	-216.7 EQ*W	251.9 (-)Ecl
	208	216.8	-213.5	251.9	-216.8 EQ*W	251.9 (-)Ecl
26	191	114.4	-102.2	113.4	-114.4 EQ*W	114.4 EQ*W
	192	114.4	-102.2	113.4	-114.4 EQ*W	114.4 EQ*W
27	204	216.6	-213.5	251.9	-216.6 EQ*W	251.9 (-)Ecl
	205	216.6	-213.5	251.9	-216.6 EQ*W	251.9 (-)Ecl
28	241	211.9	-213.5	251.9	-213.5 (-)Ecl	251.9 (-)Ecl
	242	211.7	-213.5	251.9	-213.5 (-)Ecl	251.9 (-)Ecl
29	258	140.1	-127.7	141.8	-140.1 EQ*W	141.8 (-)Ecl
	259	138.1	-127.7	141.8	-138.1 EQ*W	141.8 (-)Ecl

Special MultiBay Frames

6	236	97.0	-80.3	94.0	-97.0 EQ*W	97.0 EQ*W
	247	193.2	-160.6	187.9	-193.2 EQ*W	193.2 EQ*W
	256	97.0	-80.3	94.0	-97.0 EQ*W	97.0 EQ*W
16	231	158.8	-120.4	133.6	-158.8 EQ*W	158.8 EQ*W
	240	158.5	-120.4	133.6	-158.5 EQ*W	158.5 EQ*W
	244					
23	244	212.2	-213.5	251.9	-213.5 (-)Ecl	251.9 (-)Ecl
	245	422.5	-427.0	503.9	-427.0 (-)Ecl	503.9 (-)Ecl
	246	212.2	-213.5	251.9	-213.5 (-)Ecl	251.9 (-)Ecl



Project: **South Hill 1015 Building**

Date: **12/9/24**

Job No: **19305.04**

By: **TVM**

Sheet: **8**

Client: **Benaryoa**

Braced Frame Base Plate Reactions - Shear & Tension

IBC 2021, ASCE 7-16

Existing Base Plate Anchorage is (6) 1 1/4" Ø AB.s

G.B. ϕV_n Grade BM. = 279.0 k

PL ϕV_n Plinth = 189.0 k

Earthquake Load Comparison

Frame	NODE	Anchor Unity Uc (T)	Uc (-V)	Uc (+V)	Combined Uc Parabolic (-V ^{5/3} +T ^{5/3})		Notes:
1	215						OK, see BP1 Calc
	225						OK, see BP1 Calc
2	239	● 1.23	● 1.55	● 1.55	● 3.50	NG	Jacket and Cap
	252	● 1.29	● 1.55	● 1.55	● 3.62	NG	Jacket and Cap
3	212	● 0.69	● 0.51	● 0.51	● 0.86	OK	
	224	● 0.65	● 0.51	● 0.51	● 0.81	OK	
4	238	● 0.72	● 0.64	● 0.64	● 1.06	NG	PL Reinforce Shear
	248	● 0.85	● 0.64	● 0.64	● 1.24	NG	PL Reinforce Shear
7	195	● 0.28	● 0.33	● 0.33	● 0.28	OK	
	201	● 0.25	● 0.33	● 0.33	● 0.26	OK	
8	200	● 0.21	● 0.32	● 0.32	● 0.22	OK	
	208	● 0.58	● 0.32	● 0.32	● 0.55	OK	
<u>9</u>	<u>234</u>						New Base Plate
	<u>245</u>						New Base Plate
10	207	● 0.83	● 0.32	● 0.32	● 0.88	OK	See Frame 25
	220	● 0.43	● 0.32	● 0.32	● 0.39	OK	
<u>11</u>	<u>199</u>						New Base Plate
	<u>206</u>						New Base Plate
12	219	● 0.25	● 0.34	● 0.34	● 0.26	OK	
	228	● 0.25	● 0.34	● 0.34	● 0.26	OK	
<u>13</u>	<u>243</u>						New Base Plate
	<u>255</u>						New Base Plate
14	242	● 0.99	● 0.61	● 0.61	● 1.42	NG	Jacket Plinth
	254	● 0.76	● 0.61	● 0.61	● 1.07	NG	PL Reinforce Shear
15	197	● 0.74	● 0.57	● 0.57	● 0.99	OK	
	203	● 0.74	● 0.57	● 0.57	● 0.99	OK	
17	202	● 0.58	● 0.52	● 0.52	● 0.74	OK	
	216	● 0.57	● 0.52	● 0.52	● 0.73	OK	

	Project: 015 Building	Date: 12/9/24	Job No: 19305.04
		By: TVM	Sheet: 9
	Client: Benaryoa		

Braced Frame Base Plate Reactions - Shear & Tension

IBC 2021, ASCE 7-16

Earthquake Load Comparison

Frame	NODE	Anchor Unity			Combined Uc Parabolic	
		Uc (T)	Uc (-V)	Uc (+V)	$(-V^{5/3}+T^{5/3})$	
18	213	0.46	0.29	0.32	0.39	OK
	214	0.40	0.29	0.32	0.34	OK
19	250	0.18	0.39	0.43	0.26	OK
	251	0.13	0.39	0.43	0.24	OK
20	248					OK, see BP1 Calc
	249	0.20				
21	210	0.20	0.36	0.38	0.25	OK
	211	0.21	0.36	0.38	0.26	OK
22	260	0.46	0.57	0.60	0.66	OK
	261	0.45	0.57	0.60	0.65	OK
24	193	0.37	0.41	0.41	0.42	OK
	194	0.37	0.41	0.41	0.42	OK
25	207	0.92	0.78	0.90	1.53	NG
	208	0.88	0.78	0.90	1.47	NG
26	191	0.37	0.41	0.41	0.42	OK
	192	0.37	0.41	0.41	0.42	OK
27	204	0.84	0.78	0.90	1.41	NG
	205	0.84	0.78	0.90	1.41	NG
28	241	0.92	1.13	1.33	2.10	NG
	242	1.10	1.13	1.33	2.40	NG
29	258	0.64	0.74	0.75	1.09	NG
	259	0.64	0.73	0.75	1.06	NG

Special MultiBay Frames

6	236					
	247	-0.06	1.02	1.02	1.05	NG
16	256	0.58	0.51	0.51	0.73	OK
	231	0.68	0.84	0.84	1.27	NG
	240	0.49	0.84	0.84	1.05	NG
23	244					
	244	0.90	1.13	1.33	2.06	NG
	245	0.05	2.26	2.67	3.90	NG
	246					

Epoxy new bars.

