Project 2 - Report

SE 140A - Group 18

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I. Introduction

This project details the seismic force resisting system for an eight-story structure with Bucking-Restrained Braced Frames (BRBF) in the east-west direction. The building is located in San Jose with a zip code of 95113. The relevant seismic parameter values for this location were determined using the ASCE Hazard Tool. The structure is made of concrete slabs on a metal deck and supported with steel framing, composed of 35-foot bays. Each story is 15 feet high.

II. Equivalent Lateral Force (ELF) Procedure

The design loads were as follows, using the three tables below. Note that the additional "line" dead load, displayed below in Table 1.2, was calculated by multiplying the floor area loads by the tributary height of that floor. These line loads were applied later on just to the perimeter of the floor.

Table 2.2.1 - Area Dead Load Information

AREA DL	Floor Area Loads (psf)	Roof Area Loads (psf)
Concrete Slab	43.5	0
3-in Metal Deck	2.2	2.2
Steel Framing	15	12
Superimposed DL	15	12
TOTAL DL	**75.7**	**26.2**

Table 2.2.2 - Additional Line Dead Load Information

DL Line Loads	Floor Line Loads (plf) = (Hs) (13 psf)	Roof Line Load (plf) = (H□/2 + 6')(13)
Curtain Wall Weight (13 psf)	195	175.5

Table 2.2.3 - Area Live Load Information

Area LL	Floor Area Loads (psf)	Roof Area Loads (psf)
Partition Weight (ASCE 7 §12.7.2)	10	5

The length of the building can be found by adding the length of six bays and the two feet of overhang, resulting in a length of $212\ ft$. The width of the building is composed of four bays and two feet of overhang, resulting in a width of $142\ ft$. The perimeter was found to be $708\ ft$. The area was found to be $30,104\ ft^2$.

To design the lateral seismic force resisting systems, the equivalent lateral force (ELF) procedure was utilized. The seismic weight was found by adding up all the individual dead load components which include the area load of the flooring, partition weight, and curtain wall weight. The equation used to compute this final seismic weight is shown below in Equation 1.

$$w_i = \frac{(D_{RF} + W_{P-j})A_{FL} + W_{Cw-j}L_{PERIM}}{1000} \text{ (kips)} \qquad \text{[Equation 1]}$$

Table 2.2.4 – Effective Seismic Weight & Weight Per Floor

Effective Seismic Weight Determination					
Floor	DOF	Area Loads	Partition Weight	Curtain Wall Weight	Seismic Weight
j	i	D□ (psf)	w□-j (plf)	w_cw - j (plf)	w_i (kips)
RF	8	26.2	5	175.5	**1063.50**
8	7	75.7	10	195	**2717.97**
7	6	75.7	10	195	**2717.97**
6	5	75.7	10	195	**2717.97**
5	4	75.7	10	195	**2717.97**
4	3	75.7	10	195	**2717.97**
3	2	75.7	10	195	**2717.97**
2	1	75.7	10	195	**2717.97**
				Eff. Seismic Weight (kips) Σwi =	**20089.31**

III. Seismic Parameter Values

The seismic forces acting on the structure were calculated using the following variables.

1. Approximate Period, Ta.

The Approximate Period, Ta, of the structure was determined from ASCE ASCE 7-22 §12.8, Table 12.8-2. For the 120-foot tall building, steel buckling-restrained braced frames are to be used as the primary seismic force resisting systems for the E-W direction. Thus, the following parameters were used.

Table 2.3.1 – Structure Approximate Period

Period Ta Determination: [ASCE 7-22 §12.8]	
Factors	Steel BRBF
C□ (Table 12.8-2)	0.03
x (Table 12.8-2)	0.75
h□ (from base to Floor 9) (ft)	120
T = Ta = C□ * h□ˣ (s)	**1.09**

2. Building Design Information

The building specification given by the project and needed to determine the seismic forces are summarized below. The Risk Category was determined based on ASCE 7-22 §1.5 in Table 1.5-1. The soil site class was determined based on ASCE 7-22 §11.4.2.1.

Table 2.3.2 – Given Building Design Information

Design Info	
Building Type:	Office Building
Risk Category:	II
Site Conditions:	C, CD, D
Location:	San Jose, CA
ZIP Code	95113

3. *Spectral Response Spectrum Acceleration (S_a)*

The Multi-Period Design Response Spectrum for San Jose, CA was found on the USGS Seismic Design Geodatabase at https://asce7hazardtool.online/. The Spectrum Acceleration (S_a) was found and is summarized in the table below for the three different Site Conditions. The Spectrum Acceleration at the structure's approximate period, Ta = 1.09, is highlighted below for all three site conditions.

Table 2.3.3 – Spectrum Acceleration in San Jose

Spectral Response Spectrum Acceleration (S_a) Determination			
	S_a		
T (period)	Site Condition: C	Site Condition: CD	Site Condition: D
0	0.49	0.49	0.46
0.01	0.49	0.5	0.47
0.02	0.49	0.5	0.47
0.03	0.53	0.52	0.47
0.05	0.64	0.59	0.51
0.075	0.8	0.72	0.64
0.1	0.91	0.84	0.75
0.15	1.07	1	0.88
0.2	1.18	1.11	0.96
0.25	**1.19**	1.22	1.05
0.3	1.11	**1.26**	1.14
0.4	1.02	1.21	**1.2**
0.5	0.92	1.15	**1.2**
0.75	0.71	0.94	1.05
1	0.57	0.78	0.93

1.09	0.555	**0.783**	0.929
1.5	0.4	0.56	0.73
2	0.3	0.43	0.59
3	0.23	0.3	0.42
4	0.2	0.23	0.31
5	0.18	0.2	0.24
7.5	0.11	0.12	0.15
10	0.07	0.078	0.095

4. *Seismic Design Category*

The Seismic Design Category was found according to ASCE 7-22 §11.6 for all three site conditions and for Risk Category II. The highest value for S_{ds} and S_{d1} determined the category and the governing site condition was found. The information is summarized in the table below.

Table 2.3.4 – Seismic Design Category

Seismic Design Category Determination for **Risk Category II** Building					
	Seismic Design Category				
Factors	Site Condition: C	Site Condition: CD	Site Condition: D	Category Determination	Governing
S_1	0.62	0.62	0.62	N/A	
S_ds	1.07	1.14	1.08	CD	
S_d1	0.57	0.89	1.13	D	**D**

5. *Seismic Design Coefficients & Factors*

The Response Modification Coefficient (R), Overstrength Factor ($\boldsymbol{\Omega_o}$), Deflection Amplification Factor (Cd), and Structural Height Limit (h□ Limit) were found in ASCE 7-22 §12.2 using Table 12.2-1, Design Coefficients and Factors for Seismic Force-Resisting Systems. These values were dependent on the building frame system that we were designing for, the steel bucking-restrained braced frames. Seismic Importance Factor (I_e) was found in ASCE 7-22 §1.5 using Table 1.5-2, the Importance Factors by Risk Category of Buildings and Other Structures for Earthquake Loads table. The value was determined by the Risk Category of the structure, Risk Category II.

Table 2.3.5 – Seismic Design Coefficients & Factors

	Seismic Design Coefficients and Factors (*R*, Ω_o, *Cd*, h□ Limit, I_e)	
	Factors	Seismic Design Category D
	R	8
	Ω_o	2.5
	Cd	5
[ASCE 7-22 §12.2] Table 12.2-1	**h□ Limit**	160
[ASCE 7-22 §1.5] Table 1.5-2	**I_e**	1

6. *Seismic Response Coefficient & Base Shear*

The variables needed to calculate the Seismic Response Coefficient were summarized for each site condition. The Spectrum Acceleration, S_a, is from Table 2.3. S_1, S_ds, and S_d1 are from Table 2.4. I_e and R are from Table 2.5. The value for Cs, the seismic response coefficient, is found by following procedures outlined in ASCE 7-22 §12.8.1.1 with equation 12.8-2. There are two checks for the lower bounds of Cs given in equations 12.8-6 and 12.8-7 when S_1 is greater than 0.6. From the three site conditions, Site Condition D governs with the greatest value of Cs being 0.1162. The governing Cs value, 0.1162, was then multiplied by the Effective Seismic Weight ($\Sigma wi = 20089.31$) which is shown in Table 2.4, the Effective Lateral Force Per Floor table. This gives a base shear value of 2333.73 kips.

Table 2.3.6 – Seismic Response Coefficient & Base Shear

	Seismic Base Shear Determination			
		Seismic Design Category		
	ASCE 7-22 §12.8.1.1	Site Condition: C	Site Condition: CD	Site Condition:

				D
	S_a	0.555	0.783	0.929
	S_1	0.62	0.62	0.62
	S_ds	1.07	1.14	1.08
	S_d1	0.57	0.89	1.13
	I_e	1	1	1
Factors	R	8	8	8
[Eq. 12.8-2]	Cs = S_a (I_e/R)	**0.0694**	**0.0979**	**0.1162**
[Eq. 12.8-6]	Cs□in = 0.044(S_ds)(I_e) >= 0.01	0.04708	0.05016	0.04752
[Eq. 12.8-7] $S_1 > 0.6$	Cs□in = 0.5S_1 / (R/I_e)	0.03875	0.03875	0.03875
	Governing Cs	**0.0694**	**0.0979**	**0.1162**
			Site Condition D Governs	
	Base Shear: V = CsW (kips)	**2333.73**		

7. Seismic Force Calculation

The seismic response coefficient was found using equation 2.

