NIST GCR 21-917-48v2B

Seismic Design of Archetype Steel Buildings in Central and Eastern United States

Volume 2B – 7-story Healthcare Building in Long Island, New York Supplementary Documentation

Applied Technology Council

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**Seismic Design of Archetype Steel
Buildings in Central and Eastern
United States
Volume 2B – 7-story Healthcare
Building in Long Island, New York
Supplementary Documentation**

Prepared for
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Preface

Seismic design codes in the United States have evolved since first introduced in 1925; however, they are primarily based on the knowledge of performance requirements for buildings located in high seismicity regions of the United States, such as the West Coast. These model codes are extrapolated for use in areas with moderate seismicity, such as the Central and Eastern United States (CEUS), where member sizes of the lateral-force-resisting systems may be governed by wind requirements. There is a need to understand the seismic performance of buildings when the controlling design load is from wind effects.

In September 2018, the Applied Technology Council (ATC) commenced a task order project under National Institute of Standards and Technology (NIST) Contract SB1341-13-CQ-0009 to develop designs for archetype steel buildings to facilitate future research in understanding the seismic performance of buildings when the controlling design load is from wind. For this purpose, three archetype steel buildings were designed in accordance with older building codes and current building codes for specific locations within the CEUS. This document is one of three volumes presenting representative structural calculations supplementing design drawings for archetype steel buildings provided NIST GCR 21-917-48v2A.

The designs presented were developed by design firms PCS Structural Solutions of Seattle, Washington and Uzun + Case of Atlanta, Georgia. The Project Technical Committee, consisting of Don Scott, John Hutton, and Adrian Persaud monitored and guided the technical efforts of the Project Working Groups, which included Steve Antilla, Jared Dragovich, Hai Lin, Chris Putman, Cameron Prince, and Gavin Rinaldo. Project Working Group member McKell Bowen led the development of two of the three designs presented. The Project Review Panel, consisting of Melissa Burton (ATC Board Contact), C.B. Crouse, Ramon Gilsanz, Larry Griffis, Emily Guglielmo, Eric Hines, and Erik Madsen provided technical advice and consultation over the duration of the work. The names and affiliations of all who contributed to this report are provided in the list of Project Participants in the main Volume.

ATC also gratefully acknowledges Jay Harris (Contracting Officer's Representative) for his input and guidance throughout the project development process. ATC staff member Justin Moresco and Ginevra Rojahn provided project management support and report production services, respectively.

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Table of Contents

Preface	iii
Design Case A: 1987 NBC “Risk Category II” “Seismic Design Category B”	A-1
Design Case B: 2018 IBC Risk Category III Seismic Design Category B	B-1
Design Case C: 1987 NBC “Risk Category IV” “Seismic Design Category B”	C-1
Design Case D: 2018 IBC Risk Category IV Seismic Design Category B	D-1
Design Case E: 2018 IBC, Risk Category IV, Seismic Design Category D	E-1
Design Case F: 2018 IBC, Risk Category IV, Seismic Design Category C	F-1
Ground Motion Selection for Performance-Based Seismic Evaluation	G-1

Design Case A

1987 NBC “Risk Category III” “Seismic Design Category B”

This section presents representative structural calculations for Building 2 (7-story healthcare facility in Long Island, New York) Design Case A. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v2A.

TABLE OF CONTENTS

DESIGN CRITERIA.....	1
GRAVITY DESIGN.....	4
BEAM DESIGN.....	6
COLUMN DESIGN.....	11
LATERAL DESIGN.....	18
BRACED FRAME DESIGN.....	21
MOMENT FRAME DESIGN.....	40

DESIGN CRITERIA

DESIGN CRITERIA

VERTICAL LOADS

AREA	DESIGN DEAD LOAD	LIVE LOAD (2)	PARTITION LOAD	CONCENTRATED LOADS
OPERATING ROOMS	95 PSF	60 PSF	20 PSF	1000#
PRIVATE ROOMS	95 PSF	40 PSF	20 PSF	1000#
CORRIDORS ABOVE 1ST FLOOR	95 PSF	80 PSF	20 PSF	1000#
ROOF	100 PSF	40 PSF (1)	-	-

(1) DRIFT AND UNBALANCED SNOW LOAD PER NBC, 1987, CHAPTER 4.

(2) LIVE LOAD REDUCTION NOT PERMITTED EXCEPT AS NOTED IN NBC 1987, SECTION 1115.0.

SNOW:

$P_g = 35$ PSF = GROUND SNOW LOAD

$P_f = C_e I P_g$ = FLAT ROOF SNOW LOAD = 29.4 PSF

$P_s = C_s P_f$ = SLOPED ROOF SNOW LOAD

$C_e = 1.2$, $C_s = 0.7$

LATERAL FORCES

LATERAL FORCES ARE TRANSMITTED BY DIAPHRAGM ACTION OF ROOF AND FLOORS TO BRACED FRAME/MOMENT FRAME. LOADS ARE THEN TRANSFERRED TO FOUNDATION BY BRACED FRAME/MOMENT FRAME ACTION WHERE ULTIMATE DISPLACEMENT IS RESISTED BY PASSIVE PRESSURE OF EARTH AND/OR SLIDING FRICTION. OVERTURNING IS RESISTED BY DEAD LOAD OF THE STRUCTURE.

WIND:

THE BUILDING MEETS THE CRITERIA PER NBC 1987 SECTION 1112.0.

EXPOSURE CATEGORY = B

BASIC WIND SPEED, $V = 90$ MPH

EQUIVALENT RISK CATEGORY PER TABLE 1.5-1 = III

PRESSURE COEFFICIENT (ENCLOSED) = 0.8, -0.5

WIND IMPORTANCE FACTOR $I_w = 1.07$

DESIGN WIND BASE SHEAR = 390 KIPS

SEISMIC: (NBC 1987) $V = Z I_k C S W$

$Z = 3/8$

SEISMIC IMPORTANCE FACTOR, $I = 1.25$

EQUIVALENT RISK CATEGORY OF BUILDING PER TABLE 1113.1 = III

$k = 1.0$ AT BRACED FRAME, $k = 0.67$ AT MOMENT FRAME

W = EFFECTIVE SEISMIC WEIGHT OF BUILDING = 12180 KIPS

ANALYSIS PROCEDURE USED = EQUIVALENT LATERAL FORCE PROCEDURE

$C_s = 0.14$

DESIGN BRACE FRAME BASE SHEAR $V = 805$ KIPS

DESIGN MOMENT FRAME BASE SHEAR $V = 535$ KIPS

FOUNDATION DESIGN CRITERIA

SOIL BEARING PRESSURE: 6000 PSF (ASSUMED)

ACTIVE PRESSURE - RESTRAINED: 50 PCF +14H SEISMIC SURCHARGE (ASSUMED)

ACTIVE PRESSURE - UNRESTRAINED: 35 PCF +6H SEISMIC SURCHARGE (ASSUMED)

PASSIVE RESISTANCE: 200 PCF (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

COEFFICIENT OF FRICTION: .35 (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

*1/3 INCREASE ALLOWED FOR SEISMIC OR WIND LOADING

CONCRETE

CAST-IN-PLACE CONCRETE

ITEM	DESIGN f_c (PSI) (AT 28 DAYS U.N.O.)
FOUNDATIONS	3000
SLABS ON GRADE AND SLABS ON METAL DECK	4000

REINFORCING STEEL

ASTM A615, GRADE 60 TYPICAL UNLESS NOTED OTHERWISE.

WELDED WIRE REINFORCEMENT SHALL CONFORM TO ASTM A185. LAP ONE FULL MESH ON SIDES AND ENDS BUT NOT LESS THAN 8 INCHES. WELDED WIRE REINFORCING SHALL BE SUPPORTED TO WITHSTAND CONCRETE PLACEMENT. PULLING OF MESH INTO PLACE AFTER PLACEMENT IS NOT ALLOWED.

REINFORCING SPLICE AND DEVELOPMENT LENGTH SCHEDULE, $F_y=40$ KSI (UNLESS NOTED OTHERWISE)

BAR SIZE	MINIMUM LAP SPLICE LENGTHS ("Ls")	MINIMUM DEVELOPMENT LENGTHS ("Ld")
#3	1'-6"	1'-3"
#4	2'-0"	1'-7"
#5	2'-7"	2'-0"
#6	3'-1"	2'-4"
#7	4'-6"	3'-6"
#8	5'-2"	3'-11"
#9	5'-10"	4'-6"
#10	6'-6"	5'-0"
#11	7'-3"	5'-7"

STRUCTURAL STEEL

MATERIAL PROPERTIES

WIDE FLANGE SECTIONS: ASTM A36 ($F_y = 36$ KSI)

OTHER SHAPES AND PLATES: ASTM A36 ($F_y = 36$ KSI)

HOLLOW STRUCTURAL SECTIONS: RECTANGULAR & SQUARE - ASTM A500 GRADE B ($F_y = 46$ KSI)

MACHINE BOLTS (M.B.): ASTM A307

HIGH-STRENGTH BOLTS: A325-ASTM F1852, A490-ASTM F2280

ANCHOR BOLTS (A.B.): ASTM A490, GRADE 36, UNLESS OTHERWISE NOTED

GRAVITY DESIGN

Column "C1"

$$L_o = 80 \text{ psf} \left(.25 + \frac{15}{\sqrt{500(4)}} \right) = 80 \text{ psf} (.59) = 46.8 \text{ psf}$$

Column "C2"

$$L_o = 80 \text{ psf} \left(.25 + \frac{15}{\sqrt{972(4)}} \right) = 80 \text{ psf} (.49) = 40 \text{ psf}$$

Flare Beam "FB1"/"FB4"

$$L_o = 80 \text{ psf} \left(.25 + \frac{15}{\sqrt{1212(4)}} \right) = 80 \text{ psf} (.93) = 75 \text{ psf}$$

Flare Beam "FB3"

$$L_o = 80 \text{ psf} \left(.25 + \frac{15}{\sqrt{1212(4)}} \right) = 80 \text{ psf} (.59) = 47.2 \text{ psf}$$

Flare Beam "FB2"

$$L_o = 80 \text{ psf} \left(.25 + \frac{15}{\sqrt{1212(4)}} \right) = 80 \text{ psf} (.59) = 47.2 \text{ psf}$$