PL Reinforce Shear

PL Reinforce Shear

OK, see BP1 Calc

PL Reinforce Shear

Jacket Plinth

New Base Plate



Project: South Hill 1015 Building

Date: 12/9/24

Job No: 19305.04

By: TVM

Sheet: 10

Client: Benaryoya

Node 242 Analysis - Base Plate Anchorage

Frame 14/28

	<u>Rx</u>	<u>Ry</u>	<u>Rz</u>		<u>Rx</u>	<u>Ry</u>	<u>Rz</u>	<u>Tu</u>
D	0.74	-10.19	65.67					(0.9 - 0.2 Sds) DL + (-)EQ
Lp	1.11	-18.44	80.82					
Sp	-0.03	-0.13	19.79					
E26	-12.30	79.42	-172.97	<u>QE26</u>	-30.75	198.55	-432.43	-386.53
				<u>Ecl Y (0.3X)</u>	-27.62	-213.51	-457.09	-411.20
E31	46.05	-18.55	-168.69	<u>QE31</u>	115.13	-46.38	-421.73	-375.83
				<u>Ecl X (0.3Y)</u>	-92.05	-64.05	-403.66	-357.77

ϕT_n Plinth = 351.6 k
 ϕV_n Plinth = 189.0 k

Add a jacket around the existing plinth to add more rebar for anchorage development

Plinth is 44"x44". Add an 8" thick jacket. Plinth becomes 60"x60" 5 ksi concrete for lower development lengths
Ldh #6 is 9" and #4 is 6".

Drill and epoxy #4 hooked dowels into side of existing plinth to extend breakout cone to new jacket and develop bar.

11" edge distance from anchor to face of existing plinth (9.5" for breakout calc with washer.)

Cone vertical rise = $\sin(35^\circ) * 9.5" = 5.45$ inch

Cone depth at edge = 42" - 5.5" = 36.5" > 33" #6 jacket bars are fully developed.

Anchor capacity = 534.2 kips

Plinth capacity = 351.6 kips

Jacket Tension = 182.5 kips

Use (1) #6 at each corner and (3) at each side. Drill and epoxy 20" for full development with existing #8 plinths
#4 bar shear capacity = 4.2 kips.

of #4 required = 43 (11) per side. Use (3) rows of (4)

Anchor Unity

Uc (T) Uc (Vx) Combined

0.72 0.15 0.62

Develop Vy Concrete Cap

Node 245 Analysis - Base Plate Anchorage

Frame 23

	<u>Rx</u>	<u>Ry</u>	<u>Rz</u>	<u>Rx</u>	<u>Rz</u>	<u>Tu</u>
D	-0.90	0.03	67.58			(0.9 - 0.2 Sds) DL + (-)EQ
Lp	-1.08	-0.13	82.56			
Sp	-0.25	0.16	20.55			
E10	-9.65	159.67	-23.33	<u>QE13</u>	399.18	-58.33 -11.10

Ecl

503.90 76.27 123.50

Anchor Bolt Shear Capacity (Ignoring breako

279.0 k

Concrete Cap Capacity Required =

224.93

Similar to frame 25 load (233 kips)

Add a concrete cap and a jacket for more shear capacity.

60"x68" concrete shear capacity = $2 * \sqrt{f'c} * 0.7$ 162.7 k

Shear Steel Capacity

615.0 k

777.7 k

Node 252 Analysis

Frame 2

	<u>Rx</u>	<u>Rz</u>	<u>Rx</u>	<u>Rz</u>	<u>Tu</u>
D	-6.44	52.59			(0.9 - 0.2 Sds) DL + (-)EQ
Lp	-5.88	43.54			
Sp	-2.08	4.19			
E13	-116.4	-193.97	<u>QE13</u>	-291.00	-448.18
			<u>Ecl</u>	-228.98	-344.82
				-381.57	

Anchor capacity =

457.9 kips

Plinth capacity =

351.6 kips

Jacket Tension =

106.2 kips

Non-Prestressed Punching Shear Analysis

ACI 318-19

Normal Weight Concrete

*Existing frame column base plates are 3/4"x6"x6". A two way punching shear analysis is OK to allow for ignoring concrete breakout per engineering judgement.

1.) Input

Column Width	16 in	Note: Edge is parallel to width at edge column condition.
Column Length	20 in	
Location	Interior	
Edge Distance	in (N/A at Interior Columns)	
Thickness	36 in	
Effective Depth	28 in	
Concrete Strength f'_c	3000 psi	
Column Load P_u	457.9 kips = Tension Capacity of Existing Anchors	

2.) Punching Shear

$\lambda_s =$	0.73	
$\alpha_s =$	40	Sec 22.6.5.3
$\beta =$	1.25	Sec 22.6.5.2
$b_o =$	184 in	Sec 22.6.4.3
$\phi =$	0.75	Table 21.2.1
$V_{c_a} = 4*f'_c^{1/2} =$	159 psi	Table 22.6.5.2
$V_{c_b} = (2+4/\beta)*f'_c^{1/2} =$	207 psi	Table 22.6.5.2
$V_{c_c} = [2+(\alpha_s*d/b_o)]*f'_c^{1/2} =$	321 psi	Table 22.6.5.2
Controlling $V_c =$	159 psi	
$\phi V_c =$	119 psi	
$\phi P_n = \phi V_c * d * b_o =$	614 kips	$> P_u = 457.9$ kips OK
$\phi 2\lambda_s f'_c^{1/2} =$	59.604 psi	$< P_u = 88.88$ psi
$A_{s,min} = 5*v_{uv}*b_{slab}*b_o / \phi*\alpha_s*f_y =$	2.453 sqin	4.1 #7 bars