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{[Equation 2]}$$

The exponent K was found using linear interpolation between the values 1 and 2 based on the period of the structure. This is shown in Table 2.7.1.

Table 2.3.7 – K Calculation

K Calculation (using linear interpolation)

T (s)	k
0.5	1
1.09	**1.588**
2	2

The seismic force for each level was found by multiplying the seismic response coefficient for the corresponding floor and the seismic base shear. The numerator of the seismic response factor was found by multiplying the level's seismic weight by its structural height to the 1.588 power, the K value. The denominator is the sum of all multiplied seismic weight and story heights. C_{vx} is shown as the vertical distance factor. The shear on each individual floor is shown as the Total Lateral Force, F_x. The demand for each bucking-restrained braced frame, BRBF, is the sum of the seismic force divided by the number of BRBFs, 4, and the force caused by the additional torsional effect. The total force for a single BRBF comes out to 641.78 kips.

Table 2.3.8 – Effective Seismic Force & Force Per Floor

Effective Seismic Force Calculation								
DOF	Seismic Weight	Structural Height		Vertical Distance Factor	Total Lateral Force	Force per BRBF	Torsional Effect per BRBF	Force per BRBF
(i = x)	w_i (kips)	h_i (ft)	$w_i h_i^k$(k-ft)	$C_v x = w_i h_i^k / (\Sigma w_i h_i^k)$	F_x (kips)	$f_x = F_x/4$(kips)	$f_i = 0.05\ F/2$ (kips)	$f_{i,□o□a□} = f_i + f_{□i}$ (kips)
8	1063.50	120	2127370	0.130	304.52	76.13	7.61	83.74
7	2717.97	105	4398226	0.270	629.58	157.39	15.74	173.13
6	2717.97	90	3443392	0.211	492.90	123.22	12.32	135.55
5	2717.97	75	2577928	0.158	369.01	92.25	9.23	101.48
4	2717.97	60	1808869	0.111	258.93	64.73	6.47	71.21
3	2717.97	45	1145625	0.070	163.99	41.00	4.10	45.10
2	2717.97	30	601815	0.037	86.15	21.54	2.15	23.69

1	2717.97	15	200226	0.012	28.66	7.17	0.72	7.88
Σ	20089.31		**16303451**	1.000	**2333.73**			**641.78**

IV. Preliminary Buckling-restrained Braced Frames (BRBF) Design

The same BRB size was used for stories 1-3, 4-6, and 7-8, so three BRB sizes were chosen for the structure. Sizes for the three BRBs were determined by the largest required brace area. The required steel core area was determined using equation 3 given in AISC 341 §F5.5b.2. Required areas for each floor were then rounded up, as it is a common practice to do so.

$$A_{sc(req'd)} = \frac{P_u}{\phi F_{ysc(min)}} \quad \text{[Equation 3]}$$

P_u is the required brace axial force. This is the force in tension or compressing in the BRB due to the seismic lateral loading. Equation 4 was used to determine the axial force at each floor with $\Sigma f_{k,total}$ being the sum of the force per BRBF of each floor and those above it.

$$P_u = \frac{\Sigma f_{k,total}}{2cos\theta} \quad \text{[Equation 4]}$$

ϕ is equivalent to 0.9 and $F_{ysc(min)}$ is equivalent to 38 ksi at every level.

Table 2.4.1

Effective Seismic Force Calculation								
DOF	Force per BRBF	Story Shear	Angle of BRB	Brace Axial Force	Min. Yield Stress of Steel Core	Required Brace Areas	Rounded Brace Areas	
(i = x)	fi,□o□a□ (kips)	ΣF□,□o□a□	θ (deg.)	P_u (kips)	F_ysc□i□ (ksi)	Asc, req (in²) [AISC 341 F5.5b.2]	Asc,req (rounded) (in²)	BRB Design
8	83.74	83.74	40.60	55.148	38.00	1.61	2.00	BRB5
7	173.13	256.88	40.60	169.163	38.00	4.95	5.00	BRB5
6	135.55	392.42	40.60	258.427	38.00	7.56	8.00	BRB11

5	101.48	493.90	40.60	325.254	38.00	9.51	10.00	BRB11
4	71.21	565.11	40.60	372.146	38.00	10.88	11.00	BRB11
3	45.10	610.21	40.60	401.844	38.00	11.75	12.00	BRB13
2	23.69	633.90	40.60	417.445	38.00	12.21	13.00	BRB13
1	7.88	641.78	40.60	422.635	38.00	12.36	13.00	BRB13
Σ	**641.78**							

BRBF Preliminary Design Drawing

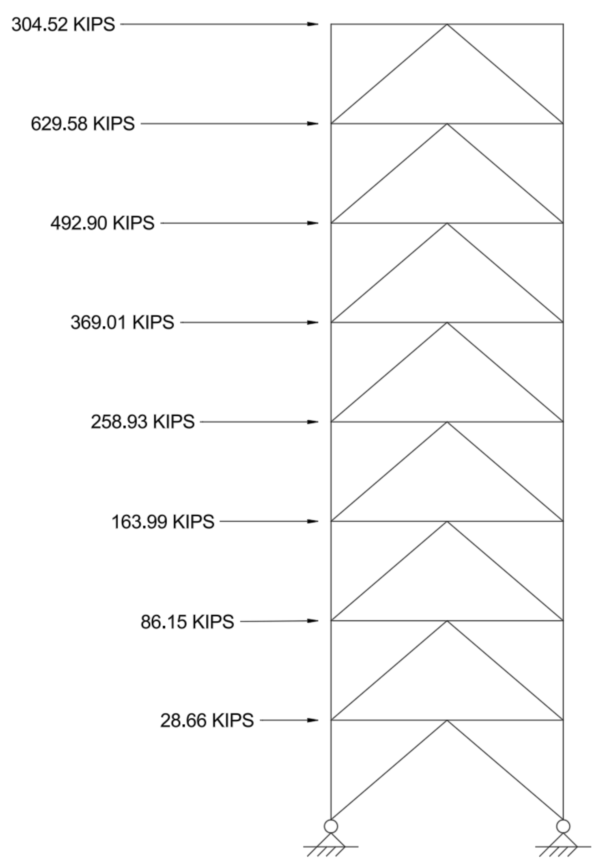


Figure 2.1.1 - ELF Procedure Results

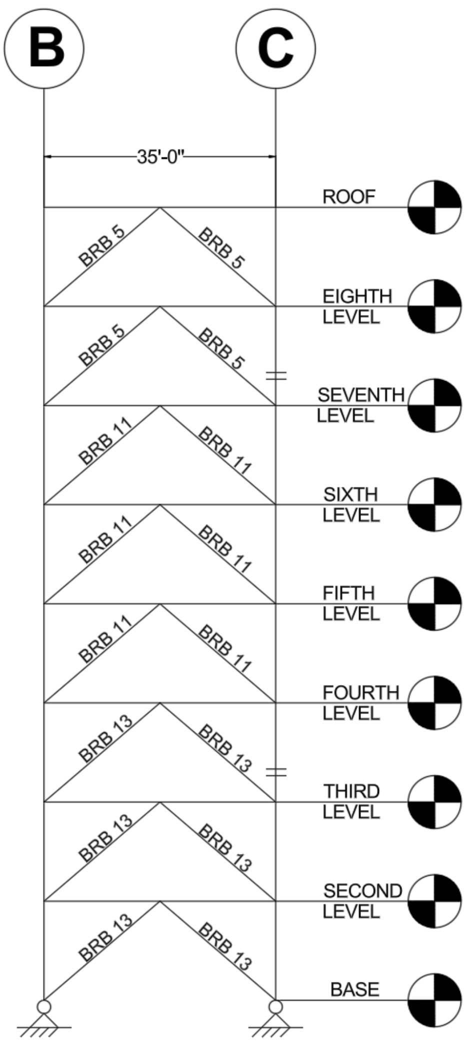


Figure 2.1.2 - BRB Preliminary Sizing

V. BRBF Beam Capacity Design

1. Beam Seismic Demand

The maximum tensile and compressive forces applied to the BRBs, $T_{max,\,i}$ and $P_{max,\,i}$, were found for each story using the following equations and the factors found in Table 5.0.1.

$$\text{Maximum tensile force: } T_{max,\,i} = \omega_i F_{y(max)} A_{sci} \qquad \text{[Equation 5]}$$

$$\text{Maximum compressive force: } P_{max,\,i} = \beta_i \omega_i F_{y(max)} A_{sci} \qquad \text{[Equation 6]}$$

Table 5.1.1 – Average values used for project

Strain hardening adjustment factor (ω)	1.5
Compression strength (β)	1.3

Table 5.1.2 – Seismic Force in BRBs

Effective Seismic Force Calculation				
DOF	F_ysc(max)	A□c	Seismic Force Level III	
(i = x)	(ksi)	(in²)	T□ax,i (kips)	P□ax,i (kips)
8	46.00		345.00	448.50
7	46.00	5.00	345.00	448.50
6	46.00		759.00	986.70
5	46.00		759.00	986.70
4	46.00	11.00	759.00	986.70
3	46.00		897.00	1166.10
2	46.00		897.00	1166.10
1	46.00	13.00	897.00	1166.10
Σ			5658.00	7355.40

From the unequal tension and compression in the BRBs, the horizontal unbalanced force, H_{ub}, and vertical unbalanced forces, V_{ub}, were found. The angle of all the BRBs from horizontal is 0.71 rad as the story height is 15ft and half of each bay is 17.5ft so arctangent of 15/17.5 is 0.71 rad.