BEAM DESIGN

Project ID:	19051
Project Name:	ATC BLDG 2
Engineer:	JAD
Date:	6/19/2020

Beam # = **FB4**

Shape **W 18x40**

Properties	Geometry	
	A	11.8 <i>in.²</i>
	d	17.9 <i>in.</i>
	b_f	6.018 <i>in.</i>
	t_f	0.524 <i>in.</i>
	t_w	0.316 <i>in.</i>
	W(weight)	40 <i>#/ft.</i>
	r_T	1.54
	d/A_f	5.68

Compact Section Criteria	
b_f/2t_f	5.74
F_y	—
d/t_w	56.6
F_y	52.9 <i>ksi</i>
F_v	20.6 <i>ksi</i>

Elastic Properties					
X-X			Y-Y		
I_x	612	<i>in.⁴</i>	I_y	19.1	<i>in.⁴</i>
S_x	68.4	<i>in.³</i>	S_y	6.34	<i>in.³</i>
r_x	7.21	<i>in.</i>	r_y	1.27	<i>in.</i>
Plastic Modulus					
Z_x	78.4	<i>in.³</i>	Z_y	9.94	<i>in.³</i>

Torsional Constant	
J	0.808 <i>in.⁴</i>

Warping Constant	
C_w	1440 <i>in.⁶</i>

Material Property	
E	29000 <i>ksi</i>
F_y	36 <i>ksi</i>

Note: Input/ verify data in yellow box:

Composite Beam AISC 7th Edition

Note: The following calculation is basic on the "Steel Tips by Steel Committee of California" dated Jan 1987 and Example 1 of Composite Design for building construction in AISC 7ed

Assumption:

Item	Ribs Perpendicular	Ribs Parallel
1. Concrete Area below Top of deck	Neglect	Include
2. Stud Reduction Factor	$\frac{0.85}{(Nr)^{1/2}} \left(\frac{wr}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right)$	$0.6 \left(\frac{wr}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right)$
3. Maximum Stud Spacing	32 in.	Not Specified
4. Deck Welding	16 in.	Not Specified
5 Minimum Width of Rib	2 in.	Depends on Nr

Building info:

Steel			Concrete			Deck		
Beam Length	32	ft.	Weight	145	pcf	Depth hr	3	in.
Beam Spacing	8	ft.	Thickness t	7.5	inches	Conc. thickness tc	4.5	in.
Fy	36	ksi	f'c	3000	psi	Stud Dia.	3/4	in.
			E'c	3122	ksi	Stud Height	4.5	in.
			β_1	0.85		Deck Orientation	Perpendicular	

Uniform Load			Deflection Criteria		
Dead Load	115	psf	Dead Load	L/240	1.6 in.
Live Load	75	psf	Live Load	L/360	1.1 in.
Roof Live Load	0	psf			
Snow Load	0	psf			

Uniform Load W/ Combo	
w(D+L+Lr or S)	190 psf

Max Moment:	kips-in	
M(max Dead)	1413.12	
M(Dead overwrite)	0	<--- input max Dead if you have uniform load only
M(Dead use)	1413.12	
M(max uniform)	2334.72	
M(max overwrite)	0	<--- input max Uniform if you have uniform load only
M(max use)	2334.72	
Max Shear	kips	
V(max Dead)	14.72	
V(max uniform)	24.32	
V(max overwrite)	0	<--- input max Uniform if you have uniform load only
V(max use)	24.32	

Solution Summary:

W 18x40

<---

Change W shapes

Partial Composite:

50%

ASD Result:

	DCR	
Concrete Stress	0.147	
Steel Stress	0.889	
Dead Load Δ	0.770	1.2315 in.
Live Load Δ	0.228	0.2434 in.
Shoring?	Need Shoreing	<---Check this
# of studs	17	at each side of max moment
Camber=	0.750	in.
Overall	GOOD	

Result illustration:			
Beam	Stud	Camber	
W 18x40	34	0.750	
o-----Δ			
24.32 Reaction			24.32 Reaction

$n=E/E_c$	9
-----------	---

1. Effective Width :

$L/4$	96	in.
s	96	in.
$16t+bf$	126.0	in.

Govern b_{eff} : 96.0 in.

2. Required Section Moduli

$M(D+L+S), S_{tr}$	98.3	in.^3
$M(D \text{ only}), S_s$	59.5	in.^3

3. Composite design section properties

a. Moment of Inertia

$Ac \perp$	432
$Ac //$	579.054
$Ac=b(tc)$	432 in.^2
$A'c=Ac/n$	48.00 in.^2
$Y_s=d/2$	8.95 in.
$Y_c=d+hr+tc/2$	23.15 in.

Section:	A	Y	AY	d	Ad^2	I_o
W 18x40	11.8	8.95	105.6	11.40	1533.0	612.0
Concrete	48	23.15	1111	2.80	376.9	81.0
Sum	59.8		1216.8		1909.8	693.0
					$I_{tr}=$	2602.85 in.^4
$Y_b=$	20.35	in.				

b. Section Moduli

			Partial Composite	
$S_{tr}=$	127.92	in.^3	$S_{eff}=$	110.5 in.^3
$S_t=$	515.21	in.^3	$I_{eff}=$	2019.7 in.^4

4. Check Stress

a. Concrete				
$f_c=M/n \cdot S_t$	0.1988	ksi <	1.35	OK
b. Steel				
Total Load, $S_{tr}=$	110.5	ksi >	98.26262626	OK
Deadl Load, $S_s=$	68.4	ksi >	59.47	OK
Web shear, $F_v=$	4.300	ksi <	20	OK

5. Check Deflection

$\Delta = \frac{5wl^4}{384 \cdot EI} = \frac{ML^2}{1920 I}$				
$\Delta_D=$	1.231477	in. <	1.6	OK
$\Delta_L=$	0.243358	in. <	1.1	OK

6. Check if shore is required

$S_{tr} \max=$	107.95	in.^3 >	127.92	Need Shoring
----------------	--------	---------	--------	--------------

7. Shear Connection, Stud

a. Max Stud dia	1.31	in. >	0.75	OK
b. Horizontal shear				
$V_h=0.85 \cdot f'_c \cdot A_c/2$	550.8	kips	Concrete	
$V_h=A_s \cdot F_y/2$	212.4	kips	Steel	<----- Govern

Reduction Factor

$$\frac{0.85}{(Nr)^{1/2}} \left(\frac{wr}{hr} \right) \cdot \left(\frac{H_s}{hr} - 1 \right) \quad \text{or} \quad 0.6 \left(\frac{wr}{hr} \right) \cdot \left(\frac{H_s}{hr} - 1 \right)$$

$Rf \perp =$	0.567
$Rf // =$	1.000
$RF \text{ use} =$	0.567
$q \text{ per stud} =$	11.5000
$q \cdot Rf =$	6.517

Stud Shear Connector	F'_c (From AISC V7 Table 1.11.4) (ksi)		
0.5	5.1	5.5	5.9
0.625	8	8.6	9.2
0.75	11.5	12.5	13.3
0.875	15.6	16.8	18

Number of Studs= 17 At each side of where max moment is

COLUMN DESIGN

Member # = C2 at FDN

Shape W 14x283

Properties	Geometry	
	A	83.3 in. ²
	d	16.74 in.
	b _f	16.11 in.
	t _f	2.07 in.
	t _w	1.29 in.
	W(weight)	283 #/ft.
	r _T	4.46
	d/A _f	0.5

Compact Section Criteria	
b _f /2t _f	3.9
F _y	— ksi
d/t _w	13
F _y	— ksi
F _y	— ksi

Elastic Properties					
X-X			Y-Y		
I _x	3840	in. ⁴	I _y	1440	in. ⁴
S _x	459	in. ³	S _y	179	in. ³
r _x	6.79	in.	r _y	4.17	in.
Plastic Modulus					
Z _x	542	in. ³	Z _y	274	in. ³

Torsional Constant	
J	104 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	19.25 ft.
Beam Spacing (w) =	0 ft.
Tributary Area to Col=	977 ft.^2
K=	1
Unbraced Length =	19.75 ft.

Loading	
Dead Load (D)=	115 psf
Live Load(L)=	240 psf
Roof Live Load(Lr)=	40 psf
Snow Load(S)=	29.7 psf

Excentricity on Column	
ex =	2 in.
ey =	1 in.

Deflection Criteria		
Total Load	L/360=	0.64 in.

Max Moment About Major Axis:	
MD(Max Dead)=	1543.6 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	1543.6 kips-in
ML(Max Live)=	469 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	469 kips-in
M(Lr or S)(Max Live)=	78.2 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	78.2 kips-in
Mu=	2090.8 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	64.3 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	64.3 kips-in
ML(Max Live)=	19.5 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	19.5 kips-in
M(Lr or S)(Max Live)=	3.3 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	3.3 kips-in
Mu=	87.1 kips-in

Max Shear	
VD(Max Dead)=	0 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	0 kips
VL(Max Live)=	0 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	0 kips
V(Lr or S)(Max Live)=	0 kips
V(Lr or S)(Live Overwrite)=	0 kips
V(Lr or S)(Max Use)=	0 kips
Vu=	0 kips

Max Axial Load	
PD(Max Dead)=	112.355 kips
PD(Dead Overwrite)=	771.8 kips
PD(Max Use)=	771.8 kips
PL(Max Live)=	234.48 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	234.48 kips
P(Lr or S)(Max Live)=	39.08 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	39.08 kips
P(E or W)=	0 kips
Pu=	1045.36 kips

Solution Summary:	
Try	W 14x283 <---- Change W shapes

Member	Major Axis Beam Bending	18%
	Beam Shear	0%
	Minor Axis Beam Bending	2%
	Axial	70%
	Deflection	0%
	Combined Bending and Axial	93%
Overall Check		93%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 19.25 ft
K= 1
KL= 19.25 ft
KL/r= 55.4

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 17.9 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 12.5$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (L_b)= 19.75 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	3.9	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	13	13.1	Good
$l_b \leq 76 b_f \sqrt{F_y}$	237	204.1	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d / A_f) F_y}$	237	1106.7	Good
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	3.9	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	3.9	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$$F_{bx} = \text{N/A} \text{ ksi}$$

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$$F_{bx} = 25.2 \text{ ksi}$$

Shear Stress Equation: $F_v = 0.40 F_y$

$$F_v = 14.4 \text{ ksi}$$

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$$F_{by} = 27 \text{ ksi}$$

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	4.6	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	0	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0.5	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.93	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.78	Less than 1.0, Okay

Column Name: **C2**

All Equations can be found on Page 3-99 of AISC 8th edition

Column= **W 14x283**

Column Properties	
b	16.11 in
d	16.74 in

Material Properties	
Footing f'_c =	3 ksi
Fy(Plate)=	36 ksi

Ultimate Axial Load = **1182 kips**

Required Base Plate Area =	$A_{pl} = \frac{P_u}{0.375 f'_c} =$	1050.7	in²
----------------------------	-------------------------------------	---------------	-----------------------

Base Plate Design	
B =	22 in.
N =	22 in.