Quantum Consulting Engineers LLC
1511 Third Avenue, Suite 323
Seattle, WA 98101

Project: 1015 Building

Date: 12/9/24 Job No: 19305.04

Designer: TVM Sheet: 1

Client: Benaroya

Checked:



Quantum Consulting Engineers LLC
1511 Third Ave
Suite 323
Seattle, WA 98105
206-957-3900

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Base Plate by FEM

Project File: Foundations.ec6

LIC# : KW-06016450, Build:20.24.05.02

QUANTUM CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2023

DESCRIPTION: Braced Frame Base Plate

Code References

Calculations per AISC Design Guide # 1, IBC 2021, CBC 2019, ASCE 7-16, AISC 360-16

Load Combination Set : IBC 2021

General Information

Material Properties

AISC Design Method

Steel Plate Fy = 36 ksi

Concrete Support f_c = 3 ksi

ϕ_c : LRFD Factor

0.650

Nominal Bearing Fp per J8

ksi

Column, Plate & Pedestal

Column Properties

Steel Section W12x65

Depth 12.1 in Area 19.1 in²

Width 12 in I_{xx} 533 in⁴

Flange Thickness 0.605 in I_{yy} 174 in⁴

Web Thickness 0.39 in

Column Rotation & Offset

Column Rotation 0 deg

Offset from "X" Plate Center in

Offset from "Z" Plate Center in

Plate Dimensions

N : Length 22.0 in

B : Width 18.0 in

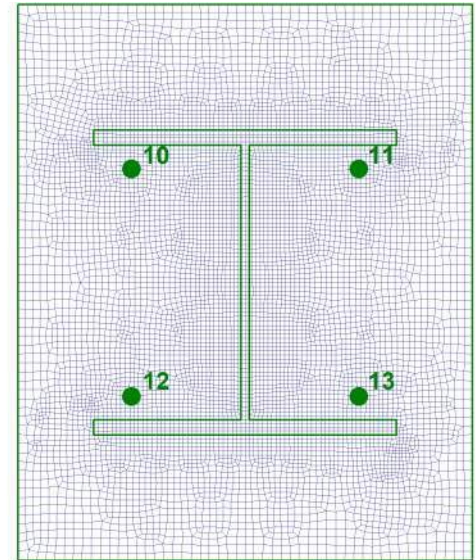
Thickness 1.50 in

Support Dimensions

Width along "X" 44.0 in

Length along "Z" 36.0 in

Column fully welded to base plate.

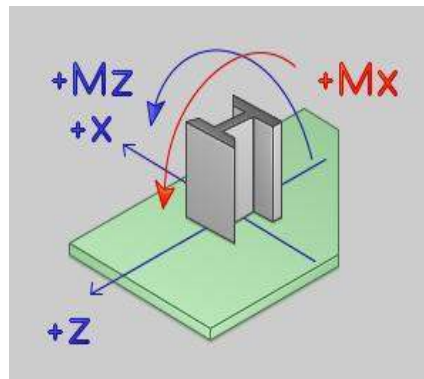
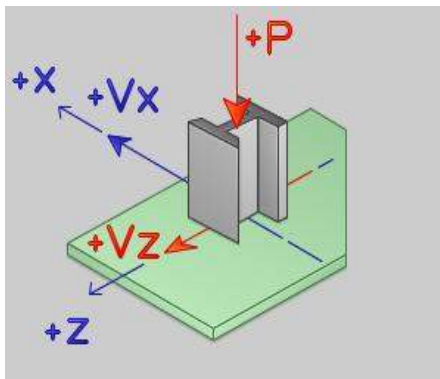


Applied Loads

	P-Y	V-X	V-Z	M-X	M-Z
D : Dead Load	78.750 k	k	k	k-ft	k-ft
L : Live	48.810 k	k	k	k-ft	k-ft
Lr : Roof Live	k	k	k	k-ft	k-ft
S : Snow	36.650 k	k	k	k-ft	k-ft
W : Wind	k	k	k	k-ft	k-ft
E : Earthquake	489.30 k	k	k	k-ft	k-ft
H : Lateral Earth	k	k	k	k-ft	k-ft

" P " = Gravity load, "+" sign is downward. Moments create higher soil pressure at +Z edge

"+" Shears push plate towards +Z edge





Quantum Consulting Engineers LLC
1511 Third Ave
Suite 323
Seattle, WA 98105
206-957-3900

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Base Plate by FEM

Project File: Foundations.ec6

LIC# : KW-06016450, Build:20.24.05.02

QUANTUM CONSULTING ENGINEERS

(c) ENERCALC INC 1983-2023

DESCRIPTION: Braced Frame Base Plate

Governing Conditions

Design Method

Governing Stress Ratio

1.085

Design Plate Size

Bending Stress Ratio

1.085

fb : Max. Bending Stress

35.2 ksi

Fb : Allowable

32.4 ksi

Location from Plate Center:

X: -6.207 in Z: 5.517 in

Plate Bearing Stress Ratio

0.792

fu : Max. Bearing Stress

2.62 ksi

Fp : Allowable

3.32 ksi

Location from Plate Center:

X: 5.912 in Z: 5.515 in

Bolt Tension Stress Ratio

0.000

+0.6988D+E

Maximum Bolt Tension

0.00 k

Allowable Tension/Pullout

k

Location from Plate Center:

X: 4.500 in Z: -4.500 in

< 36 KSI Yeild Stress. OK per
engineering judgment for
amplified load.