$$H_{ub,\,i} = (P_{max,\,i} + T_{max,\,i}) \cos\theta_i = (\beta_i + 1)\,\omega_i F_{ysc(max)} A_{sci} \cos\theta_i \qquad \text{[Equation 7]}$$

$$V_{ub,i} = (P_{max,i} + T_{max,i})\sin\theta_i = (\beta_i - 1)\,\omega_i F_{ysc(max)} A_{sci} \sin\theta_i \qquad \text{[Equation 8]}$$

Table 5.1.3 – Horizontal and vertical unbalanced BRB beam forces

Unbalanced Loads on BRBF Beam		
DOF	$H_u$bi	$V_u$bi
(i = x)	(kips)	(kips)
8	602.47	67.36
7	602.47	67.36
6	1325.43	148.19
5	1325.43	148.19
4	1325.43	148.19
3	1566.42	175.13
2	1566.42	175.13
1	1566.42	175.13

The bending moment, M_{Emh}, axial force, P_{Emh}, and beam shear, V_{Emh}, were found from the unbalanced forces on the beam. The unbalanced force was applied at the beam's midspan. The length of the beam used, L', was the length in between the beam splice. The splice was made 2.50ft from the ends so the length used was 30ft (L' = 35ft (bay width) - 2(2.5ft) = 30ft). The various demands on the beam were found with the following equations.

$$\text{Bending Moment: } M_{Emh} = \frac{(V_{ub,i} * L')}{4} \qquad \text{[Equation 9]}$$

$$\text{Axial Force: } P_{Emh} = \frac{H_{ub,i}}{2} \qquad \text{[Equation 10]}$$

$$\text{Beam Shear: } V_{Emh} = \frac{V_{ub,i}}{2} \qquad \text{[Equation 11]}$$

Table 5.1.4 – BRB beam demands

Member Force Demands in Beam due to Unbalanced Load (E_mh)			
DOF	M_Emh	P_Emh	V_Emh
(i = x)	(kip-ft)	(kips)	(kips)
8	505.18	301.24	33.68
7	505.18	301.24	33.68

6	1111.39	662.72	74.09
5	1111.39	662.72	74.09
4	1111.39	662.72	74.09
3	1313.46	783.21	87.56
2	1313.46	783.21	87.56
1	1313.46	783.21	87.56

The demands are summarized on the table below.

Table 5.1.5 – BRB beam demand summary

BRBF Beam Seismic Demands						
DOF	Seismic Force Level III		Unbalanced Forces		E_cl	
(i = x)	T□$_{ax,i}$ (kips)	P□$_{ax,i}$ (kips)	$H_u$bi (kips)	$V_u$bi (kips)	P_Emh (kips)	M_Emh (kip-ft)
8	345.00	448.50	602.47	67.36	301.24	505.18
7	345.00	448.50	602.47	67.36	301.24	505.18
6	759.00	986.70	1325.43	148.19	662.72	1111.39
5	759.00	986.70	1325.43	148.19	662.72	1111.39
4	759.00	986.70	1325.43	148.19	662.72	1111.39
3	897.00	1166.10	1566.42	175.13	783.21	1313.46
2	897.00	1166.10	1566.42	175.13	783.21	1313.46
1	897.00	1166.10	1566.42	175.13	783.21	1313.46

2. *Beam-Column Demand*

The dead point load and distributed load, P_D and w_D, and the live point load and distributed load, P_L and w_L, on the beam were used to find the moment due to dead load, M_D, and moment due to live load, M_L. M_{Emh} is the bending moment due to unbalanced seismic forces found in Table 5.4.

Three load combinations were considered

1. $M_{u1} = (1.2 + 0.2S_{DS})M_D + 0.5M_L - M_{Emh}$ [Equation 12]
2. $M_{u2} = (0.9 + 0.2S_{DS})M_D - M_{Emh}$ [Equation 13]
3. $M_{ug} = max(1.2M_D + 1.6M_L, 1.4M_D)$ [Equation 14]

Table 5.2.1 – Factored moment demands on BRBF

BRBF1 Beam (Beam-Column) Demands							
DOF	P_Emh	S_DS	L'	P_D	P_L	w_D	w_L
(i = x)	(kips)	(-)	(ft)	(kips)	(kips)	(kip/ft)	(kip/ft)
8	301.24	1.08	30	17.1	17.27	0.195	0
7	301.24	1.08	30	17.1	17.27	0.195	0
6	662.72	1.08	30	17.1	17.27	0.195	0
5	662.72	1.08	30	17.1	17.27	0.195	0
4	662.72	1.08	30	17.1	17.27	0.195	0
3	783.21	1.08	30	17.1	17.27	0.195	0
2	783.21	1.08	30	17.1	17.27	0.195	0
1	783.21	1.08	30	17.1	17.27	0.195	0

DOF	Moment (kip-ft)			Factored Moment (kip-ft)		
(i = x)	M_D	M_L	M_Emh	Combo 1 (M_{u1})	Combo 2 (M_{u2})	Gravity-Only (M_{ug})
8	193.19	172.67	505.18	-145.29	-373.04	508.09
7	193.19	172.67	505.18	-145.29	-373.04	508.09
6	193.19	172.67	1111.39	-751.50	-979.25	508.09
5	193.19	172.67	1111.39	-751.50	-979.25	508.09
4	193.19	172.67	1111.39	-751.50	-979.25	508.09
3	193.19	172.67	1313.46	-953.57	-1181.32	508.09
2	193.19	172.67	1313.46	-953.57	-1181.32	508.09
1	193.19	172.67	1313.46	-953.57	-1181.32	508.09

The equivalent axial load method was used to select a compressive trial section. M_{u1} and M_{u2} are both negative so the seismic force dominates over gravity and they have the same unbraced length, L_b , of L' divided by two, or 15ft. L_c is equal to L_b. Since they are both negative, M_{u2} will govern.

We can estimate the beam depth with the following equation:

$$D = 1.2\sqrt[4]{P_u L_c} + 1.1\sqrt[4]{M_u L_c} + \frac{\sqrt{P_u M_u}}{3L_c^2} \qquad \text{[Equation 15]}$$

These beam depths were rounded to the closest beam size.

The value of m was chosen from Figure 5.2 based on the length of the beam and the estimated depth. The value of B_1 was found from Figure 5.3 and all three depths result in a value of 1.02.

Table 1. Values of *m* and *u* ($F_y = 50$ ksi[a] for $C_b = 1.0$)

L_c (ft)	8	10	12	14	16	18	20	22	24+	Values of *u*
	Values of *m*									
W8	2.7	2.5	2.3	2	1.8	1.6	1.3	1.2	1	2.3
W10	2.2	2.1	2	1.9	1.8	1.6	1.4	1.3	1.2	2.3
W12	1.9	1.8	1.7	1.7	1.6	1.5	1.4	1.3	1.2	2.3
W14	1.6	1.6	1.5	1.5	1.4	1.4	1.3	1.2	1.2	2.3
W16	1.4	1.4	1.3	1.2	1.1	1	0.9	0.9	0.8	4.1
W18	1.3	1.2	1.2	1.1	1	1	0.9	0.8	0.7	4.1
W21	1.1	1.1	1	1	1	0.9	0.8	0.8	0.7	4.3
W24	1	1	0.9	0.9	0.9	0.8	0.8	0.7	0.7	4.4
W27	0.9	0.9	0.8	0.8	0.8	0.8	0.7	0.7	0.7	4.6
W30	0.8	0.8	0.8	0.8	0.7	0.7	0.7	0.7	0.6	4.8
W33	0.8	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.6	5.1
W36	0.7	0.7	0.7	0.6	0.6	0.6	0.6	0.5	0.5	6.1

[a] For $F_y = 65$ ksi, multiply *m* value by 0.95.

Figure 5.2 Value of m from Reynolds and Uang (2019)

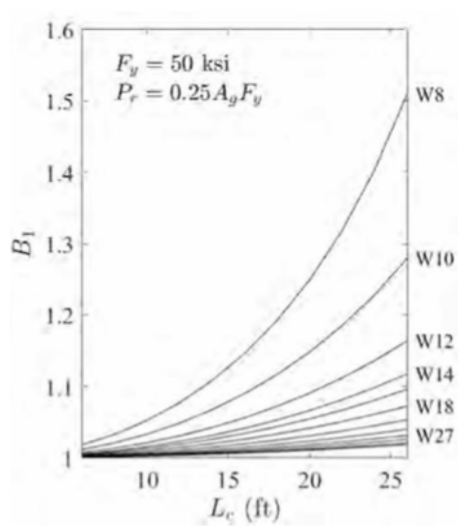


Figure 5.3 Value of B_1 (major axis bending) from Reynolds and Uang (2019)

Table 5.2.2 – Equivalent Axial Load Method Factors

Preliminary BRBF1 Beam Design								
DOF	Factored Moment (kip-ft)		Unbraced Length 2	P_Emh	Estimated Depth	Rounded Depth	Chosen m	Chosen B1
(i = x)	Combo 1 (M_{u1})	Combo 2 (M_{u2})	Lb = Lc (ft)	(kips)	d (in)	d (in)		

8	-145.29	-373.04	15	301.24	19.85	21	0.9	1.02
7	-145.29	-373.04	15	301.24	19.85	21	0.9	1.02
6	-751.50	-979.25	15	662.72	25.29	27	0.8	1.02
5	-751.50	-979.25	15	662.72	25.29	27	0.8	1.02
4	-751.50	-979.25	15	662.72	25.29	27	0.8	1.02
3	-953.57	-1181.32	15	783.21	26.61	27	0.8	1.02
2	-953.57	-1181.32	15	783.21	26.61	27	0.8	1.02
1	-953.57	-1181.32	15	783.21	26.61	27	0.8	1.02

The equivalent axial load, P_{eq}, is calculated with the following equations:

$$P_r = P_{Emh} \quad \text{[Equation 16]}$$

$$M_r = B_1 M_u \quad \text{[Equation 17]}$$

$$P_{eq} = P_r + mM_{rx} \quad \text{[Equation 18]}$$

The ratio of $\frac{P_r}{P_{eq}} \geq 0.2$ is satisfied for each of the beams.