Actual Bearing Pressure =	$f_p = \frac{P_u}{B * N} =$	2.4	ksi
---------------------------	-----------------------------	------------	------------

Plate Thickness		
$m = \frac{N - 0.95d}{2} =$	3.0485	in
$n = \frac{B - 0.8b}{2} =$	4.556	in
$t = m \sqrt{\frac{f_p}{0.25 F_y}} \leq n \sqrt{\frac{f_p}{0.25 F_y}} =$	2.35	in.

- P = Total column load, kips
- $A = B \times N$ = Area of plate, square inches
- F_b = Allowable bending stress in base plate, ksi
- F_p = Allowable bearing pressure on support, ksi
- f_p = Actual bearing pressure
- f'_c = Compressive strength of concrete, ksi or psi
- t = Thickness of plate, inches

LATERAL DESIGN

Seismic Force Back 1987 → Section 1113

$$V = ZIKCSW \rightarrow 1113.4$$

Where:

$$Z = 2/8 \rightarrow \text{Table 1113.4.1}$$

$$I = 1.25 \rightarrow \text{Table 1113.1}$$

$$K = 1.00 @ BF, 0.67 @ MP \rightarrow \text{Table 1113.4.3}$$

$$CS = 0.14 \rightarrow \text{Section 1113.4.4}$$

$$\begin{aligned} V_{BF} &= 2/8 (1.25) 1.00 (0.14) = .066 W = .066 (12180 u) = 804 u \\ V_{MP} &= 2/8 (1.25) .67 (0.14) = .044 W = .044 (12180 u) = 535 u \end{aligned} \left. \vphantom{\begin{aligned} V_{BF} &= 2/8 (1.25) 1.00 (0.14) = .066 W = .066 (12180 u) = 804 u \\ V_{MP} &= 2/8 (1.25) .67 (0.14) = .044 W = .044 (12180 u) = 535 u \end{aligned}} \right\} \text{Control Both Directions}$$

Force Dist.

$$\text{Roof: } L_n = 1540 u, h_n = 100'$$

$$6^{\text{th}}: L_6 = 1773 u, h_6 = 85'-9"$$

$$5^{\text{th}}: L_5 = 1773 u, h_5 = 73'-1"$$

$$4^{\text{th}}: L_4 = 1773 u, h_4 = 60'-5"$$

$$3^{\text{rd}}: L_3 = 1773 u, h_3 = 47'-9"$$

$$2^{\text{nd}}: L_2 = 1773 u, h_2 = 35'-1"$$

$$1^{\text{st}}: L_1 = 1773 u, h_1 = 18'-9"$$

$$\Sigma W_i = 12180 k \quad \Sigma h_i = 425'-10"$$

$$\Sigma L_i h_i = 722830$$

Braced Frame

$$\begin{aligned} F_r &= V \frac{W_r h_r}{\Sigma W_i h_i} = 804 u \left(\frac{1540 u (100')}{722830} \right) = 171 u \\ F_6 &= V \frac{W_6 h_6}{\Sigma W_i h_i} = 804 u \left(\frac{1773 u (85'-9")}{722830} \right) = 169 u \\ F_5 &= V \frac{W_5 h_5}{\Sigma W_i h_i} = 804 u \left(\frac{1773 u (73'-1")}{722830} \right) = 144 u \\ F_4 &= V \frac{W_4 h_4}{\Sigma W_i h_i} = 804 u \left(\frac{1773 u (60'-5")}{722830} \right) = 119 u \\ F_3 &= V \frac{W_3 h_3}{\Sigma W_i h_i} = 804 u \left(\frac{1773 u (47'-9")}{722830} \right) = 94 u \\ F_2 &= V \frac{W_2 h_2}{\Sigma W_i h_i} = 804 u \left(\frac{1773 u (35'-1")}{722830} \right) = 62 u \\ F_1 &= V \frac{W_1 h_1}{\Sigma W_i h_i} = 804 u \left(\frac{1773 u (18'-9")}{722830} \right) = 37 \end{aligned}$$

$$\Sigma F = 804 u \rightarrow 0 u g$$

Moment Frame

$$\begin{aligned} F_r &= V \frac{W_r h_r}{\Sigma W_i h_i} = 535 u \left(\frac{1540 u (100')}{722830} \right) = 113 u \\ F_6 &= V \frac{W_6 h_6}{\Sigma W_i h_i} = 535 u \left(\frac{1773 u (85'-9")}{722830} \right) = 112 u \\ F_5 &= V \frac{W_5 h_5}{\Sigma W_i h_i} = 535 u \left(\frac{1773 u (73'-1")}{722830} \right) = 95.9 u \\ F_4 &= V \frac{W_4 h_4}{\Sigma W_i h_i} = 535 u \left(\frac{1773 u (60'-5")}{722830} \right) = 79.2 u \\ F_3 &= V \frac{W_3 h_3}{\Sigma W_i h_i} = 535 u \left(\frac{1773 u (47'-9")}{722830} \right) = 62.5 u \\ F_2 &= V \frac{W_2 h_2}{\Sigma W_i h_i} = 535 u \left(\frac{1773 u (35'-1")}{722830} \right) = 46.0 u \\ F_1 &= V \frac{W_1 h_1}{\Sigma W_i h_i} = 535 u \left(\frac{1773 u (18'-9")}{722830} \right) = 24.6 u \end{aligned}$$

$$\Sigma F = 535 u \rightarrow 0 u g$$

Wind Loads

- Exposure B
- Basic wind speed = 90MPH
- $I_w = 1.07$

Effective wind pressures

feet	Pressure
0-20	15 PSF
20-40	17 PSF
40-60	21 PSF
60-100	23 PSF
100-150	27 PSF

Wall pressures

$$C_p = 0.8 \rightarrow P_e$$

$$C_p = -0.5 \rightarrow P_h$$

$$P_x = I_w (P_e + P_h)$$

$$P_1 = 1.07 (15 \text{ PSF} (0.8)) = 12.75 \text{ PSF}$$

$$P_2 = 1.07 (17 \text{ PSF} (0.8)) = 15.6 \text{ PSF}$$

$$P_3 = 1.07 (21 \text{ PSF} (0.8)) = 19.2 \text{ PSF}$$

$$P_4 = 1.07 (23 \text{ PSF} (0.8)) = 21.1 \text{ PSF}$$

$$P_w = 1.07 (27 \text{ PSF} (0.5)) = 15.2 \text{ PSF}$$

Base shear

$$V_w = L (P_1 (h_1)^2 + P_2 (h_2) + P_3 (h_3) + P_4 (h_4) + P_5 (h_5))$$

$$= 124' (12.75 \text{ PSF} (18'-9'')^2 + 15.6 \text{ PSF} (1'-3'') + 19.2 \text{ PSF} (20') + 21.1 \text{ PSF} (40') + 15.2 \text{ PSF} (100 - 18.75''))$$

$$= 360 \text{ k}$$

FOR FRAMING NOTES SEE 52.10
SECOND FLOOR FRAMING PLAN
 1/8" = 1'-0"

This publication is available free of charge from: <https://doi.org/10.6028/NIST.SP.1155-48v2B>

Moment Frame

Braced
Frames

Moment Frame

MOMENT FRAME DESIGN

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GR2.21-917-48V2B>

Example Calc

CONSULTANTS

NIST
National Institute of
Standards and Technology

ATC
Applied Technology Council

PCS
Structural Solutions

**UZUN
+CASE**

PROJECT
**HEALTHCARE
FACILITY**
LONG-RANGE RISK TYPE
PCS STRUCTURAL SOLUTIONS
C202004
199' MRC WITH EQUIVALENT RISK CATEGORY II

DESIGN DRAWINGS
15 DECEMBER, 2020

Job Number	19-021
Checked	Checker
Approved	Approver
TITLE	

**MOMENT FRAME
ELEVATION**

SHEET NUMBER

S6.01

Example Calc

Defl. = 1.13" < .005 h

Envelope Only Solution

SK - 2

Feb 3, 2021 at 2:09 PM

Moment Frame Design 1-20-21.r3d

Beam: **M60**

Shape: **W36x230 ASD8th**

Material: **A36 Gr.36**

Length: **32 ft**

I Joint: **N2**

J Joint: **N10**

Envelope

Code Check: **0.607 (LC 2)**

Report Based On 97 Sections

AISC 9th: ASD Code Check

Max Bending Check **0.607 (LC 2)** Max Shear Check **0.223 (y) (LC 2)** Max Defl Ratio **L/3481**
Location **32 ft** Location **24 ft** Location **9.667 ft**
Equation **H2-1** Span **1**

Compact

Fy	36 ksi	Cm	.6	.85
Fa	20.233 ksi	Lb	8 ft	8 ft
Ft	21.6 ksi	KL/r	25.744	6.445
Fby	27 ksi	Sway	No	No
Fbz	23.76 ksi	L Comp Flange	8 ft	
Fvy	14.4 ksi			
Fvz	14.4 ksi			
Cb	2.3			

Member # = MF 1st Floor Beam

Shape W 36x230

Properties	Geometry	
	A	67.6 in. ²
	d	35.9 in.
	b _f	16.47 in.
	t _f	1.26 in.
	t _w	0.76 in.
	W(weight)	230 #/ft.
	r _T	4.3
	d/A _f	1.73

Compact Section Criteria	
b _f /2t _f	6.5
F _y	— ksi
d/t _w	47.2
F _y	— ksi
F _y	29.6 ksi

Elastic Properties					
X-X			Y-Y		
I _x	15000	in. ⁴	I _y	940	in. ⁴
S _x	837	in. ³	S _y	114	in. ³
r _x	14.9	in	r _y	3.73	in
Plastic Modulus					
Z _x	943	in. ³	Z _y	176	in. ³

Torsional Constant	
J	28.6 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	32 ft.
Beam Spacing (w) =	0 ft.
Tributary Area to Col=	0 ft.^2
K=	1
Unbraced Length =	32 ft.