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Version 3.3.2410.2

Company:		Date:	9/8/2022
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Address:			
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E-mail:			

1. Project information

Project description: Frame 9, 11, & 13 base plate design

Location:

Design name: Design

Comment:

2. Input Data & Anchor Parameters

General

Design method: ACI 318-19

Units: Imperial units

Anchor Information:

Anchor type: Cast-in-place

Material: F1554 Grade 36

Diameter (inch): 1.250

Effective Embedment depth, h_{ef} (inch): 30.000

Anchor category: -

Anchor ductility: Yes

h_{min} (inch): 32.00

c_{min} (inch): 7.50

s_{min} (inch): 7.50

Base Material

Concrete: Normal-weight

Concrete thickness, h (inch): 13.78

State: Uncracked

Compressive strength, f'_c (psi): 3000

Reinforcement condition: A tension, A shear

Supplemental edge reinforcement: Yes

Reinforcement provided at corners: Yes

Ignore concrete breakout in tension: No

Ignore concrete breakout in shear: No

Ignore ϕ_{do} requirement: No

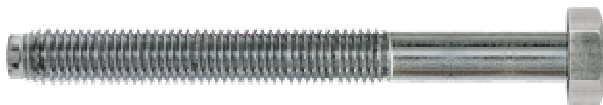
Build-up grout pad: Yes

Base Plate

Length x Width x Thickness (inch): 30.00 x 14.00 x 0.25

Recommended Anchor

Anchor Name: Heavy Hex Bolt - 1 1/4"Ø Heavy Hex Bolt, F1554 Gr. 36





Company:		Date:	9/8/2022
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Address:			
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Load and Geometry

Load factor source: ACI 318 Section 5.3

Load combination: not set

Seismic design: Yes

Anchors subjected to sustained tension: Not applicable

Ductility section for tension: 17.10.5.3 (d) is satisfied

Ductility section for shear: 17.10.6.3 (c) is satisfied

Ω_0 factor: not set

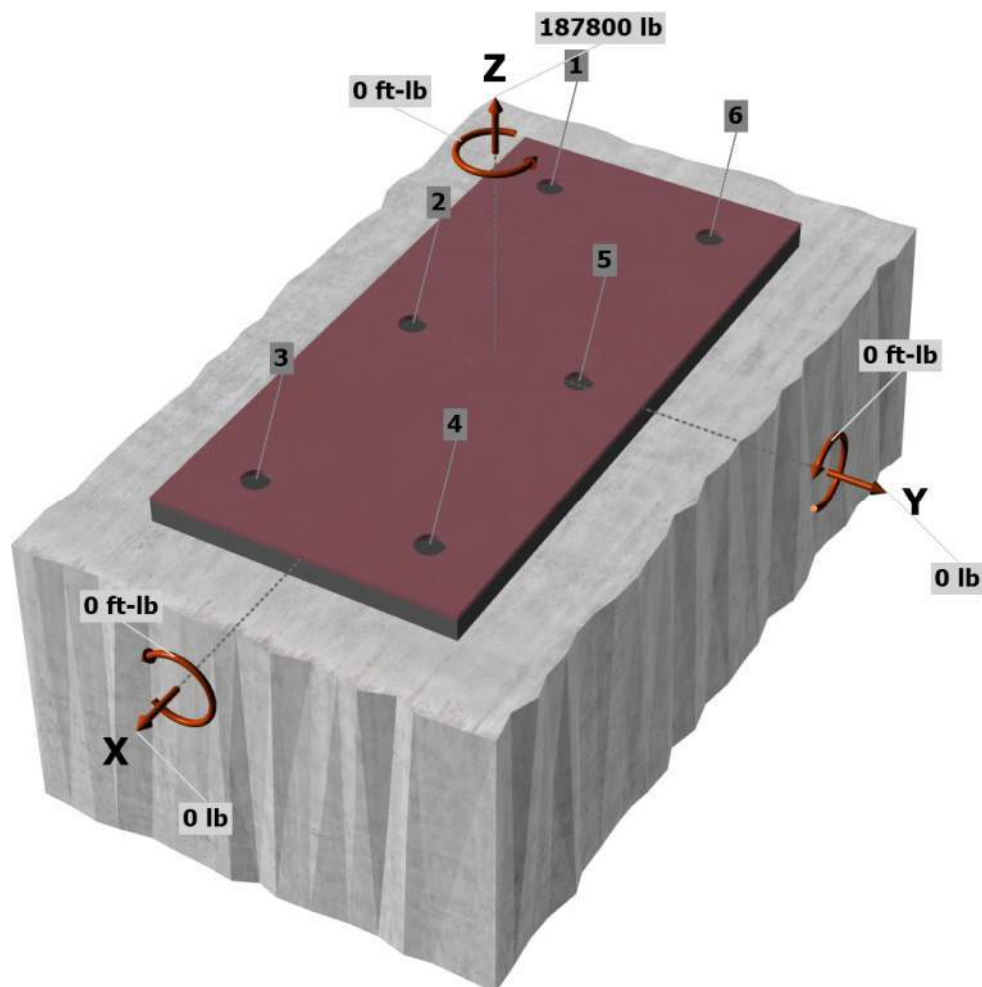
Apply entire shear load at front row: No

Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 187800
 V_{uax} [lb]: 0
 V_{uay} [lb]: 0
 M_{ux} [ft-lb]: 0
 M_{uy} [ft-lb]: 0
 M_{uz} [ft-lb]: 0