Table 5.2.3 – Beam Selection from Equivalent Axial Load Method

Preliminary BRBF1 Beam Design					
DOF (i = x)	$M_r = B_1M_u$	Peq	P_r/P_{eq}	Preliminary Beam Design	Super Table ΦPn (kips)
8	380.50	**643.68**	0.47	W21x101	1010.00
7	380.50	**643.68**	0.47		
6	998.83	**1461.78**	0.45	W27x146	1540.00
5	998.83	**1461.78**	0.45		
4	998.83	**1461.78**	0.45		
3	1204.95	**1747.17**	0.45	W27x178	1890.00
2	1204.95	**1747.17**	0.45		
1	1204.95	**1747.17**	0.45		

VI. BRBF Beam Splice Design

1. *Bolt Design*

After designing the beams, it was necessary to create a beam splice between the beam stub attached to the column and the full-length beam. Assuming shear strength governs for both interior and exterior bolts, the number of bolts required to resist the P_{Emh} was calculated by dividing P_{Emh} by the shear strength of the bolts from AISCM Table 7-1.

$$n_{req} = (P_{Emh})/(\phi r_n) \qquad \text{[Equation 19]}$$

Next, the required number of bolts was rounded up to the nearest even number. The bolt spacing and edge distance were determined such that the full bearing and tearout strength could be utilized from AISCM Table 7-4 and Table 7-5. With a bolt diameter $d_b = 1\ in$, the spacing $s = 3.5\ in$ and the edge distance $l_e = 3\ in$. These values can help determine the final half-length of the WT shape.

Table 6.1.1 – Bolt Design and Spacing

Bolt Design & Spacing			
Floor 2 Beam:	W27x178	Values	Units
	Φr□ =	63.6	kips
	P_e□□ =	783.21	kips
	n_{re}q = P_e□□ /Φr□ =	12.315	bolts
	n =	14	bolts
	d_b =	1	in
1" A325-N, Group 120 Bolt:	d□ =	1.125	in
	Φ□ =	0.75	
	F_u =	65	ksi
	U_a =	0.85	
Rupture Limit State (RLS)	P_u =	391.61	kips

2. *WT-Shape Design*

Moving on, a WT-shape design was bravely selected assuming Rupture Limit State (RLS) governs. Since the section is still unknown, Case 7 was assumed, which provides a U-factor of $U_a = 0.85$. Knowing that $P_u = P_{emh}/2$, the required area was found using $A_{e,req} = P_u/(\phi_t F_u)$. The $A_{n,req}$ then can be easily found by dividing the $A_{e,req}$ by the $U_a = 0.85$ factor. Lastly, the required area of the WT section can be found by $A_{g,req} = A_{n,req} - 2d_h t_f$. Through trial and error, a reasonable trial section was selected such that the $A_{WT} > A_{g,req}$, taking into account that t_f will be different for every tested section.

Section WT8x33.5 was tested. It has a flange thickness of $t_f = 0.665\ in$ and $A_{WT} = 9.81\ in^2$, satisfying the $A_{WT} > A_{g,req}$ requirements for the given flange thickness. Next, the bolts were spaced to remain within the maximum and minimum bolt spacing and edge distance requirements. The minimum spacing equation is $s_{min} = 8/3 * d_b$. The maximum spacing equation is $s_{max} = max(14t, 7)$. The minimum edge spacing can be found directly from AISCM Table J3.4, while the maximum edge spacing values can be found using the following equation: $l_{e,max} = max(12t, 6)$. The s_{full} and $l_{e,full}$ values can be pulled directly from AISCM Table 7-4 and Table 7-5.

The $s = 3.5\ in$, which allows for s_{full} to be used while remaining well within $s_{min} = 2.67\ in$ and $s_{max} = 9.31\ in$ requirement. Similarly, the edge distance $l_e = 3\ in$, which allows for $l_{e,full}$ to be used while remaining within the $l_{min} = 1.25\ in$ and $l_{max} = 7.98\ in\ bounds$.

The calculations and values can be found below in Table 6.2.1

Table 6.2.1 – WT-Shape Design and Spacing

WT Design & Spacing			
WT8x33.5	WT Shape t_f =	0.665	in
	A_e,req = Pu / (Φ□F_u) =	8.03	in²
	A□,req = A_e,req / U_a =	9.45	in²
	Ag,req = A□,req - 2d□t_f =	7.95	in²
	A_wt =	9.81	in²
Bolt Layout	s_full =	3.125	in
	s_min = 8/3d_b	2.67	in
	s =	**3.5**	in
	s□ax = max(14t,7in)	9.31	in
	l_e_full	2.5625	in
	l_e,min =	1.25	in

	l_e =	**3**	in
	l_e,max =	7.98	in
Length of Half the WT-Shape	L = 2(l_e) + s(n/2 -1) + 0.5/2	27.25	in
Length of Full WT-Shape	L_full	54.50	in

3. Bolt Checks

Now that the WT-shape and bolts have been designed, the first check involves the bolts themselves. The bolt group strength can be found by multiplying the individual shear strengths of these bolts by the total number of bolts by interior and exterior bolts respectively. Once summed together, the final bolt group strength is 1187.55 kips, which exceeds the demand $P_{emh} = 783.21$ kips, resulting in a DCR of 0.66

Table 6.3.1 – Bolt Design Checks

Bolt Design Checks			
Interior & Edge [Table 7-1]	Shear Strength, Φr□	63.6	kips
	Bearing Strength per in, Φr□	117	kip/in
	t = min(t_w_beam, 2*t_f_WT)	0.725	in
Interior [Table 7-4]	Bearing Strength, Φr□	84.825	kips
	Bearing Strength per in, Φr□	117	kip/in
	t = min(t_w_beam, 2*t_f_WT)	0.725	in
Edge [Table 7-5]	Bearing Strength, Φr□	84.825	kips
	n_interior	12	bolts
	n_exterior	2	bolts
	Φr□, interior	84.825	kips
	Φr□, exterior	84.825	kips
	Bolt Strength = n_i(Φr□$_i$) + n_e(Φr□$_e$)	**1187.550**	kips
	P_e□□ =	**783.21**	kips
Bolt Group Strength	DCR = P_e□□/(Bolt Strength)	**0.660**	

4. *WT-Shape Checks*

Next, the WT-shape itself must be checked. The design tensile strength must be smaller than the "supply" factored shear values based on three limit states: Yielding Limit State (YLS), Rupture Limit State (RLS), and Block Shear Limit State. The minimum shear values out of these will be utilized to determine the DCR.

The YLS calculations involve a simple process of comparing the yielding strength with the demand.

$$\phi r_{n,YLS} = \phi F_y A_g \qquad \text{[Equation 20]}$$

The RLS calculations were slightly more complicated. The U-factor for Case 2 was calculated using $U = 1 - \bar{x}/L$. The eccentricity $\bar{x}$ was found using Table 1-8, while the length (L = 18.05 in) was calculated based on the designed spacing. This gave a U factor of 0.914, which was used to find the An value $A_n = A_g - \sum d_h t$, resulting in a value of 8.314 in^2. The A_e value was found by multiplying A_n with the U factor, resulting in a final effective area A_e value of 7.595 in^2. The final nominal RLS value was then calculated to be $\Phi P_n = \Phi F_u A_e = 444.32\ kips$.

For the Block Shear Limit State, the total shear plane length was calculated then multiplied by the element thickness. Note that the length of the shear plane was found by $L_{plane} = 2(l_e + s(n/2 - 1)) = 48\ in$. When multiplied by the thickness of the WT-shape flange $t_f = 0.665\ in$, the final shear $A_{gv} = 31.92\ in^2$. The $A_{nv} = A_{gv} - nd_h t = 21.45\ in^2$.

$A_{nt} = (2\ tension\ pl.)(tension\ pl.\ length - \#\ holes * diameter)(elem.\ thickness) = 3.242\ in^2$.

The block shear equation is

$R_n = 0.60 F_u A□_v + (U_{bs})(F_u)(A□□) <= 0.6 F_y A_{gv} + (U\_bs)(F_u)(A□□) = 1047.13\ kips$

. When factored, the final block shear capacity value is 785.34 kips.

To check the supply and demand, the minimum of the limit state shear values was found to be from the RLS, with a value of 444.32 kips. The demand is Pu = 391.61 kips, resulting in a DCR of 0.89, which is satisfactory.