Loading	
Dead Load (D)=	0 psf
Live Load(L)=	0 psf
Roof Live Load(Lr)=	0 psf
Snow Load(S)=	0 psf

Eccentricity on Column	
ex =	0 in.
ey =	0 in.

Deflection Criteria		
Total Load	L/360=	1.07 in.

Max Moment About Major Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	11830 kips-in
M(Lr or S)(Max Use)=	11830 kips-in
Mu=	11830 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Shear	
VD(Max Dead)=	0 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	0 kips
VL(Max Live)=	0 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	0 kips
V(Lr or S)(Max Live)=	0 kips
V(Lr or S)(Live Overwrite)=	88 kips
V(Lr or S)(Max Use)=	88 kips
Vu=	88 kips

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	19 kips
Pu=	19 kips

Solution Summary:	
Try	W 36x230 <---- Change W shapes

Member	Major Axis Beam Bending	58%
	Beam Shear	22%
	Minor Axis Beam Bending	0%
	Axial	2%
	Deflection	0%
	Combined Bending and Axial	60%
Overall Check		60%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 32 ft
K= 1
KL= 32 ft
KL/r= 102.9

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 12.6 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 0.3$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (L_b)= 32 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	6.5	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	47.2	67.3	Good
$l_b \leq 76 b_f \sqrt{F_y}$	384	208.6	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d / A_f) F_y}$	384	321.1	No Good, Use Alternate EQN
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	6.5	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	6.5	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$$F_{bx} = \text{N/A} \text{ ksi}$$

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$$F_{bx} = 24.4 \text{ ksi}$$

Shear Stress Equation: $F_v = 0.40 F_y$

$$F_v = 14.4 \text{ ksi}$$

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$$F_{by} = 27 \text{ ksi}$$

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	14.1	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	3.2	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.6	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.59	Less than 1.0, Okay

Column: **M1**

Shape: **W14x342 ASD8th**

Material: **A36 Gr.36**

Length: **19.75 ft**

I Joint: **N1**

J Joint: **N2**

Envelope

Code Check: **0.705 (LC 2)**

Report Based On 97 Sections

Dz _____ **in**

Vz _____ **k**

T _____ **k-ft**

My _____ **k-ft**

f(z) _____ **ksi**

AISC 9th: ASD Code Check

Max Bending Check **0.705 (LC 2)**
Location **0 ft**
Equation **H1-2**

Max Shear Check **0.141 (y) (LC 2)**
Location **0 ft**
Max Defl Ratio **L/314**

Compact

Fy **36 ksi**
Fa **17.809 ksi**
Ft **21.6 ksi**
Fby **27 ksi**
Fbz **21.6 ksi**
Fvy **14.4 ksi**
Fvz **14.4 ksi**
Cb **2.3**

	y-y	z-z
Cm	.6	.338
Lb	19.75 ft	19.75 ft
KL/r	55.985	34.026
Sway	No	No
L Comp Flange	19.75 ft	

Member # = MF 1st Floor Column

Shape W 14x398

Properties	Geometry	
	A	117 in. ²
	d	18.29 in.
	b _f	16.59 in.
	t _f	2.845 in.
	t _w	1.77 in.
	W(weight)	398 #/ft.
	r _T	4.61
	d/A _f	0.39

Compact Section Criteria	
b _f /2t _f	2.9
F _y	— ksi
d/t _w	10.3
F _y	— ksi
F _y	— ksi

Elastic Properties					
X-X			Y-Y		
I _x	6000	in. ⁴	I _y	2170	in. ⁴
S _x	656	in. ³	S _y	262	in. ³
r _x	7.16	in	r _y	4.31	in
Plastic Modulus					
Z _x	801	in. ³	Z _y	402	in. ³

Torsional Constant	
J	273 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	19.75 ft.
Beam Spacing (w) =	0 ft.
Tributary Area to Col=	0 ft.^2
K=	1
Unbraced Length =	19.75 ft.

Loading	
Dead Load (D)=	0 psf
Live Load(L)=	0 psf
Roof Live Load(Lr)=	0 psf
Snow Load(S)=	0 psf

Excentricity on Column	
ex =	0 in.
ey =	0 in.

Deflection Criteria		
Total Load	L/360=	0.66 in.

Max Moment About Major Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	13320 kips-in
M(Lr or S)(Max Use)=	13320 kips-in
Mu=	13320 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Shear	
VD(Max Dead)=	0 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	0 kips
VL(Max Live)=	0 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	0 kips
V(Lr or S)(Max Live)=	0 kips
V(Lr or S)(Live Overwrite)=	75 kips
V(Lr or S)(Max Use)=	75 kips
Vu=	75 kips

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	606 kips
Pu=	606 kips

Solution Summary:	
Try	W 14x398 <---- Change W shapes

Member	Major Axis Beam Bending	60%
	Beam Shear	16%
	Minor Axis Beam Bending	0%
	Axial	29%
	Deflection	0%
	Combined Bending and Axial	91%
Overall Check		91%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 19.75 ft
K= 1
KL= 19.75 ft
KL/r= 55

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 17.9 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 5.2$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (L_b)= 19.75 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	2.9	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	10.3	45.6	Good
$l_b \leq 76 b_f \sqrt{F_y}$	237	210.1	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d / A_f) F_y}$	237	1433.6	Good
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	2.9	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	2.9	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$F_{bx} = \text{N/A}$ ksi

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$F_{bx} = 25.5$ ksi

Shear Stress Equation: $F_v = 0.40 F_y$

$F_v = 14.4$ ksi

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$F_{by} = 27$ ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	15.26	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	2.3	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.91	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.84	Less than 1.0, Okay

MF: W36 BM

Input :

Please input or verify the data in yellow box	
Beam Size=	W 36x170
Column Size=	W 14x342
Seat Angle Size=	L 8x8x1
Seat Angle Length (in)	16
Bolt Dia.(in)=	1 1/4
Angle to column bolt number	4
Angle to BM flange bolt number	6
Bolt fy (ksi)	36

Beam	db	36.17	in.	Sx	580	in. ³
	tw	0.68	in.	Ix	10500	in. ⁴
	bf	12.03	in.	tf	1.1	in.
	Zx	668	in. ³	Fyb	36	ksi

Column	dc	17.54	in.	bf	16.36	in.
	tf	2.47	in.			
	tw	1.54	in.			
	k	3.125	in.	Fyc	36	ksi

Angle	tf	2.25	in.	Bolt Type A490X		
	b	3	in.			
	a	3	in.			
	Check if $a < 1.25b$	OK		Fya	36	ksi

Stiffener If Required	tst=	0.75	in	Fy=	36	ksi
	dst=	6.180	in.	Weld Size D	3	/16"
	l=	8.27	in ⁴			

Moment(Service)/(kip-ft)	1020	kip-ft
Reaction(Service)/(kips)	82	kips

ASD Result summary		DCR
Beam	Beam Bending	67%
	Beam Web checked?	Yes, Checked
Angle	Angle to beam	86%
	Angle to column	96%
PryOut	Fastner	96%
	Angle bending	OK
Column	Column checked?	Yes, Checked
	Web shear check	93%
Stiffener	Compression Stiffener	OK, Stiffener Not Required
	Tension Stiffener	OK, Stiffener Not Required

<---Additional check required to determine beam web shear capacity. Use spreadsheet 'shearconnection 1980'

<---Additional check required to determine column bending capacity. Use spreadsheet 'HSS Column_ 1980 or W-Shape Column 1980'

<--- Check Calculation below for detail if stiffener is needed

Check items	Code Ref. AISC 8ed	Result
a. Capacity in bending on beam: $f_b = \frac{M}{S} =$ 15.83 ksi $F_b = 0.66F_y =$ 23.76 ksi b. Beam Web connection Note: beam web connection should be calculation prior to Moment frame detail calculation using spreadsheet "ShearConnection1980"	1.5.1.4.1	OK
c. Flange Connection $T = C = \frac{M}{d} =$ 253.80 kips 1. Angle to beam flange: Single shear per bolt: 49.1 kips Total Shear= 294.6 kips 2. Angle to column Tension per bolt: 66.3 kips Total Tension: 265.2 kips		OK
d. Compute pry out force Q $T @ \text{Each Bolt} = T / \text{Bolt \#} =$ 63.5 Kips $p =$ 4.00 inches $b' =$ 2.38 inches $a' =$ 3 5/8 inches $a' + b' =$ 6.00 inches $d' =$ 1.31 inches $\delta = 1 - (d' \div p) =$ 0.671875 $M = p \times t f^2 \times F_y / 8 =$ 91.125 Kips-in $\alpha = \left(\frac{T b'}{M} - 1 \right) / \delta =$ 0.972969 <1 $B_c = T \left[1 + \frac{\delta * \alpha}{(1 + \delta * \alpha)} \left(\frac{b'}{a'} \right) \right]$ 79.88 Kips $t_f = \left[\frac{8 * B_c * a' * b'}{p * F_y * [a' + \delta \alpha (a' + b')]} \right]^{1/2}$ 2.25 in	Trib Width Per Bolt $b - d / 2$ $a + d / 2$ Bolt Hole Size Ratio of net area and the gross area -Eq (1) of 4-89 Allowable bending moment tributary to tee stub flange -Eq (2) of 4-89 Eq (3) of 4-89 Load per bolt including prying action, Kips -Eq(4) of 4-89 Required thickness of tee-stub flange -Eq(5) of 4-89	OK