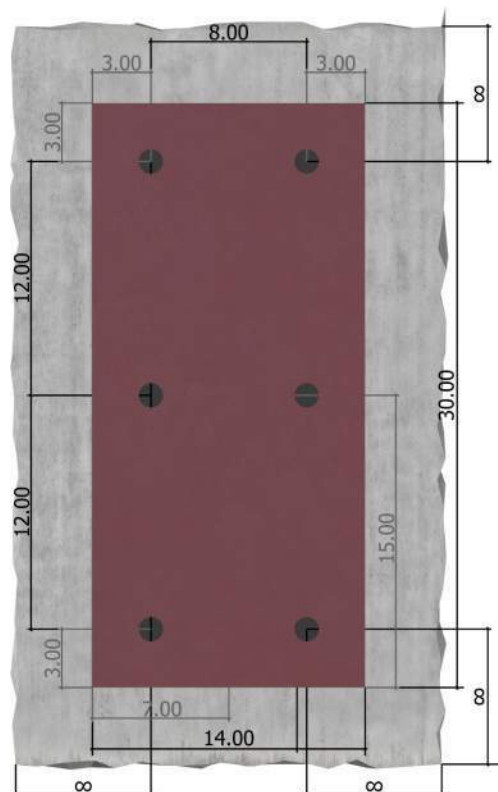
<Figure 1>





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E-mail:			

<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2 + (V_{uay})^2}$ (lb)
1	31300.0	0.0	0.0	0.0
2	31300.0	0.0	0.0	0.0
3	31300.0	0.0	0.0	0.0
4	31300.0	0.0	0.0	0.0
5	31300.0	0.0	0.0	0.0
6	31300.0	0.0	0.0	0.0
Sum	187800.0	0.0	0.0	0.0

Maximum concrete compression strain (%): 0.00

Maximum concrete compression stress (psi): 0

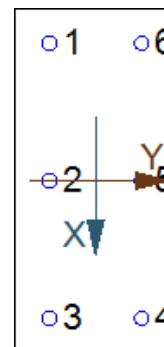
Resultant tension force (lb): 187800

Resultant compression force (lb): 0

Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00

Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>





Company:		Date:	9/8/2022
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4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
56200	0.75	42150

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} \text{ (Eq. 17.6.2.2.1)}$$

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	3000	30.000	216000

$$0.75\phi N_{cbg} = 0.75\phi (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \text{ (Sec. 17.5.1.2 \& Eq. 17.6.2.1a)}$$

A_{Nc} (in ²)	A_{Nco} (in ²)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cbg}$ (lb)
11172.00	8100.00	-	1.000	1.000	1.25	1.000	216000	0.75	209475

6. Pullout Strength of Anchor in Tension (Sec. 17.6.3)

$$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8A_{brg} f'_c \text{ (Sec. 17.5.1.2, Eq. 17.6.3.1 \& 17.6.3.2.2a)}$$

$\Psi_{c,P}$	A_{brg} (in ²)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.4	2.24	3000	0.70	39461



Company:		Date:	9/8/2022
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E-mail:			

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	31300	42150	0.74	Pass
Concrete breakout	187800	209475	0.90	Pass (Governs)
Pullout	31300	39461	0.79	Pass

1 1/4"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 30.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.
- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.
- Designer must exercise own judgement to determine if this design is suitable.



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Company:		Date:	9/8/2022
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Address:			
Phone:			
E-mail:			

1. Project information

Project description: Frame 6 Base Plate (Rebar Developed by plinth Comment:
reinf, concrete breakout does not occur.)

Location:

Design name: Design

2. Input Data & Anchor Parameters

General

Design method: ACI 318-19

Units: Imperial units

Anchor Information:

Anchor type: Cast-in-place

Material: F1554 Grade 36

Diameter (inch): 1.250

Effective Embedment depth, h_{ef} (inch): 30.000

Anchor category: -

Anchor ductility: Yes

h_{min} (inch): 32.00

C_{min} (inch): 7.50

S_{min} (inch): 7.50

Base Material

Concrete: Normal-weight

Concrete thickness, h (inch): 13.78

State: Uncracked

Compressive strength, f'_c (psi): 3000

Reinforcement condition: A tension, A shear

Supplemental edge reinforcement: Yes

Reinforcement provided at corners: Yes

Ignore concrete breakout in tension: Yes

Ignore concrete breakout in shear: No

Ignore 6d requirement: No

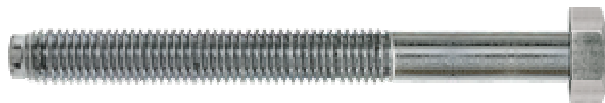
Build-up grout pad: Yes

Base Plate

Length x Width x Thickness (inch): 30.00 x 14.00 x 0.25

Recommended Anchor

Anchor Name: Heavy Hex Bolt - 1 1/4"Ø Heavy Hex Bolt, F1554 Gr. 36





Company:		Date:	9/8/2022
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Address:			
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E-mail:			