Table 6.4.1 – Bolt Design Checks

WT-Shape Design Check			
YLS	ΦPn = ΦFyAg	441.45	kips
	x̄ =	1.56	in
	l = (s)(n-1)	18.05	in
	U = 1-x̄/L	0.914	
	U_LB = (b_f)(t_f)/(Ag)	0.519	
	An = Ag - ∑d□t	8.314	in^2
	Ae = (An)(U)	7.595	in^2
RLS (U-Factor: Case 2)	ΦPn = ΦFuAe	444.32	kips
	A_gv = (total shear plane length)(elem thickness)	31.92	in^2
	A_nv = A_gv - nd□t	21.45	in^2
	A□t = (2 t planes)(t plane length - #holes * diam)(elem thickness)	3.242	in^2
	Rn = 0.60F_uA□$_v$ + (U_bs)(F_u)(A□□) <= 0.6FyAgv + (U_bs)(F_u)(A□□)	1047.13	kips
Block Shear	ΦRn =	785.34	kips
	ΦRn_final = min(YLS, RLS, Block Shear) =	**441.45**	kips
	Pu =	**391.61**	kips
Governing	DCR = Pu / ΦRn_final	**0.89**	

5. *Beam Checks*

Next, the W-shape beam itself must be checked. Similar to the WT-shape, the design tensile strength must be smaller than the "supply" factored shear values based on three limit states: Yielding Limit State (YLS), Rupture Limit State (RLS), and Block Shear Limit State. The minimum shear values out of these will be utilized to determine the DCR.

The YLS calculations involve a simple process of comparing the yielding strength with the demand.

$$\phi r_{n,YLS} = \phi F_y A_g = 2362.5\ kips \quad \text{[Equation 21]}$$

The RLS calculations were slightly more complicated. The U-factor for Case 2 was calculated using $U = 1 - \bar{x}/L$. The eccentricity x̄ was found by dividing the Zy by the gross area A. The length was then calculated based on the designed spacing. This gave a U factor of 0.871, which was used to find the An value $A_n = A_g - \sum d_h t$, resulting in a value of 50.87 in^2. The A_e value was found by multiplying A_n with the U factor, resulting in a final effective area A_e value of 44.32 in^2. The final nominal RLS value was calculated to be $\Phi P_n = \Phi F_u A_e = 3592.72\ kips$.

For the Block Shear Limit State, the total shear plane length was calculated then multiplied by the element thickness. Note that the length of the shear plane was found by $L_{plane} = 2(l_e + s(n/2 - 1)) = 48\ in$. When multiplied by the thickness of the WT-shape flange $t_f = 0.665\ in$, the final shear $A_{gv} = 31.92\ in^2$. The $A_{nv} = A_{gv} - nd_h t = 21.45\ in^2$. $A_{nt} = (2\ tension\ pl.)(tension\ pl.\ length\ -\ \#\ holes\ *\ diameter)(elem.\ thickness) = 3.242\ in^2$.
The block shear equation is
$R_n = 0.60F_{\mathrm{u}}A_{\square\mathrm{v}} + (U_{bs})(F_{\mathrm{u}})(A_{\square\square}) <= 0.6F_y A_{gv} + (U\_bs)(F_{\mathrm{u}})(A_{\square\square}) = 4236.42\ kips$
. When factored, the final block shear capacity value is 3177.32 kips.

To check the supply and demand, the minimum of the limit state shear values was found to be from the RLS, with a value of 2362.5 kips. The demand is Pu = 391.61 kips, resulting in a DCR of 0.17, which is satisfactory.

Table 6.5.1 – Bolt Design Checks

Beam Design Check (W27x178)			
Info	Ag =	52.5	in²
	t_w =	0.725	in
	F_u =	65	ksi
YLS	ΦPn = ΦFyAg	2362.5	kips
RLS (U-Factor: Case 2)	x̄ = Z_y/Ag	2.32	in
	l = (s)(n-1)	18.05	in
	U = 1-x̄/L	0.871	
	U_LB = (b_w)(t_w)/(Ag)	0.318	
	An = Ag - ∑d□t	50.87	in²

	Ae = A□U =	44.32	in²
	ΦPn = ΦFuAe	2592.719092	kips
	A_gv = (total shear plane length)(elem thickness)	31.92	in²
	A_nv = A_gv - nd□t	20.50	in²
Block Shear	A□t = (2 t planes)(t plane length - #holes * diam)(elem thickness)	52.875	in²
	Rn = 0.60F_uA□$_v$ + (U_bs)(F_u)(A□□) <= 0.6FyAgv + (U_bs)(F_u)(A□□)	4236.42	kips
	ΦRn =	3177.32	kips
	ΦRn_final = min(YLS, RLS, Block Shear) =	**2362.50**	kips
	Pu =	**391.61**	kips
Governing	DCR = Pu / ΦRn_final	**0.17**	

VII. BRBF Column Capacity Design

The vertical columns framing the tension and compression members were spliced at the third and sixth floor 4ft above the floor. Three columns were connected to span the whole building height.

The columns used for the BRBF were determined by the demand of the vertical components of tension and compression caused by the members resisting the lateral seismic loading. The shear demand caused by the unequal tension and compression in the BRBs, V_{ub}, causes an upwards force on the columns connected to the beams and is also added as part of the demand. Compression force demand will govern the design of BRBF columns so tension was not considered. The total compressive force in the columns, Cumulative P_{Emh}, was determined with the following equation:

$$P_{Emh,\,i} = \sum_{i+1}^{8} P_{max,\,i}\, sin\theta_i - \sum_{i}^{8} \frac{V_{ub,i}}{2} \qquad \text{[Equation 22]}$$

The cumulative compression force at each floor is the sum of the vertical unbalanced force, dead load, and live load using the first load combination found in Equation 12. The maximum compression force for each column is found at the lowest floor. For the top most column, the max compression force is on the seventh floor. For the middle column, the fourth floor. For the bottom most column, the first floor. These maximum values are shown in the right most column in Table 6.1.

Table 7.1 – BRBF Column Demand

BRBF Column Sizing							
DOF	Seismic Force Level 3		Vertical Unbalanced Force	Cumulative P_Emh	Cumulative Gravity Load Effects		Cumulative Comp. Force
(i = x)	T_max (kips)	P_max (kips)	V_ub (kips)	(kips)	ΣP_D (kips)	ΣP_L (kips)	(kips)
8	345.00	292.35	67.36	33.68	17.13	17.27	**290.76**
7	345.00	292.35	67.36	224.99	34.25	34.53	
6	759.00	643.16	148.19	443.25	51.38	51.80	**1745.80**
5	759.00	643.16	148.19	1012.32	68.50	69.07	
4	759.00	643.16	148.19	1581.39	85.63	86.33	
3	897.00	760.10	175.13	2136.99	102.75	103.60	**3745.13**
2	897.00	760.10	175.13	2809.53	119.88	120.87	
1	897.00	760.10	175.13	3482.07	137.00	138.13	

Column sizes for the BRBF were chosen from the maximum cumulative compression force in each of the three columns and are shown in Table 6.2. The same beam depth of 14ft was chosen for each of the three members so that the connection between them would be stronger and easier. The demand capacity ratio shows that the chosen members for compression are very economical. All three columns chosen pass the check for highly ductile members that AISC 341 §F4.5a requires.

Table 7.2 – BRBF Column Size Selection & Check

BRBF Column Sizing					
DOF	Cumulative Comp. Force	Column Size	Comp. Strength	DCR	Highly Ductile Check
(i = x)	(kips)	(-)	ϕPn (kips)	(-)	(-)
8	**290.76**	**W14X48**	**331**	**0.88**	NL
7					
6	**1745.80**	**W14X132**	**1930**	**0.90**	NL
5					

4					
3					
2	**3745.13**	**W14X283**	**4340**	**0.86**	NL
1					

VIII. BRBF Design Drawings

1. Elevation Drawing

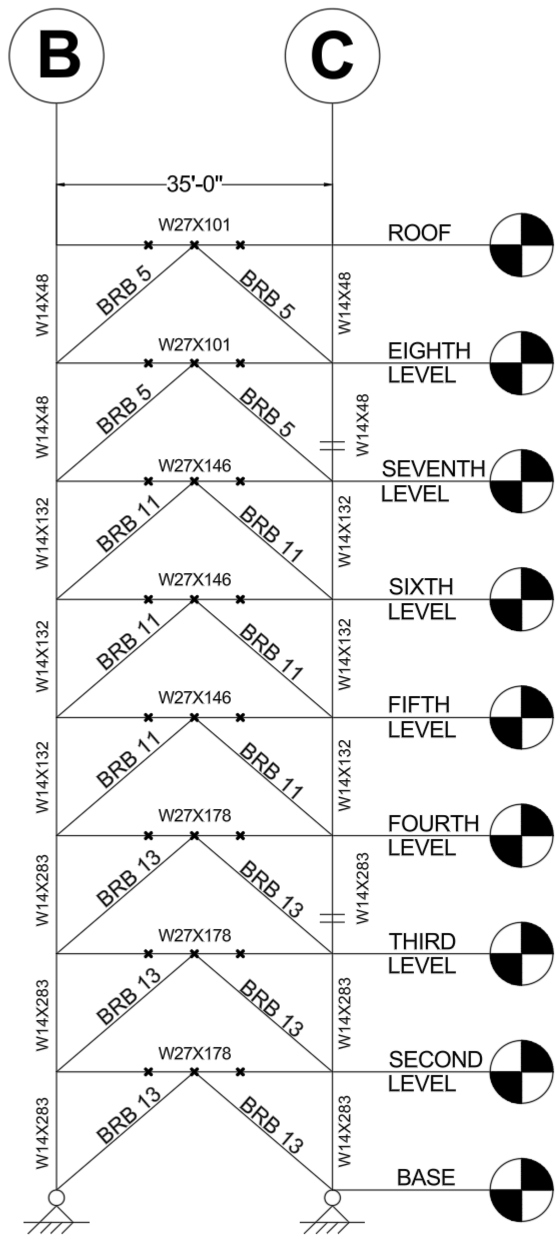


Figure 8.1 Elevation View of BRBF

IX. Story Drift Check

The BRBF was modeled in RISA 2D to observe how it would deform when loaded. The beams were pinned at each end in the connection to the column and all BRB tension and compression elements were pinned. The bottom most column is pinned at the base. Any other connection is fixed (splices, midspan of beams). The loading found in the ELF Procedure was applied at the nodes with half on one side of the structure and the other half applied at the node opposite to it. The loading applied at each level is shown in Table 9.1.