Check items	Code Ref. AISC 8ed	Result
Column:		
1. Column check Note: Column calculation prior to Moment frame detail calculation using spreadsheet "Column Design" 2. Column web shear $t_w = \frac{32M}{(A_{bc} * F_y)}$ Where $A_{bc} = d_b * d_c$ 634.42 in.^2 MIN. t _w = 1.429 in. COL. t _w = 1.540 in.	1.5.1.2 Commentary	OK
3. Web crippling check, stiffener needed? Horizontal Force at stiffeners= $P_{bf} = \frac{4}{3} * \frac{M \times 12}{d + \frac{1}{2} * (t_A + t_B)}$ 424.7788 Kips Where t _A =t _B 2.2500 inches $A_{st} \geq [P_{bf} - F_{yc} t(t_b + 5k)] \div F_{yst} =$ -15.7281 in^2 At Compression Flange $d_c > \frac{4100 * t_w^3 * \sqrt{F_{yc}}}{P_{bf}} =$ 211.51 At Tension Flange $t_f < 0.4 * \sqrt{\frac{P_{bf}}{F_{yc}}} =$ 1.374	Formula 1.15-1 1.15-2 1.15-3	OK, Stiffener Not Required OK, Stiffener Not Required
4. If Stiffener is need, w(Stiffener Width)=b/3-(t/2)= 3.240 in, min w(use)= 4 in t>tb/2= 1.125 in Area Furnished, A _{st} = 6 in Width To Thickness Ratio= 5.333333 < 95/√F _y = 15.83333 Stiffener Length= 8 in Min Length= 6.3 in Min Weld Length= $l = \frac{[P_{bf} - F_{yc} t(t_b + 5k)]}{0.928D \times 1.65} =$ -1.237 in		Stiffener not thick enough Stiffener Area OK Stiffener Ratio OK Stiffener Length OK Check Weld Here

MF Base conn

$$M_u = 1110 \text{ k-ft}$$

$$T_u = 80 \text{ k}$$

Total tension on stud

$$T = M_u / d + T_u / 2 = 1110 \text{ k-ft} (12) / 18'' + 80 \text{ k} / 2 = 750 \text{ k}$$

$$V_n / \text{bolt} = 15.6 \text{ k} \text{ for } \frac{3}{8}'' \phi \times 7 \frac{1}{2}'' \text{ Heated stud}$$

$$\# \text{ of Bolts Req'd} = T / V_n = 750 \text{ k} / 15.6 \text{ k/bolt} = 50 \rightarrow \text{use 50 bolts or 5 rows of 10 bolts}$$

BRACED FRAME DESIGN

CONSULTANTS

NIST
National Institute of
Standards and Technology

ATC
Applied Technology Council

PCS
Structural Solutions

**UZUN
+CASE**

PROJECT
**HEALTHCARE
FACILITY**
LONG-RANGE, HIGH-RISK
PCS STRUCTURAL SOLUTIONS
C2020-14
199' HIG WITH EQUIVALENT RISK CATEGORY II

DESIGN DRAWINGS
15 DECEMBER, 2020

Job Number 19-021
Checked Checker
Approved Approver
TITLE

**BRACED FRAME
ELEVATIONS**

SHEET NUMBER

S6.00

10.608 NIST.GCR.21-917-48v2B

Example Calcs

SK - 1

Feb 3, 2021 at 2:03 PM

20' BF Brace Design 1-20-21.r3d

HBrace: **M34**

Shape: **STS10x10x0.375_ASD8th**

Material: **A36 Gr.36**

Length: **22.137 ft**

I Joint: **N1**

J Joint: **N10**

Envelope

Code Check: **0.905 (LC 2)**

Report Based On 97 Sections

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

AISC 9th: ASD Code Check

Max Bending Check **0.905 (LC 2)**
Location **0 ft**
Equation **H2-1**

Max Shear Check **0.000 (z) (LC 2)**
Location **0 ft**
Max Defl Ratio **L/10000**

Compact

Fy **36 ksi**
Fa **16.617 ksi**
Ft **21.6 ksi**
Fby **23.76 ksi**
Fbz **23.76 ksi**
Fvy **14.4 ksi**
Fvz **14.4 ksi**
Cb **1.75**

Cm **.6**
Lb **22.137 ft**
KL/r **66.76**
Sway **No**
L Comp Flange **22.137 ft**

z-z **.6**
22.137 ft
68.188
No

Member # = BF 1 1st Floor Brace

Shape ST S 10x10x0.375

Properties	Geometry	
	A	14.1 in. ²
	d	10 in.
	b	10 in.
	t	0.375 in.
	W(weight)	47.9 #/ft.

Compact Section Criteria	
b/t	26.66666667
h/t	26.66666667

Material Property	
E	29000 ksi
F _y	46 ksi

Elastic Properties					
X-X			Y-Y		
I _x	214	in. ⁴	I _y	214	in. ⁴
S _x	42.9	in. ³	S _y	42.9	in. ³
r _x	3.9	in.	r _y	3.9	in.

Brace Design AISC Eighth Edition

Building info:

Steel Properties	
Column Height =	22.14 ft
Tributary Area to Col=	0 ft^2
K factor =	1

Loading	
Dead Load (D)=	80 psf
Live Load(L)=	60 psf
Roof Live Load(Lr)=	0 psf
Snow Load(S)=	25 psf

Excentricity on Column	
ex =	0 in
ey =	0 in

Max Moment About Major Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	348 kips
Pu=	348 kips

Solution Summary:

Try **ST S 10x10x0.375** Change Tube shapes

Member	Major Axis Beam Bending	0%
	Minor Axis Beam Bending	0%
	Axial	93%
	Combined Bending and Axial	93%
Overall Check		93%

Brace Design AISC Eighth Edition

Column Height= 22.14 ft
 K= 1
 KL= 22.14 ft
 KL/r= 68.1

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	111
$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2
Column in Tension	
$F_a = 0.6F_y$	section 1.5.1.1

Fa = 26.6 ksi Note if Seismic/Wind loads occurs, 1/3 allowable stress will be used

Calculate Stresses in Column

Axial Stress: fa= Pu/A 24.7 ksi Column Okay

Non-Composite Major Axis Beam Design AISC Eighth Edition

Note: All Equations in this section can be found in the AISC Eighth Edition Appendix A

Checks for Typical Major Bending Stress Equation		
Equation	Variable Value	Limit Value
$b/t \leq 238 / \sqrt{F_y}$	26.66666667	35.1
Major Axis Bending Equation: $F_{bx} = 0.6F_y$		Fbx= 36.708 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.6F_y$ Fby = 36.708 ksi

Calculate Stresses in Beam

Major Bending Stress: fbx= Mux/Sx=	0	ksi	Column Okay
Minor Bending Stress: fby= Muy/Sy=	0	ksi	Column Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.93	Less than 1.0, Column Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.89	Less than 1.0, Column Okay

Beam: **M15**
 Shape: **W18x60_ASD8th**
 Material: **A36 Gr.36**
 Length: **20 ft**
 I Joint: **N2**
 J Joint: **N18**
Envelope
 Code Check: **0.112 (LC 2)**
 Report Based On 97 Sections

AISC 9th: ASD Code Check

Max Bending Check	0.112 (LC 2)	Max Shear Check	0.006 (y) (LC 1)	Max Defl Ratio	L/10000
Location	10 ft	Location	10 ft	Location	0 ft
Equation	H1-1			Span	NA

Compact

Fy	36 ksi	Cm	.6	Z-Z	.6
Fa	16.313 ksi	Lb	20 ft	Lb	20 ft
Ft	21.6 ksi	KL/r	71.124	KL/r	16.049
Fby	27 ksi	Sway	No	Sway	No
Fbz	14.393 ksi	L Comp Flange	20 ft	L Comp Flange	20 ft
Fvy	14.4 ksi				
Fvz	14.4 ksi				
Cb	1				

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Member # = **BF 1 1st Floor Beam**

Shape **W 18x60**

Properties	Geometry	
	A	17.6 $in.^2$
	d	18.24 $in.$
	b_f	7.555 $in.$
	t_f	0.695 $in.$
	t_w	0.415 $in.$
	$W(weight)$	60 $\#/ft.$
	r_T	1.96
	d/A_f	3.47

Compact Section Criteria	
$b_f/2t_f$	5.4
F_y	— ksi
d/t_w	44
F_y	— ksi
F_y	34.2 ksi

Elastic Properties					
X-X			Y-Y		
I_x	984	$in.^4$	I_y	50.1	$in.^4$
S_x	108	$in.^3$	S_y	13.3	$in.^3$
r_x	7.47	$in.$	r_y	1.69	$in.$
Plastic Modulus					
Z_x	123	$in.^3$	Z_y	20.6	$in.^3$

Torsional Constant	
J	2.17 $in.^4$

Warping Constant	
C_w	— $in.^6$

Material Property	
E	29000 ksi
F_y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	20 ft.
Beam Spacing (w) =	8 ft.
Tributary Area to Col=	0 ft.^2
K=	1
Unbraced Length =	20 ft.

Loading	
Dead Load (D)=	115 psf
Live Load(L)=	80 psf
Roof Live Load(Lr)=	0 psf
Snow Load(S)=	0 psf

Excentricity on Column	
ex =	0 in.
ey =	0 in.

Deflection Criteria		
Total Load	L/360=	0.67 in.

Max Moment About Major Axis:	
MD(Max Dead)=	552 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	552 kips-in
ML(Max Live)=	384 kips-in
ML(Live Overwrite)=	kips-in
ML(Max Use)=	384 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	936 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Shear	
VD(Max Dead)=	9.2 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	9.2 kips
VL(Max Live)=	6.4 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	6.4 kips
V(Lr or S)(Max Live)=	3.04 kips
V(Lr or S)(Live Overwrite)=	55 kips
V(Lr or S)(Max Use)=	55 kips
Vu=	70.6 kips

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	31.3 kips
Pu=	31.3 kips

Solution Summary:	
Try	W 18x60 <---- Change W shapes

Member	Major Axis Beam Bending	35%
	Beam Shear	65%
	Minor Axis Beam Bending	0%
	Axial	24%
	Deflection	30%
	Combined Bending and Axial	60%
Overall Check		65%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 20 ft
K= 1
KL= 20 ft
KL/r= 142

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 7.4 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 1.8$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (L_b)= 20 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	5.4	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	44	60.7	Good
$l_b \leq 76 b_f \sqrt{F_y}$	240	95.7	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d / A_f) F_y}$	240	159.9	No Good, Use Alternate EQN
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	5.4	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	5.4	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$$F_{bx} = \text{N/A} \text{ ksi}$$

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$$F_{bx} = 24.8 \text{ ksi}$$

Shear Stress Equation: $F_v = 0.40 F_y$

$$F_v = 14.4 \text{ ksi}$$

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$$F_{by} = 27 \text{ ksi}$$

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	8.7	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	9.3	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0.2	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.6	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.43	Less than 1.0, Okay