Load and Geometry

Load factor source: ACI 318 Section 5.3

Load combination: not set

Seismic design: Yes

Anchors subjected to sustained tension: Not applicable

Ductility section for tension: 17.10.5.3 (d) is satisfied

Ductility section for shear: 17.10.6.3 (c) is satisfied

Ω_0 factor: not set

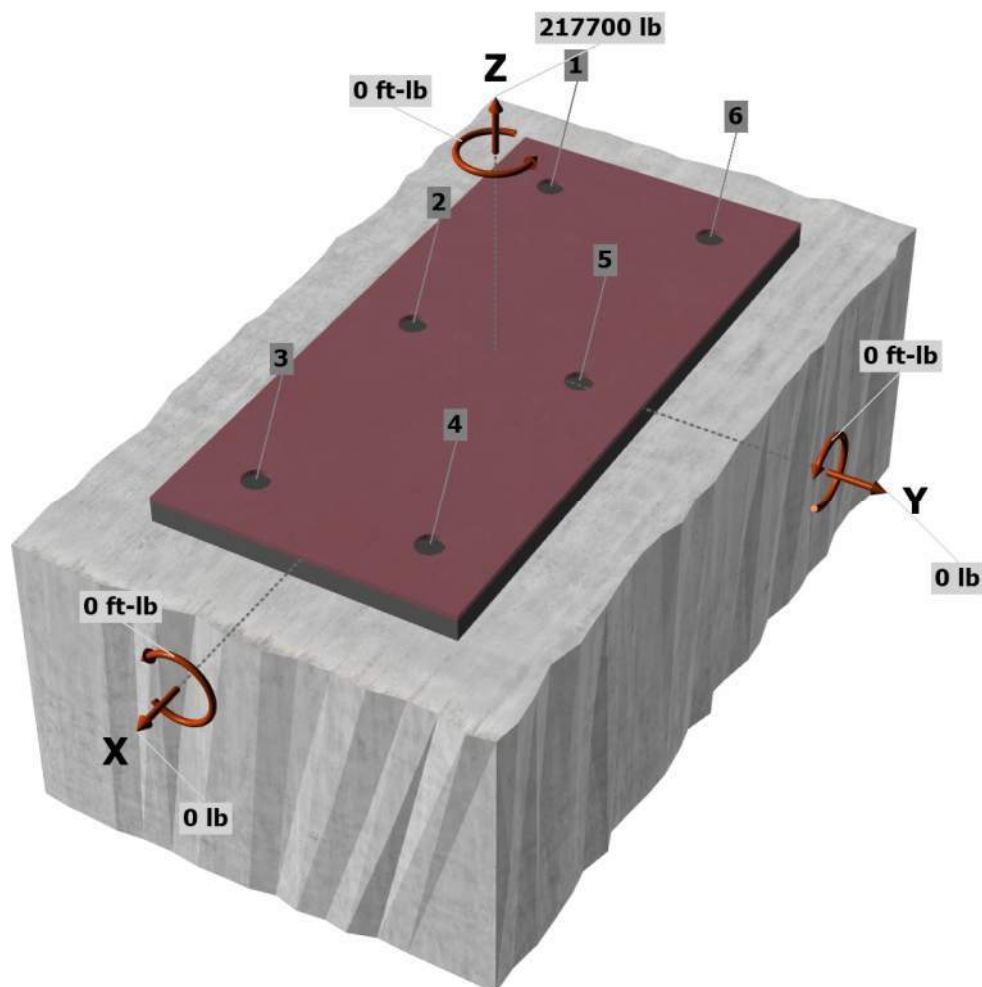
Apply entire shear load at front row: No

Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 217700
 V_{uax} [lb]: 0
 V_{uay} [lb]: 0
 M_{ux} [ft-lb]: 0
 M_{uy} [ft-lb]: 0
 M_{uz} [ft-lb]: 0

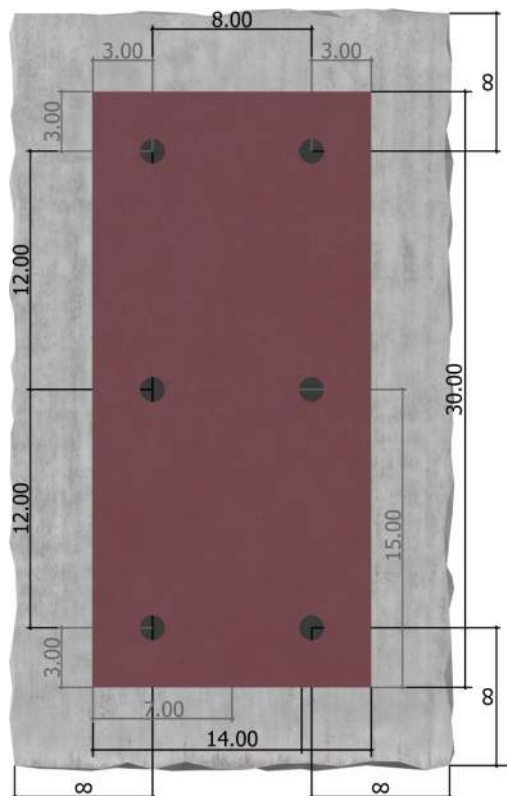
<Figure 1>





Company:		Date:	9/8/2022
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E-mail:			

<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2 + (V_{uay})^2}$ (lb)
1	36283.3	0.0	0.0	0.0
2	36283.3	0.0	0.0	0.0
3	36283.3	0.0	0.0	0.0
4	36283.3	0.0	0.0	0.0
5	36283.3	0.0	0.0	0.0
6	36283.3	0.0	0.0	0.0
Sum	217700.0	0.0	0.0	0.0

Maximum concrete compression strain (%): 0.00

Maximum concrete compression stress (psi): 0

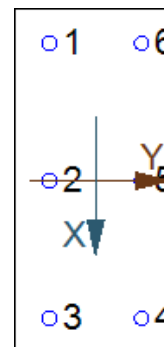
Resultant tension force (lb): 217700

Resultant compression force (lb): 0

Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00

Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>





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4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
56200	0.75	42150

6. Pullout Strength of Anchor in Tension (Sec. 17.6.3)

$0.75\phi N_{pn} = 0.75\phi\psi_{c,P}N_p = 0.75\phi\psi_{c,P}8A_{brg}f'_c$ (Sec. 17.5.1.2, Eq. 17.6.3.1 & 17.6.3.2.2a)

$\psi_{c,P}$	A_{brg} (in ²)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.4	2.24	3000	0.70	39461



Company:		Date:	9/8/2022
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Project:			
Address:			
Phone:			
E-mail:			

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	36283	42150	0.86	Pass
Pullout	36283	39461	0.92	Pass (Governs)

1 1/4"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 30.000 inch meets the selected design criteria.