Table 9.1 – Lateral Loading Applied at Each Story

DOF (i = x)	Applied Force (kips)	Force at each side (kips)
8	334.97	167.485
7	692.54	346.27
6	542.19	271.095
5	405.92	202.96
4	284.82	142.41
3	180.39	90.195
2	94.76	47.38
1	31.53	15.765

The members selected for the beam, column, and BRB are listed below with their area (A), moment of inertia (I), and Young's Modulus (E). These values are needed for the RISA model.

Table 9.2 – Summary of Beam Selection

DOF	**Beams (Floor *i* + 1)**			
(i = x)	Section	A (in^2)	I (in^4)	E
8	W21x101	29.8	2420	29000
7	W21x101	29.8	2420	
6	W27x146	43.2	5660	
5	W27x146	43.2	5660	
4	W27x146	43.2	5660	
3	W27x178	52.5	7020	
2	W27x178	52.5	7020	
1	W27x178	52.5	7020	

Table 9.3 – Summary of Column Selection

DOF	Columns (Floor *i* + 1)			
(i = x)	Section	A (in^2)	I (in^4)	E
8	W14X48	14.1	484	29000
7	W14X48	14.1	484	
6	W14X132	38.8	1530	
5	W14X132	38.8	1530	
4	W14X132	38.8	1530	
3	W14X283	83.3	3840	
2	W14X283	83.3	3840	
1	W14X283	83.3	3840	

The moment of inertia, I_x , and Young's Modulus, E, is modified to fit the parameters of the design. I_x does not matter because the loading is purely axial for BRBs but the value inputted into RISA is an equation of area: $I = \frac{s^4}{12} = \frac{A_{sc}^2}{12}$. The Young's Modulus must be adjusted to consider K_{Eff} so it is changed to be 1.3E, or 37700.

The educational edition of RISA 2D is limited to 50 members so the top most BRB was adjusted to be a diagonal instead of a chevron bracing. The area of the BRB was adjusted to reflect this geometric change using Equation 23.

$$1(\frac{EA}{l} cos\, \theta) = \frac{EA'}{l} cos\, \theta' \qquad \text{[Equation 23]}$$

Table 9.3 – Summary of BRB Selection

DOF	BRB (Story *i*)			
(i = x)	Section	A (in^2)	I (in^4)	E
8	BRB 5	27.294	62.080203	37700
7	BRB 5	5	2.083333333	
6	BRB 11	11	10.08333333	
5	BRB 11	11	10.08333333	
4	BRB 11	11	10.08333333	
3	BRB 13	13	14.08333333	
2	BRB 13	13	14.08333333	
1	BRB 13	13	14.08333333	

The final loading and frame structure that was analyzed in the story drift check is shown below in Figure 9.1. Each story is 15ft in height and the bay width is 35ft.

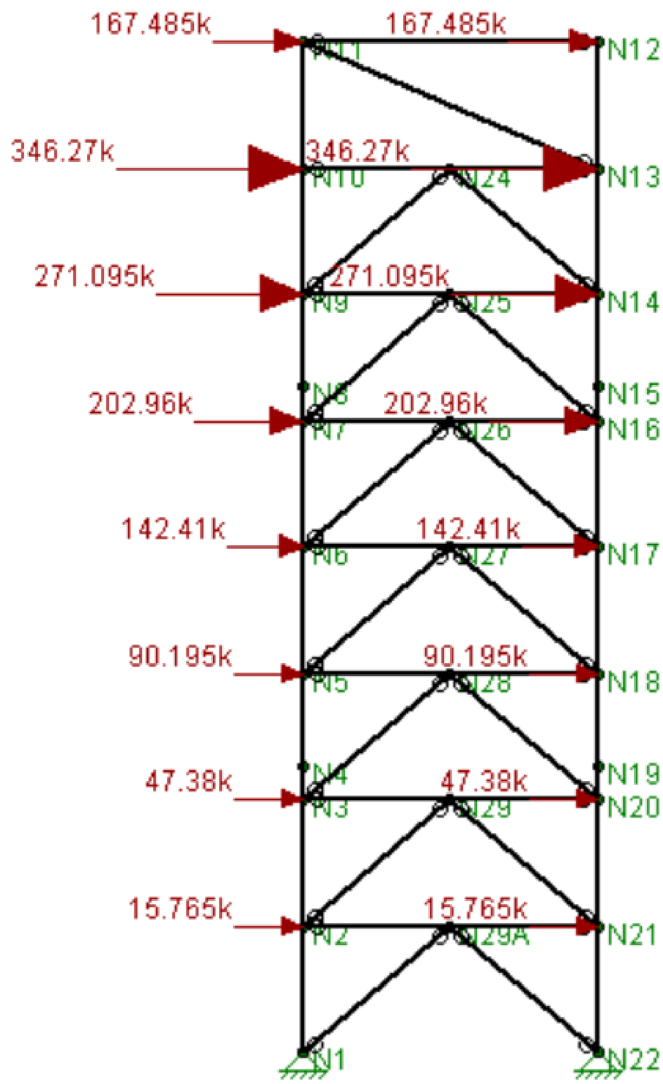


Figure 9.1 BRBF Loading in RISA 2D

The analysis was run and the resulting story drift is shown in Table 9.4. The story drift was the drift of each individual story and it was modified by being multiplied by a factor of 5 to change it from a seismic force level 2 to seismic force level 3. The allowable story drift is 0.02H, or 0.02

multiplied by the story height. The modified story drift was significantly higher than the allowable drift so it did not pass the check.

Table 9.4 – Story Drift Check

DOF	Floor i+1 Node Disp. (in)	Story *i* Drift			Check
(i = x)		Story Drift (in)	Modified (in)	Allowable (in)	
8	17.518	1.947	9.735	3.6	NG
7	15.571	2.965	14.825	3.6	NG
6	12.606	2.465	12.325	3.6	NG
5	10.141	2.548	12.74	3.6	NG
4	7.593	2.447	12.235	3.6	NG
3	5.146	1.962	9.81	3.6	NG
2	3.184	1.74	8.7	3.6	NG
1	1.444	1.444	7.22	3.6	NG

X. Gusset Connection Design

A gusset connection connects the BRB to the column and beam. The design of the gusset plate and welds were done for the eighth floor, fifth floor, and second floor.

1. Gusset Demands from BRB

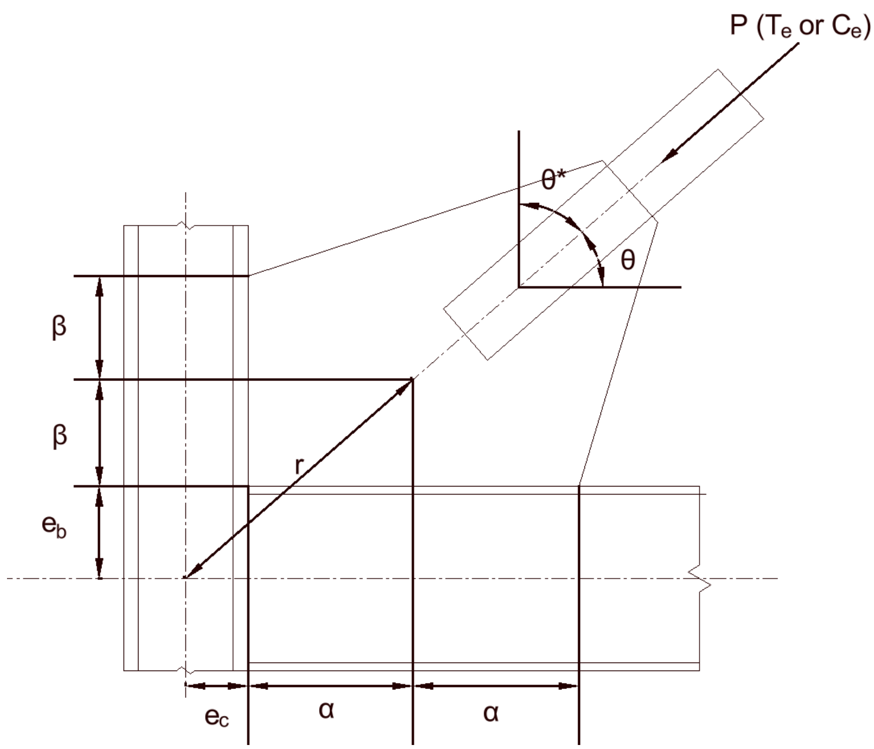
The demands for the gusset plate was found using the Uniform Force Method, UFM. The basis of UFM is that no moment exists at the

1) gusset-to-beam interface
2) gusset-to-column interface
3) beam-to-column interface.