Column: **M36**

Shape: **W14x145 ASD8th**

Material: **A36 Gr.36**

Length: **19.75 ft**

I Joint: **N31**

J Joint: **N32**

Envelope

Code Check: **1.001 (LC 2)**

Report Based On 97 Sections

Dz _____ in

-0.389 at 19.75 ft

703.327 at 0 ft

155.049 at 0 ft

-0.095 at 0 ft

Vy _____ k

-1.748 at 0 ft

Vz _____ k

34.514 at 19.75 ft

My _____ k-ft

16.471 at 0 ft

3.631 at 0 ft

1.79 at 19.75 ft

f(y) _____ ksi

-1.79 at 19.75 ft

f(z) _____ ksi

AISC 9th: ASD Code Check

Max Bending Check **1.001 (LC 2)**

Location **19.75 ft**

Equation **H1-1**

Max Shear Check

Location

Max Defl Ratio

0.012 (y) (LC 2)

0 ft

L/608

Compact

Fy **36 ksi**
Fa **17.476 ksi**
Ft **21.6 ksi**
Fby **27 ksi**
Fbz **21.6 ksi**
Fvy **14.4 ksi**
Fvz **14.4 ksi**
Cb **1**

	y-y	z-z
Cm	.6	.6
Lb	19.75 ft	19.75 ft
KL/r	59.521	37.451
Sway	No	No
L Comp Flange	19.75 ft	

Member # = BF 1 Col FDN

Shape W 14x145

Properties	Geometry	
	A	42.7 in. ²
	d	14.78 in.
	b _f	15.5 in.
	t _f	1.09 in.
	t _w	0.68 in.
	W(weight)	145 #/ft.
	r _T	4.28
	d/A _f	0.88

Compact Section Criteria	
b _f /2t _f	7.1
F _y	— ksi
d/t _w	21.7
F _y	— ksi
F _y	— ksi

Elastic Properties				
X-X			Y-Y	
I _x	1710	in. ⁴	I _y	677 in. ⁴
S _x	232	in. ³	S _y	87.3 in. ³
r _x	6.33	in	r _y	3.98 in
Plastic Modulus				
Z _x	260	in. ³	Z _y	133 in. ³

Torsional Constant	
J	15.2 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	19.75 ft.
Beam Spacing (w) =	0 ft.
Tributary Area to Col=	0 ft.^2
K=	1
Unbraced Length =	19.75 ft.

Loading	
Dead Load (D)=	107 psf
Live Load(L)=	80 psf
Roof Live Load(Lr)=	40 psf
Snow Load(S)=	29.7 psf

Excentricity on Column	
ex =	0 in.
ey =	0 in.

Deflection Criteria		
Total Load	L/360=	0.66 in.

Max Moment About Major Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	54.18 kips-in
M(Lr or S)(Max Use)=	54.18 kips-in
Mu=	54.18 kips-in

Max Shear	
VD(Max Dead)=	0 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	0 kips
VL(Max Live)=	0 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	0 kips
V(Lr or S)(Max Live)=	0 kips
V(Lr or S)(Live Overwrite)=	0 kips
V(Lr or S)(Max Use)=	0 kips
Vu=	0 kips

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	704 kips
Pu=	704 kips

Solution Summary:	
Try	W 14x145 <---- Change W shapes

Member	Major Axis Beam Bending	0%
	Beam Shear	0%
	Minor Axis Beam Bending	2%
	Axial	94%
	Deflection	0%
	Combined Bending and Axial	98%
Overall Check		98%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 19.75 ft
K= 1
KL= 19.75 ft
KL/r= 59.5

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 17.5 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 16.5$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (Lb)= 19.75 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	7.1	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	21.7	-4.7	No Good, Use Alternate EQN
$l_b / \sqrt{F_y} \leq 76 b_f$	237	196.3	No Good, Use Alternate EQN
$l_b / \sqrt{F_y} \leq \frac{20,000}{(d / A_f)}$	237	635.1	Good
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	7.1	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	7.1	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$$F_{bx} = \text{N/A} \text{ ksi}$$

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$$F_{bx} = 24.2 \text{ ksi}$$

Shear Stress Equation: $F_v = 0.40 F_y$

$$F_v = 14.4 \text{ ksi}$$

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$$F_{by} = 27 \text{ ksi}$$

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	0	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	0	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0.6	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.98	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.79	Less than 1.0, Okay

Braced Frame # **HSS 10x10**

Brace Size: **ST S 10x10x0.375**

Brace Properties	
A	14.1 in ²
t	0.375 in
B	10 in
F _y	46 ksi
Ultimate Axial Load applied to Brace=	355 kips

Weld Properties	
D	8/16"

Gusset Plate Properties	
tp	1.5 in
F _{yp}	36 ksi
E	29000 ksi
K	0.7
L _p	7.5 in
L	12 in
r	0.433 in
KL/r	12.1
C _c	126

Brace Yielding	$P_a = 0.6F_y A_{net} =$	358.1	kips	Okay
----------------	--------------------------	-------	------	------

Required Weld Length	$L_w = \frac{P_a}{0.928D * 4} =$	12	in
----------------------	----------------------------------	----	----

Gusset Plate Yielding	$P_a = 0.6F_{yp} A_p =$	388.8	kips	Okay
-----------------------	-------------------------	-------	------	------

Gusset Plate Buckling	$P_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}]F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3} A_p =$	378.8	kips	Okay
-----------------------	---	-------	------	------

Foundation Cals

$$T_u = 660 \text{ k}$$

$$V_u = 120 \text{ k}$$

Check Shear Wdg

$$\phi V_c = \phi (.85)^2 f'_c A_g = .9 (.85)^2 (345) 6' (8') = 97 \text{ k} > V_u$$

$$P_{\text{Bending}} = \frac{60 \text{ k} (6 \text{ ft})}{\left(\frac{8' (2')^2}{4}\right)} = 22.5 \text{ ksi} < 1.33 (.6) F_y = 1.33 (.6) 36 \text{ ksi} = 28.8 \text{ ksi}$$

Column Weld

$$T_u = 660 \text{ k}$$

Try 3/8" Weld

$$.928 (6) [(4.78' - 1.1' (2)) + 15.5' (2) + (15.5' - .68' (2)) 1.5] = 716 \text{ k} > T_u \rightarrow \text{OK}$$

Check (6) 1-1/2" Ø A490 Anchors

$$T_n = 95.4 \text{ k} (8) = 763 \text{ k} \rightarrow \text{OK}$$

Design Case B

2018 IBC Risk Category III Seismic Design Category B

This section presents representative structural calculations for Building 2 (7-story healthcare facility in Long Island, New York) Design Case B. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v2A.

TABLE OF CONTENTS

DESIGN CRITERIA.....	1
GRAVITY DESIGN.....	11
BEAM DESIGN.....	12
COLUMN DESIGN.....	17
LATERAL DESIGN.....	21
BRACED FRAME DESIGN.....	35
MOMENT FRAME DESIGN.....	46

DESIGN CRITERIA

Project: ATC BLDG 2 RC3 Job Number: 19-051
 Sheet: _____ of _____ Name: JAD
 Originating Office: Tacoma Date: 07/14/21

DESIGN CRITERIA CHECKLIST

CODE: IBC 2018, ASCE 7-16 LOCATION: Long Island, NY

VERTICAL DESIGN CRITERIA

	DEAD	LIVE	PARTITION	CONCENTRATED
ROOF:	100 PSF	40 PSF		
OPERATING ROOMS	95 PSF	60 PSF	15 PSF	1000 #
LIBRARY:	95 PSF	150 PSF	15 PSF	1000 #
PRIVATE ROOMS	95 PSF	100 PSF	15 PSF	
CORRIDORS ABOVE THE 1st FLOOR	95 PSF	80 PSF	15 PSF	

WIND DESIGN CRITERIA

BASIC WIND SPEED (V) = 140 MPH ▼ (Per ASCE 7-16 Sec. 26.5.1, Fig. 26.5-1A; 1B; 1C & 1D, or as required by Bld'g Dept.)
 RISK CATEGORY: III & IV ▼ (Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)
 EXPOSURE CATEGORY: B ▼ (Per ASCE 7-16 Section 26.7.3)
 TOPOGRAPHIC FACTOR (K_{zt}): 1.00 ▼ (Per ASCE 7-16 Section 26.8.2)
 MEAN ROOF HEIGHT: 30 FT ▼ (See ASCE 7-16 Section 26.2 - Definitions)
 ELEVATION: 0 FT ▼ (See ASCE 7-16 Section 26.9)
 ENCLOSURE CLASSIFICATION: Enclosed ▼ (See ASCE 7-16 Section 26.2 & Table 26.13-1)
 ROOF TYPE: Monoslope ▼ (See ASCE 7-16 Figure 27.3-1)
 ROOF SLOPE (θ): 0.00:12 (Enter vertical rise in 12 horizontal units)
 θ (degrees): 0.00

SEISMIC DESIGN CRITERIA

RISK CATEGORY: III ▼ (Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)
 SITE CLASS: D ▼ (Per IBC Section 1613.2.2, Assumed as "D" or per Geotech.)
 IMPORTANCE FACTOR (I_E): 1.25 (Per ASCE 7-16 Table 1.5-2)
 STRUCTURAL SYSTEM (R): 3.5 (Per ASCE 7-16 Table 12.2-1)
 OVERSTRENGTH FACTOR (Ω_o): 3.0 (Per ASCE 7-16 Table 12.2-1)
 INFORMATION BELOW FROM "EARTHQUAKE SPECTRAL RESPONSE ACCELERATION MAPS" PER USGS
 LATITUDE: 40.647 $S_s =$ 0.245 $F_a =$ 1.600
 LONGITUDE: -73.564 $S_1 =$ 0.055 $F_v =$ 2.400

DEFLECTION CRITERIA

FLOOR (LIVE): L/ 480 ▼ ROOF (LIVE): L/ 360 ▼
 FLOOR (TOTAL): L/ 360 ▼ ROOF (TOTAL): L/ 240 ▼
 WALLS: L/ 360 ▼ SPECIAL: L/ ▼