12. Warnings

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.5.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

Base Plate Shear Lug Design

Per IBC 2021 and AISC 360-16

Frame: 9
Node: 245

Vu Req'd: 87.2 k
Pu: -188.6 k
n: 6.00
Nsa: 64.8 k
nNsa: 389.1 k
Base Plate Area: 420.0 sqin

Shear Lug Bearing on Concrete:

Grout Below Column, tg: 1.5 in
 $\psi_{brg,sl} = 0.52$ ACI EQ 17.11.2.2.1a & c
f'c: 3 ksi
Shear Lug Width, b: 14 in
Shear Lug Depth, d: 3.0 in Below Concrete
of Shear Lugs: 2
Lug Thickness, t: 1.0 in
of Stiffeners: 0
Stiffener Thickness: 0.50 in
A_eff: 28 sq in ACI Sec. 17.11.2.1.1
Vm: 147.2 k $1.7 * f'c * A_{eff} * \psi_{brg,sl}$ ACI EQ 17.11.2.1
 ϕ : 0.65
 ϕVn : 95.7 k ACI EQ 17.11.2.1
DCR: 0.91 OK

Shear Lug Bending:

Vu: 43.6 k per lug
d: 4.5 in
d_eff: 2.0 in
b: 14 in
Mu: 87 k-in
Z: 3.5 in³ (b*t²/4)
Fy: 36 ksi
Mn: 126.0 k-in
 ϕ : 0.9
 ϕMn : 113.4 k-in
DCR: 0.77 OK

	Quantum Consulting Engineers LLC	Project: South Hill 1015	Date: 12/9/24	Job No: 19305.04
	1511 Third Avenue, Suite 323	Designer: TVM	Sheet:	
	Seattle, WA 98101	Client: Benaroya	Checked:	

Base Plate Shear Lug Design

Per IBC 2021, ACI318-19, and AISC 360-16

Shear Lug Weld to Baseplate:

Mu: **87 k-in**
b: 14 in
s: 1.21 in
fb: 5.2 k/in
fv: 3.1 k/in
fr: **6.0 k/in**
Weld Strength: 70 ksi
Fillet Weld Size: **0.313**
 ϕ : 0.75
 Θ : 90 degrees
 ϕ Rn: **10.4 k/in** (AISC Eq 8-1)
DCR: **0.58** OK

Base Plate Bending:

Mu: **55 k-in**
d_eff: 2.5 in
b: **14.0 in**
t: **1.5 in**
Z: 7.9 in³ ($b \cdot t^2 / 4$)
Fy: **36 ksi**
Mn: 283.5 k-in
 ϕ : 0.9
 ϕ Mn: **255.2 k-in**
DCR: **0.21** OK OK By Inspection



Quantum Consulting Engineers LLC
1511 Third Avenue, Suite 323
Seattle, WA 98101

Project:	South Hill 1015	Date:	12/9/24	Job No:	19305.04
		Designer:	TVM	Sheet:	
Client:	Benaroya	Checked:			

Base Plate Shear Lug Design

Per IBC 2021, ACI318-19, and AISC 360-16

Frame: 23
Node: 244

Vu Req'd: 214.0 k
Pu: -315.4 k
n: 6.00
Nsa: 101.7 k
nNsa: 610.5 k
Base Plate Area: 784.0 sqin

Shear Lug Bearing on Concrete:

Grout Below Column, tg: 1.0 in
 $\psi_{brg,sl} = 0.48$ ACI EQ 17.11.2.2.1a & c
f'c: 3 ksi
Shear Lug Width, b: 30 in
Shear Lug Depth, d: 6.0 in Below Concrete
of Shear Lugs: 1
Lug Thickness, t: 1.5 in
of Stiffeners: 3
Stiffener Thickness: 0.75 in
A_eff: 151 sq in ACI Sec. 17.11.2.1.1
Vm: 371.6 k $1.7 * f'c * A_{eff} * \psi_{brg,sl}$ ACI EQ 17.11.2.1
 ϕ : 0.65
 ϕVn : 241.5 k ACI EQ 17.11.2.1
DCR: 0.89 OK

Shear Lug Bending:

Vu: 214.0 k per lug
d: 7.0 in
d_eff: 1.8 in in
b: 30 in
Mu: 375 k-in
Z: 16.9 in³ (b*t²/4)
Fy: 36 ksi
Mn: 607.5 k-in
 ϕ : 0.9
 ϕMn : 546.8 k-in
DCR: 0.68 OK

	Quantum Consulting Engineers LLC	Project: South Hill 1015	Date: 12/9/24	Job No: 19305.04
	1511 Third Avenue, Suite 323	Designer: TVM	Sheet:	
	Seattle, WA 98101	Client: Benaroya	Checked:	

Base Plate Shear Lug Design

Per IBC 2021, ACI318-19, and AISC 360-16

Shear Lug Weld to Baseplate:

Mu: **375 k-in**
b: 30 in
s: 1.71 in
fb: 7.3 k/in
fv: 3.6 k/in
fr: **8.1 k/in**
Weld Strength: 70 ksi
Fillet Weld Size: **0.313**
 ϕ : 0.75
 Θ : 90 degrees
 ϕ Rn: **10.4 k/in** (AISC Eq 8-1)
DCR: **0.78** OK

Base Plate Bending:

d_eff: 3.5 in
b: **30.0 in**
t: **1.5 in**
Mu: **376 k-in**
Z: 16.9 in³ ($b \cdot t^2 / 4$)
Fy: **36 ksi**
Mn: 607.5 k-in
 ϕ : 0.9
 ϕ Mn: **546.8 k-in**
DCR: **0.69** OK
Z_req'd: 11.6 in³

Make the base plate wider



Quantum Consulting Engineers LLC
1511 Third Avenue, Suite 323
Seattle, WA 98101

Project:	South Hill 1015	Date:	12/9/24	Job No:	19305.04
		Designer:	TVM	Sheet:	
Client:	Benaroya	Checked:			

Base Plate Shear Lug Design

Per IBC 2021, ACI318-19, and AISC 360-16

Frame: 28
Node: 241

Vu Req'd: 214.0 k
Pu: -324.1 k
n: 6.00
Nsa: 101.7 k
nNsa: 610.5 k
Base Plate Area: 784.0 sqin

Shear Lug Bearing on Concrete:

Grout Below Column, tg: 1.5 in
 $\psi_{brg,sl} = 0.47$ ACI EQ 17.11.2.2.1a & c
f'c: 3 ksi
Shear Lug Width, b: 30 in
Shear Lug Depth, d: 6.0 in Below Concrete
of Shear Lugs: 1
Lug Thickness, t: 1.5 in
of Stiffeners: 3
Stiffener Thickness: 0.75 in
A_eff: 151 sq in ACI Sec. 17.11.2.1.1
Vm: 360.7 k $1.7 * f'c * A_{eff} * \psi_{brg,sl}$ ACI EQ 17.11.2.1
 ϕ : 0.65
 ϕVn : 234.4 k ACI EQ 17.11.2.1
DCR: 0.91 OK

Shear Lug Bending:

Vu: 214.0 k per lug
d: 7.5 in
d_eff: 2.3 in in
b: 30 in
Mu: 482 k-in
Z: 16.9 in³ (b*t²/4)
Fy: 36 ksi
Mn: 607.5 k-in
 ϕ : 0.9
 ϕMn : 546.8 k-in
DCR: 0.88 OK



Quantum Consulting Engineers LLC
1511 Third Avenue, Suite 323
Seattle, WA 98101

Project: South Hill 1015	Date: 12/9/24	Job No: 19305.04
Designer: TVM	Sheet:	
Client: Benaroya	Checked:	

Base Plate Shear Lug Design

Per IBC 2021, ACI318-19, and AISC 360-16

Shear Lug Weld to Baseplate:

Mu: **482 k-in**
b: 30 in
s: 1.71 in
fb: 9.4 k/in
fv: 3.6 k/in
fr: **10.0 k/in**
Weld Strength: 70 ksi
Fillet Weld Size: **0.313**
 ϕ : 0.75
 Θ : 90 degrees
 ϕR_n : **10.4 k/in** (AISC Eq 8-1)
DCR: **0.96** OK

Base Plate Bending:

d_eff: 4.0 in
b: **30.0 in**
t: **1.5 in**
Mu: **429 k-in**
Z: 16.9 in³ ($b \cdot t^2 / 4$)
Fy: **36 ksi**
Mn: 607.5 k-in
 ϕ : 0.9
 ϕM_n : **546.8 k-in**
DCR: **0.79** OK

Z_req'd: 13.3 in³

Make the base plate wider



Quantum Consulting Engineers LLC
1511 Third Avenue, Suite 323
Seattle, WA 98101

Project:	South Hill 1015	Date:	12/9/24	Job No:	19305.04
		Designer:	TVM	Sheet:	
Client:	Benaroya	Checked:			

Concrete Shear Cap Retrofit

IBC 2021, ACI 318-19

Reinforcing existing base plates for shear at grade beams with a 10" thick concrete cap.

Frame 25, 27, & 28

1.) Base Condition

Shear Force	$V_u =$	252 kips
Column		W12x65
Flange Width	$b_f =$	12 in
Fillet Width	$k_1 =$	1 in
Grout Thickness	$t_g =$	1 in
Concrete	$f'_c =$	5 ksi
Thickness	$t_c =$	10 in

2.) Concrete Bearing

Bearing Area	$A = b_f (t_c - t_g) =$	108 sqin	
Bearing Strength	$\phi B_n = \phi 0.85 A f'_c =$	298 kips	ACI Table 22.8.3.2
Unity Check	$U_c = V_u / \phi B_n =$	0.84	OK

3.) Adhesive Rebar Anchor

Bar Size		#5	
Bar Area	$A_{se,V} =$	0.31 sqin	Note: See Simpson Design
Bar Strength	$f_{uta} =$	60 ksi	Pryout does not control design
Shear Strength	$\phi V_{sa} = \phi 0.6 A_{se,V} f_{uta} =$	7.25 kips	
# of Bars Req'd		35	

Position half of bars on each side of column. Use drag steel to spread load

Drag Load	$V_u / 2 =$	126 kips	
Bar Size		4	
Bar Area	$A_{se,V} =$	0.2 sqin	
Bar Strength	$f_y =$	60 ksi	
Tension Strength	$\phi T_n = \phi A f_y =$	10.8 ksi	
# of Bars Req'd		12 total or	6 per side



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Project: South Hill 1015 Building

Date: 12/9/24 Job No: 19305.04

Designer: TVM Sheet: 1

Client: Benaroya

Checked:

Concrete Shear Cap Retrofit

IBC 2021, ACI 318-19

Reinforcing existing base plates for shear at grade beams with a 10" thick concrete cap.

Frame 2

1.) Base Condition

Shear Force	$V_u =$	294 kips
Column		W12x65
Flange Width	$b_f =$	12 in
Fillet Width	$k_1 =$	1 in
Grout Thickness	$t_g =$	1 in
Concrete	$f'_c =$	5 ksi
Thickness	$t_c =$	10 in

2.) Concrete Bearing

Bearing Area	$A = b_f (t_c - t_g) =$	108 sqin	
Bearing Strength	$\phi B_n = \phi 0.85 A f'_c =$	298 kips	ACI Table 22.8.3.2
Unity Check	$U_c = V_u / \phi B_n =$	0.99	OK

3.) Adhesive Rebar Anchor

Bar Size		#5	
Bar Area	$A_{se,V} =$	0.31 sqin	Note: See Simpson Design
Bar Strength	$f_{uta} =$	60 ksi	Pryout does not control design
Shear Strength	$\phi V_{sa} = \phi 0.6 A_{se,V} f_{uta} =$	7.25 kips	
# of Bars Req'd		41	

Position half of bars on each side of column. Use drag steel to spread load

Drag Load	$V_u / 2 =$	147 kips	
Bar Size		4	
Bar Area	$A_{se,V} =$	0.2 sqin	
Bar Strength	$f_y =$	60 ksi	
Tension Strength	$\phi T_n = \phi A f_y =$	10.8 ksi	
# of Bars Req'd		14 total or	7 per side