For these three conditions to be satisfied, the following equation must be satisfied:

$$\alpha - \beta\, tan\theta^{*} = e_{b}\, tan\theta^{*} - e_{c} \qquad \text{[Equation 24.1]}$$

Figure 10.1.1 Gusset Plate Dimensions



The variables used in Equation 24.1 for the second, fifth, and eighth story are summarized in Table 10.1. Their physical meanings are described below and can be seen in Figure 10.1.1:

2α: Length of gusset connected to the beam

2**β**: Length of gusset connected to the column

θ^*: Angle of the BRB from vertical

e_b: Half the depth of the beam

e_c: Half the depth of the column

α is limited to half the length of the beam splice connected to the column. L_a, the distance from the center of the column to the splice, is set to be 2.5', or 30". This make L_c, the length of the beam splice connected to the column, equal to $L_a - e_c$. L_c was divided by two and rounded down to the nearest half an inch so that $2\alpha < L_c$. All the unknowns in Equation 24.1 are found other than the value of **β** so **β** can be found after reorganizing Equation 24. The equation for **β** is shown in Equation 24.2.

$$\beta = \frac{1}{tan\theta^*}\alpha + (\frac{e_c}{tan\theta^*} - e_b) \quad \text{[Equation 24.2]}$$

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} \quad \text{[Equation 25]}$$

Table 10.1.1 Gusset Plate Required Dimensions

Important Dimensions									
Story	La	Lc (in)	e_b (in)	e_c (in)	r	θ*	PICK		
(-)	(in)	La - e_c	d_beam / 2	d_col / 2	(in)	(rad)	**α (in)**	**β (in)**	**Round β (in)**
8th	30	23.1	10.7	6.9	24.351	0.8622	**11.5**	5.07	**5.25**
5th	30	22.65	13.7	7.35	15.547	0.8622	**11**	2.03	**2.25**
2nd	30	21.65	13.9	8.35	16.215	0.8622	**10.5**	2.26	**2.5**

The horizontal and vertical force demands on the beam and column that were transferred from the BRB to the gusset plate are found with the following equations. H indicates the horizontal force while V indicates the vertical force. The subscript b indicates that it's the force on the beam while the subscript c indicates that it's on the column. P is the tension force from the BRB, T_e.

$$H_b = (\frac{\alpha}{r})P \qquad V_b = (\frac{e_b}{r})P \quad \text{[Equations 26]}$$

$$H_c = \left(\frac{e_c}{r}\right) P \qquad V_c = \left(\frac{\beta}{r}\right) P \qquad \text{[Equations 27]}$$

Figure 10.1.1 Force Demands on Gusset Plate Interface

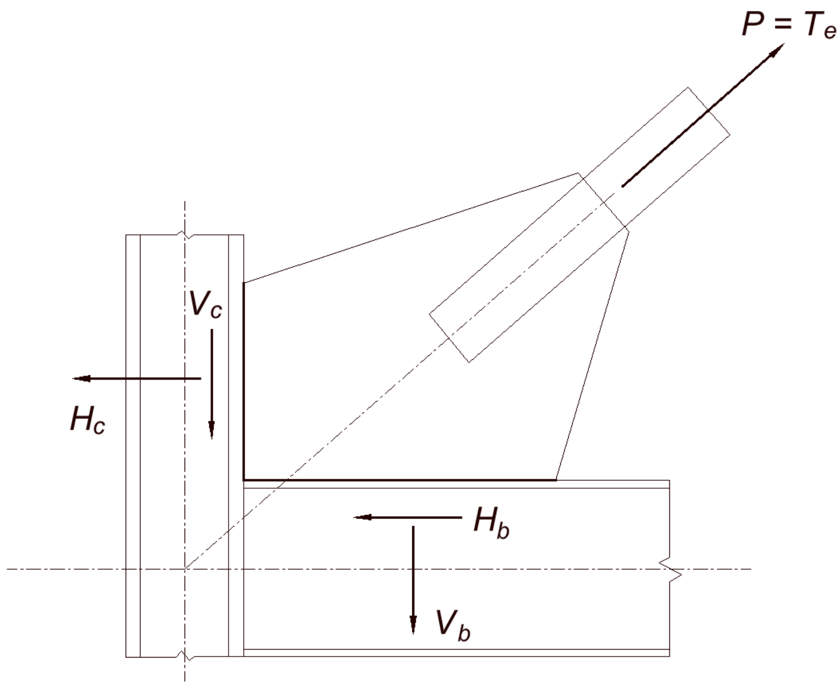


Table 10.1.2 Horizontal & Vertical Force Demands from BRB

Uniform Force Method					
Story	Te	H_b	V_b	H_c	V_c
(-)	(kips)	(kips)	(kips)	(kips)	(kips)
8th	345	162.93	151.60	97.76	71.84
5th	759	537.01	668.83	358.82	98.99
2nd	897	580.84	768.93	461.91	124.81

2. Gusset-to-Beam Connection Capacity Design

The demand per unit length, f_x and f_y, at the beam connection is found by dividing the horizontal and vertical forces by the length of the gusset weld connection to the beam, $2\boldsymbol{\alpha}$. The resultant unit demand, f_r, is then found from the horizontal and vertical components. θ_w is the angle of the unit demand, f_r, from the horizontal axis.

$$f_r = \sqrt{f_x^2 + f_y^2}$$ [Equation 28]

$$\theta_w = tan^{-1}(\frac{f_y}{f_x})$$ [Equation 29]

The supply of the welds, ϕr_n, must be greater than the demand, f_r. The supply is given in Equation 29 with *w* being the minimum weld size and F_{Exx} being 70 ksi. The minimum weld size, *w,* was found from the equation and increased by 25% to account for the modified Richard factor of 1.25. The final weld size was rounded to the next 1/16" and is shown in bold in Table 10.1.3.

$$\phi r_n = 0.75[2 \times 0.707w \times 1"][0.6F_{Exx}(1.0 + 0.50\, sin^{1.5}\theta_w)]$$ [Equation 30]

Table 10.2.1 Gusset-to-Beam Weld Size

Gusset-to-Beam Weld						
Story	f_x	f_y	f_r	θ_w	w	**Rounded w**
(-)	(kip/in)	(kip/in)	(kip/in)	(rad)	(in)	**(in)**
8th	7.08	6.59	9.68	0.749	0.21	**0.25**
5th	24.41	30.40	38.99	0.894	0.81	**0.875**
2nd	27.66	36.62	45.89	0.924	0.95	**1**

3. Gusset-to-Column Connection Capacity Design

The demand per unit length, f_x and f_y, and resultant unit demand, f_r, are found the same way that they were found for the gusset-to-beam design. The angle θ_w is different as it is now the angle between the vertical axis and the resultant unit demand.

$$\theta_w = tan^{-1}(\frac{f_x}{f_y})$$ [Equation 31]

The weld size, *w*, is found the same way as the gusset-to-beam weld using Equation 29.

Table 10.3.1 Gusset-to-Column Weld Size

Gusset-to-Column Weld						
Story	f_x	f_y	f_r	θ_w	w	**Rounded w**
(-)	(kip/in)	(kip/in)	(kip/in)	(rad)	(in)	**(in)**
8th	9.31	6.84	11.55	0.94	0.24	**0.25**
5th	79.74	22.00	82.72	1.30	1.58	**1.625**
2nd	92.38	24.96	95.69	1.31	1.82	**1.875**

4. Gusset Plate Thickness

The gusset plate thickness, t_{PL}, was designed considering the shear and axial force demands at the welds to the beam and column when the BRB is in tension.

The shear force capacity is given in Equation 31 with the gross area for the gusset-to-beam interface being shown in Equation 32 and the gusset-to-column interface in Equation 33.

$$V_n = 0.60\,F_y\,A_{gv} \quad \text{[Equation 32]}$$

$$A_{gv} = (a\alpha)(t_{PL}) \quad \text{[Equation 33]}$$

$$A_{gv} = (a\beta)(t_{PL}) \quad \text{[Equation 34]}$$

The axial force capacity is given in Equation 34 with the gross area for the for the gusset-to-beam interface being shown in Equation 35 and the gusset-to-column interface in Equation 36.

$$P_n = F_y\,A_g \quad \text{[Equation 35]}$$

$$A_g = (2\alpha)(t_{PL}) \quad \text{[Equation 36]}$$

$$A_g = (2\beta)(t_{PL}) \quad \text{[Equation 37]}$$

Equations 32 and 33 produce 2 checks for the shear force, one at the gusset-to-beam interface (Equation 37) and the other at the gusset-to-column interface (Equation 38). Equations 35 and 36 produce 2 checks for the axial force, one at the gusset-to-beam interface (Equation 39) and the other at the gusset-to-column interface (Equation 40).