SOIL DESIGN CRITERIA

REPORT: NO **SEE SOILS REPORT FOR ACTIVE, PASSIVE PRESSURES AND FRICTION COEFFICIENT**
 BEARING: 6000 PSF
 ACTIVE: 35 PCF
 PASSIVE: 250 PCF
 COEFFICIENT OF FRICTION: 0.35
 PILE TYPE: NONE
 VERTICAL CAPACITY: N/A
 UPLIFT CAPACITY: N/A
 MINIMUM FOOTING DIMENSIONS:
 CONTINUOUS: 1'-4"
 SPREAD: 1'-6"
 FROST DEPTH: 1'-6"
 LATERAL CAPACITY: N/A
 SIZE: N/A

Project: ATC BLDG 2 RC3

Job Number: 19-051

Sheet: _____ of _____

Name: JAD

Originating Office: Tacoma

Date: 07/14/21

MATERIALS

CONCRETE

Footings/Piles:	3000 PSI	Columns:	4000 PSI
Slabs/Walls:	4000 PSI	Beams:	4000 PSI
-	-	-	-

REINFORCING

Steel Grade = 60 $f_y =$ 60 KSI

STRUCTURAL STEEL

W-Flange Beams	ASTM A992	$f_y =$	50 KSI
Shapes & Plates	ASTM A36	$f_y =$	36 KSI
Pipes	ASTM A53, Grade B	$f_y =$	35 KSI
HSS Rect.	ASTM A500, Grade C	$f_y =$	50 KSI
HSS Round	ASTM A500, Grade C	$f_y =$	46 KSI

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

DESIGN CRITERIA - WIND

FIGURE 27.3-8: Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases per ASCE 7-16

FIGURE 27.3-8 Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases

DESIGN CRITERIA - WIND

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, C_p , for Enclosed and Partially Enclosed Buildings - Walls and Roofs per ASCE 7-16

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, C_p , for Enclosed and Partially Enclosed Buildings—Walls and Roofs

DESIGN CRITERIA - WIND

FIGURE 30.3-1: Components and Cladding [$h \leq 60$ ft]: External Pressure Coefficients, (G_Cp), for Enclosed and Partially Enclosed Buildings - Walls

Diagram

Notation

a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

Exception: For buildings with $\theta = 0^\circ$ to 7° and a least horizontal dimension greater than 300 ft (90 m), dimension a shall be limited to a maximum of $0.8h$.

h = Mean roof height, in ft (m), except that eave height shall be used for $\theta \leq 10^\circ$.

θ = Angle of plane of roof from horizontal, in degrees.

External Pressure Coefficient, (G_Cp) - Walls

Notes

1. Vertical scale denotes (G_Cp) to be used with qh .
2. Horizontal scale denotes effective wind area, in ft^2 (m^2).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (G_Cp) for walls shall be reduced by 10% when $\theta \leq 10^\circ$.

FIGURE 30.3-1 Components and Cladding [$h \leq 60$ ft ($h \leq 18.3$ m)]: External Pressure Coefficients, (G_Cp), for Enclosed and Partially Enclosed Buildings—Walls

DESIGN CRITERIA - SEISMIC

ASCE 7-16 SECTION 12.8 - EQUIVALENT LATERAL FORCE PROCEDURE

RISK CATEGORY:	III	LATITUDE:	40.647
SITE CLASS:	D	LONGITUDE:	-73.564
IMPORTANCE FACTOR (I_E):	1.25	$S_S =$	0.245
STRUCTURAL SYSTEM (R):	3.5	$S_1 =$	0.055
OVERSTRENGTH FACTOR (Ω_o):	3	$F_a =$	1.600
		$F_v =$	2.400

ASCE 7-16 SECTION 11.4 SEISMIC GROUND MOTION VALUES

Section 11.4.4 - Coefficients and Risk-Targeted Maximum Considered Earthquake (MCER) Spectral Response Acceleration Parameters

$$S_{MS} = F_a * S_S = 0.392 \quad S_{M1} = F_v * S_1 = 0.132$$

Section 11.4.5 - Design Spectral Response Acceleration Parameters

$$S_{DS} = 2/3 * S_{MS} = 0.261 \quad S_{D1} = 2/3 * S_{M1} = 0.088$$

ASCE 7-16 SECTION 11.6 - SEISMIC DESIGN CATEGORY - SECTION 12.8.2 - PERIOD DETERMINATION

ASCE 7-16 TABLE 11.6-1			
SEISMIC DESIGN CATEGORY BASED ON S_{DS}			
	RISK CATEGORY:		
	I & II	III	IV
$< 0.167g$	A	A	A
$< 0.33g$	B	B	C
$< 0.50g$	C	C	D
$\geq 0.50g$	D	D	D
	B		

ASCE 7-16 TABLE 11.6-2			
SEISMIC DESIGN CATEGORY BASED ON S_{D1}			
	RISK CATEGORY:		
	I & II	III	IV
$< 0.067g$	A	A	A
$< 0.133g$	B	B	C
$< 0.20g$	C	C	D
$\geq 0.20g$	D	D	D
	B		

Each building and structure shall be assigned to the most severe Seismic Design Category in accordance with Table 11.6-1 or Table 11.6-2, irrespective of the fundamental period of vibration of the structure.

PERIOD DETERMINATION:

$$C_t = 0.028$$

$$h_n = 100 \text{ FT}$$

$$x = 0.8$$

$$T_a = C_t * h_n^x = 1.115$$

ASCE 7-16 SECTION 12.8.1.1 - SEISMIC RESPONSE COEFFICIENT

GENERAL EQUATION: $C_S = S_{DS}/(R/I) = 0.093$ EQ. 12.8-2

MAXIMUM: $C_S = S_{D1}/(T^*(R/I)) = 0.028$ <--CONTROLS EQ. 12.8-3

MINIMUM: $C_S = 0.044 * S_{DS} * I > 0.01 = 0.014$ EQ. 12.8-5

For structures located where $S_1 > 0.6g$
 $C_S = 0.5 * S_1/(R/I) = 0.000$ EQ. 12.8-6

ASCE 7-10 SECTION 12.8.1 - SEISMIC BASE SHEAR

$$V = C_S * W = 0.028 * W$$

W = the total dead load and applicable portion of other loads as indicated in Section 12.7.2

Search Information

Coordinates: 40.647053, -73.5641258
Elevation: 42 ft
Timestamp: 2021-06-29T20:48:41.993Z
Hazard Type: Seismic
Reference Document: ASCE7-16
Risk Category: IV
Site Class: D-default

MCE_R Horizontal Response Spectrum

Design Horizontal Response Spectrum

Basic Parameters

Name	Value	Description
S _S	0.245	MCE _R ground motion (period=0.2s)
S ₁	0.055	MCE _R ground motion (period=1.0s)
S _{MS}	0.392	Site-modified spectral acceleration value
S _M	0.132	Site-modified spectral acceleration value
S _{DS}	0.261	Numeric seismic design value at 0.2s SA
S _{D1}	0.088	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	C	Seismic design category
F _a	1.6	Site amplification factor at 0.2s
F _v	2.4	Site amplification factor at 1.0s
CR _S	0.943	Coefficient of risk (0.2s)
CR	0.939	Coefficient of risk (1.0s)
PGA	0.146	MCE _G peak ground acceleration
F _{PGA}	1.508	Site amplification factor at PGA
PGA _M	0.22	Site modified peak ground acceleration
T _L	6	Long-period transition period (s)
SsRT	0.245	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.26	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.055	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.059	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)

S1D	0.6	Factored deterministic hazard spectral acceleration (2% probability of exceedance in 50 years)
PGAd	0.5	Factored deterministic acceleration value (PGA)

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

Disclaimer

Hazard loads are provided by the U.S. Geological Survey [Seismic Design Web Services](#).

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This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

Search Information

Coordinates: 40.647053, -73.5641258
Elevation: 42 ft
Timestamp: 2021-06-29T16:29:22.950Z
Hazard Type: Wind

ASCE 7-16

MRI 10-Year	75 mph
MRI 25-Year	84 mph
MRI 50-Year	93 mph
MRI 100-Year	99 mph
Risk Category I	110 mph
Risk Category II	122 mph
Risk Category III	▲ 131 mph

If the structure under consideration is a healthcare facility and you are also within 1 mile of the coastal mean high water line, you are in a wind-borne debris region. If other occupancy, use the Risk Category II basic wind speed contours to determine if you are in a wind-borne debris region.

Risk Category IV ▲ 137 mph

You are in a wind-borne debris region if you are also within 1 mile of the coastal mean high water line.

ASCE 7-10

MRI 10-Year	77 mph
MRI 25-Year	87 mph
MRI 50-Year	94 mph
MRI 100-Year	102 mph
Risk Category I	114 mph
Risk Category II	125 mph
Risk Category III-IV	▲ 135 mph

If the structure under consideration is a healthcare facility and you are also within 1 mile of the coastal mean high water line, you are in a wind-borne debris region. If other occupancy, use the Risk Category II basic wind speed contours to determine if you are in a wind-borne debris region.

ASCE 7-05

ASCE 7-05 Wind Speed ▲ 115 mph

You are in a wind-borne debris region if you are also within 1 mile of the coastal mean high water line.