These four checks were done for the connection at the eighth, fifth, and second floor. Each of the checks give a required minimum thickness, t_{PL}, so the largest of the four values was used as the plate thickness at each floor. t_{PL} was then rounded up to the nearest 1/8"

$$H_b \le [\phi V_n = (1.0)0.60F_y\,(2\alpha\,t_{PL})] \quad \text{[Equation 38]}$$

$$V_c \le [\phi V_n = (1.0)0.60F_y\,(2\beta\,t_{PL})] \quad \text{[Equation 39]}$$

$$V_b \le [\phi P_n = (0.9)F_y\,(2\alpha\,t_{PL})] \quad \text{[Equation 40]}$$

$$H_b \le [\phi P_n = (0.9)F_y\,(2\beta\,t_{PL})] \quad \text{[Equation 41]}$$

Table 10.4.1 Minimum Gusset Plate Thickness

	Shear Force				Axial Force				
Story	H_b		V_c		V_b		H_c		**Plate Thickness**
	demand	resulting t_PL	demand	resulting t_PL	demand	resulting t_PL	demand	resulting t_PL	**(in)**
8th	162.93	0.24	71.84	0.23	151.60	0.15	97.76	0.32	**0.375**
5th	537.01	0.81	98.99	0.73	668.83	0.68	358.82	3.30	**3.375**
2nd	580.84	0.92	124.81	0.83	768.93	0.81	461.91	3.42	**3.5**

5. Beam & Column Web Local Yielding Check

The horizontal and vertical demands on the beam and column webs where the gusset plate is welded must be checked to see that the column and beam do not yield. The demands are caused by compression from the BRB and the vertical force on the beam, V_b, and horizontal force on the column, H_c, must be less than the strength of the beam and column. The supply of the beam is given in Equation 41 and the supply of the column is given in Equation 42. k_b and k_c are the curvature of the beam and column respectively. t_{wb} and t_{wc} is the thickness of the web of the beam and column respectively.

$$\phi R_{nb} = 1.0(2\alpha + 2.5\,k_b)\,F_{yb}\,t_{wb} \quad \text{[Equation 42]}$$

$$\phi R_{nc} = 1.0(2\beta + 5\,k_c)\,F_{yc}\,t_{wc} \quad \text{[Equation 43]}$$

The beam and column for all three floors passed the web local yielding check with the demand capacity ratio being less than 1 as seen in Table 10.5.1 and 10.5.2.

Table 10.5.1 Beam Web Local Yielding Check

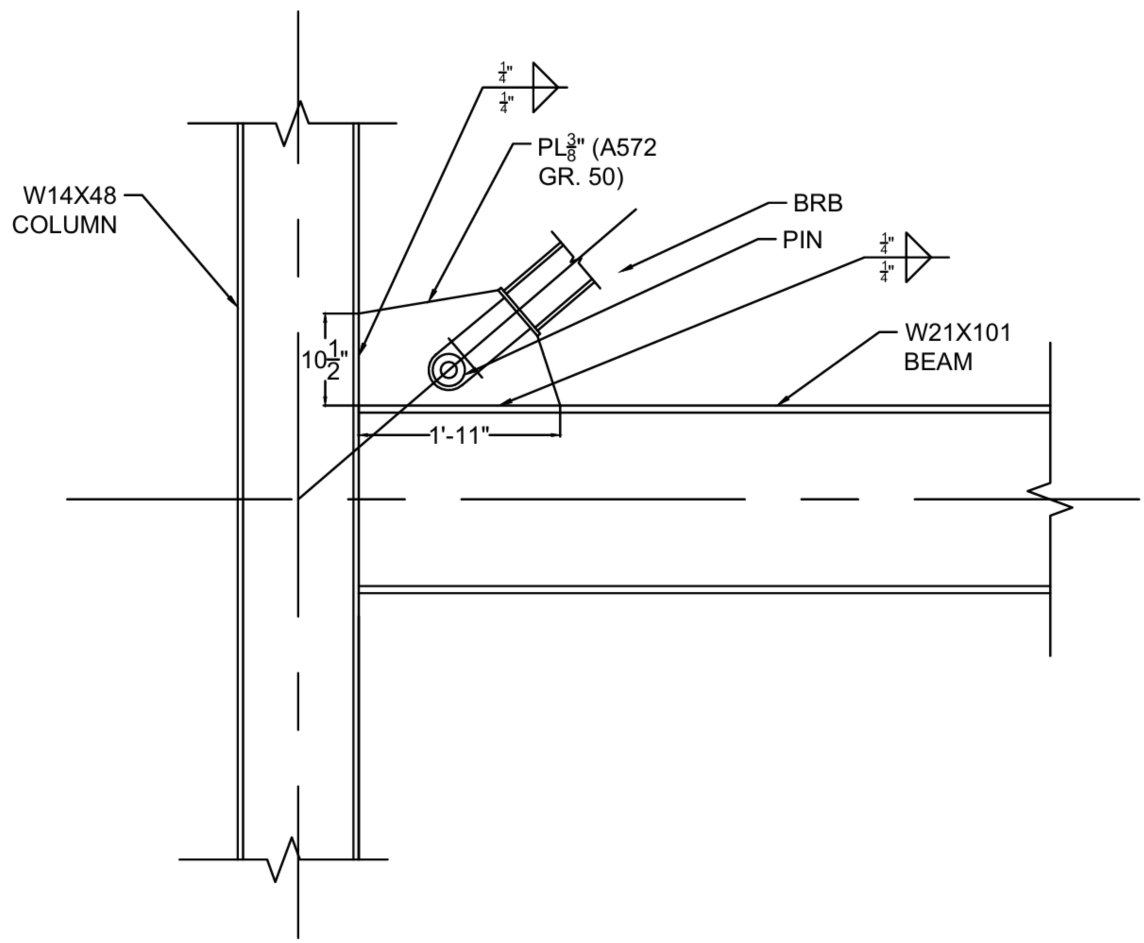
Story			**Beam WLY**		
	k_b	t_wb	Demand (Vb)	Supply	DCR
8th	1.30	0.50	151.60	656.25	0.23
5th	1.76	0.61	668.83	798.60	0.84
2nd	1.98	0.73	768.93	940.69	0.82

Table 10.5.2 Column Web Local Yielding Check

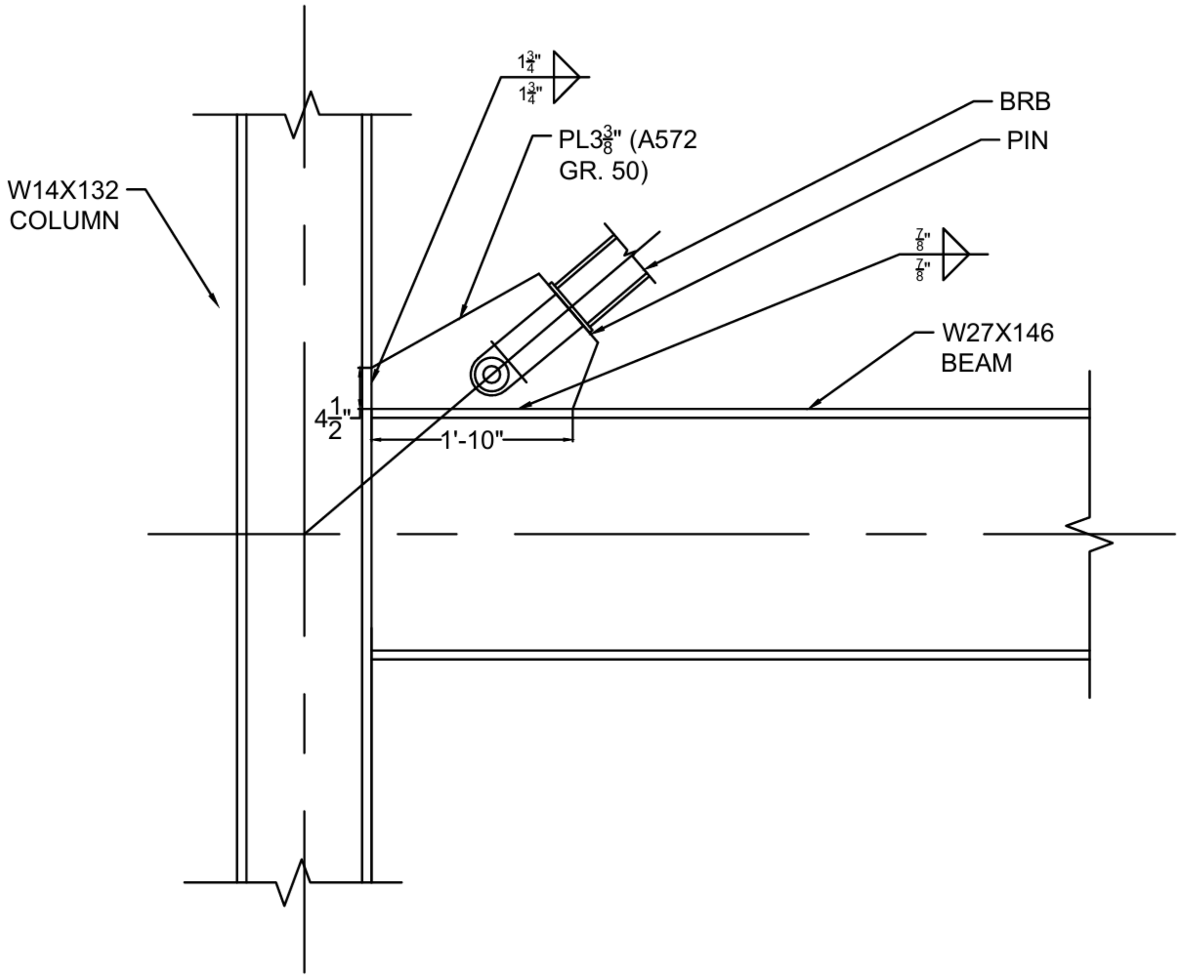
Story			**Column WLY**		
	k_c	t_wc	Demand (Hc)	Supply	DCR
8th	1.19	0.34	97.76	279.65	0.35
5th	1.63	0.65	358.82	407.96	0.88
2nd	2.67	1.29	461.91	1183.58	0.39

XI. Connection Design Drawings

1. Eighth Floor



2. *Fifth Floor*



3. Second Floor