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

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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GRAVITY DESIGN

BEAM DESIGN

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (10-20-20)

Building Code: IBC

Floor Map

10/20/20 10:06:28

Steel Code: AISC 360-16 LRFD

Floor Type: Floor

Beam Numbers

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Beam Design

02/02/21 12:25:21

Steel Code: AISC 360-16 LRFD

Floor Type: Floor**Beam Number = 98****SPAN INFORMATION (ft): I-End (47.25,-123.50) J-End (47.25,-91.50)**Beam Size (User Selected) = W21X50 Fy = 50.0 ksi

Total Beam Length (ft) = 32.00

COMPOSITE PROPERTIES (Not Shored):

		Left		Right
Deck Label		Deck		Deck
Concrete thickness (in)		4.50		4.50
Unit weight concrete (pcf)		150.00		150.00
f _c (ksi)		5.00		5.00
Decking Orientation		perpendicular		perpendicular
Decking type		VERCO W3 Formlok		VERCO W3 Formlok
beff (in)	=	91.50	Y bar(in)	= 23.01
M _{nf} (kip-ft)	=	1038.49	M _n (kip-ft)	= 846.60
C (kips)	=	372.16	PNA (in)	= 20.19
I _{eff} (in ⁴)	=	3120.64	I _{tr} (in ⁴)	= 3986.68
Stud length (in)	=	4.50	Stud diam (in)	= 0.75
R _g Stud Red. Factor	R _g [1] = 1.00		R _g [2] = 0.85	
R _p Stud Red. Factor	R _p [1] = 0.60		R _p [2] = 0.60	
Stud Capacity (kips)	Q _n [1] = 17.2		Q _n [2] = 14.6	
# of studs:	Max = 64	Partial = 48	Actual = 48	
Number of Stud Rows = 2		Percent of Full Composite Action = 50.63		

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.724	0.610	0.610	---	NonR	0.114	0.152
	32.000	0.724	0.610	0.610			0.114	0.152

SHEAR (Ultimate): Max V_u (1.2DL+1.6LL) = 32.45 kips 1.00V_n = 237.12 kips**MOMENTS (Ultimate):**

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	124.9	16.0	0.0	1.00	0.90	412.50
	Init DL	1.4DL	109.3	16.0	---	---		
	Max +	1.2DL+1.6LL	259.6	16.0	---	---	0.90	761.94
Controlling		1.2DL+1.6LL	259.6	16.0	---	---	0.90	761.94

REACTIONS (kips):

	Left	Right
Initial reaction	12.20	12.20
DL reaction	11.59	11.59
Max +LL reaction	11.59	11.59
Max +total reaction (factored)	32.45	32.45

DEFLECTIONS:

			Ratio
Initial load (in)	at 16.00 ft =	-0.504	L/D = 761
Live load (in)	at 16.00 ft =	-0.189	L/D = 2033 > 360 ^{14 of 57} 0.18
Post Comp load (in)	at 16.00 ft =	-0.219	L/D = 1756 > 240 0.14

RAM Structural System

Gravity Beam Design

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Page 2/2

02/02/21 12:25:21

Steel Code: AISC 360-16 LRFD

Net Total load (in)	at 16.00 ft	=	-0.723	L/D =	531	>	240	0.45
---------------------	-------------	---	--------	-------	-----	---	-----	------

Project ID:
Project # :
Bay ID : 1-Floor: B-C , 5-4
Model : ATC BLDG 2 ASCE RC 3 (10-20-20)

VIBRATION ANALYSIS:

Equipment: User Defined
Evaluation Criterion: Sensitive Equipment
Reference: Murray, T.M., Allen, D.E. and Ungar, E.E.,
"Floor Vibrations Due to Human Activity",
AISC Design Guide #11, 1997

Vibrational Velocity: V= 6000. μ -in/sec (User Defined)
Beams are not continuous at girders
Girders are not continuous at columns

EVALUATION:

Walking Speed (Steps/min)	V (μ -in/sec)	Evaluation V(limit) (μ -in/sec)	Criterion
100	26932	> 6000	Not Satisfied
75	5985	<= 6000	Satisfied
50	1609	<= 6000	Satisfied

PARAMETER SUMMARY:

Parameter	Beam	Left Girder	Right Girder	
Member	W21X50	W24X68	W24X68	
Uniform Load, plf	751.5	1644.9	2646.8	
Effective I, in ⁴	4150.8	6410.2	7206.5	
Frequency, Hz	9.21	8.52	7.12	5.63

FRAMING: (User Modified Bay)

Girder Span = 30.50 ft
Beam Spans:
Left = 0.00 ft
Center = 32.00 ft
Right = 20.33 ft

Girders/Walls:
Left -W24X68
Right -W24X68

Beam -W21X50
4 spaces at 91.50 in

Concrete: dc = 7.50 in
f'c = 5.00 ksi
wt. = 150.0 pcf
Deck Height = 3.00 in

Loading: Dead = 4.00 psf
Live = 11.00 psf
Collateral = 0.00 psf

Warning: The bay configuration and/or properties analyzed here have been modified by the user and differ from the actual configuration and/or properties of the bay in the RAM Structural System.

COLUMN DESIGN

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RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Column Design

02/12/21 11:17:28

Steel Code: AISC360-16 LRFD

Story level Level 2, Column Line B-4, Column # 7

Fy (ksi) = 50.00

Column Size

= W14X176

Orientation (deg.) = 0.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	15.67	15.67
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	10.10	10.35
Bottom _____	10.10	10.35

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	545.17	488.28	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 1435.44	0.90Pnx (kip) = 2189.60	Pu/0.90Pnx = 0.656
	0.90Pny (kip) = 1986.72	Pu/0.90Pny = 0.723
	0.90Pn (kip) = 1986.72	Pu/0.90Pn = 0.723

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 3:

	Dead	Live	Roof
Axial (kip) _____	545.17	488.28	0.00
Moments Top Mx (kip-ft) _____	-1.64	-1.64	0.00
My (kip-ft) _____	0.21	0.21	0.00
Bot Mx (kip-ft) _____	-2.05	-16.72	0.00
My (kip-ft) _____	0.27	5.71	0.00

Reverse curvature about X-Axis

Reverse curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 1435.44	0.90*Pn (kip) = 1986.72
Mux (kip-ft) = -29.21	0.90*Mnx (kip-ft) = 1200.00
Muy (kip-ft) = 9.46	0.90*Mny (kip-ft) = 611.25
Rm = 1.00	
Cbx = 1.86	
Cmx = 0.54	Cmy = 0.57
Pex (kip) = 17322.52	Pey (kip) = 6783.30
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.723

Eq H1-1a: $0.723 + 8/9(0.024 + 0.015) = 0.758$ This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Column Design

Page 2/2

02/12/21 11:17:28

Steel Code: AISC360-16 LRFD

Story level Level 1, Column Line B-4, Column # 7

Fy (ksi) = 50.00

Column Size

= W14X176

Orientation (deg.) = 0.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	19.75	19.75
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	10.10	10.35
Bottom _____	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	635.24	578.35	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 1687.64	0.90Pnx (kip) = 2110.42	Pu/0.90Pnx = 0.800
	0.90Pny (kip) = 1808.38	Pu/0.90Pny = 0.933
	0.90Pn (kip) = 1808.38	Pu/0.90Pn = 0.933

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 6:

	Dead	Live	Roof
Axial (kip) _____	635.24	567.03	0.00
Moments Top Mx (kip-ft) _____	-1.63	-1.63	0.00
My (kip-ft) _____	0.21	4.53	0.00
Bot Mx (kip-ft) _____	0.00	0.00	0.00
My (kip-ft) _____	0.00	0.00	0.00

Single curvature about X-Axis

Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 1669.53	0.90*Pn (kip) = 1808.38
Mux (kip-ft) = -4.56	0.90*Mnx (kip-ft) = 1200.00
Muy (kip-ft) = 7.50	0.90*Mny (kip-ft) = 611.25
Rm = 1.00	
Cbx = 1.67	
Cmx = 0.60	Cmy = 0.60
Pex (kip) = 10904.73	Pey (kip) = 4270.17
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.923

Eq H1-1a: $0.923 + 8/9(0.004 + 0.012) = 0.938$ This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

LATERAL DESIGN

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

07/14/21 17:21:05

LOAD CASE: EQ ELF

Seismic ASCE 7-16 Equivalent Lateral Force

Importance Factor: 1.25 TL: 6.00 s

Site Class D: Stiff Soil, Default

Ss: 0.245 g S1: 0.055 g

Use Specified: SDs: 0.261 g SD1: 0.088 g

Risk Category: III Seismic Design Category: B

Provisions for: Force

Ground Level: Base

Dir	Eccent	R	Ta Equation			Building Period-T			
X	+ And -	3.5	Std,Ct=0.028,x=0.80			User Defined			
Y	+ And -	3.3	Std,Ct=0.020,x=0.75			User Defined			
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
X	1.118	1.700	1.150	1.150	0.093	0.027	0.014	0.027	1.325
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
Y	0.634	1.700	0.630	0.630	0.100	0.054	0.014	0.054	1.065

Total Building Weight (kips) = 11286.99

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	79.63	0.00	62.23	-55.23
Level 6	1	86.10	68.26	0.00	62.23	-55.23
Level 5	1	73.43	55.28	0.00	62.23	-55.23
Level 4	1	60.76	43.01	0.00	62.23	-55.23
Level 3	1	48.09	31.55	0.00	62.23	-55.23
Level 2	1	35.42	21.04	0.00	62.23	-55.23
Level 1	1	19.75	9.70	0.00	62.23	-55.23

APPLIED STORY FORCES

Type: EQ_ASCE716_X+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	79.63	0.00
Level 6	86.10	68.26	0.00
Level 5	73.43	55.28	0.00
Level 4	60.76	43.01	0.00
Level 3	48.09	31.55	0.00

RAM Structural System

Bentley

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

Page 2/7

07/14/21 17:21:05

Level 2	35.42	21.04	0.00
Level 1	19.75	9.70	0.00
		<hr/>	<hr/>
		308.46	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	100.35	79.63	0.00	62.23	-67.82
Level 6	1	86.10	68.26	0.00	62.23	-67.82
Level 5	1	73.43	55.28	0.00	62.23	-67.82
Level 4	1	60.76	43.01	0.00	62.23	-67.82
Level 3	1	48.09	31.55	0.00	62.23	-67.82
Level 2	1	35.42	21.04	0.00	62.23	-67.82
Level 1	1	19.75	9.70	0.00	62.23	-67.82

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	79.63	0.00
Level 6	86.10	68.26	0.00
Level 5	73.43	55.28	0.00
Level 4	60.76	43.01	0.00
Level 3	48.09	31.55	0.00
Level 2	35.42	21.04	0.00
Level 1	19.75	9.70	0.00
		<hr/>	<hr/>
		308.46	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	100.35	0.00	141.94	68.57	-61.53
Level 6	1	86.10	0.00	126.60	68.57	-61.53
Level 5	1	73.43	0.00	106.86	68.57	-61.53
Level 4	1	60.76	0.00	87.34	68.57	-61.53
Level 3	1	48.09	0.00	68.09	68.57	-61.53
Level 2	1	35.42	0.00	49.16	68.57	-61.53
Level 1	1	19.75	0.00	26.39	68.57	-61.53

RAM Structural System

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

Loads and Applied Forces

Page 3/7

07/14/21 17:21:05

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	141.94
Level 6	86.10	0.00	126.60
Level 5	73.43	0.00	106.86
Level 4	60.76	0.00	87.34
Level 3	48.09	0.00	68.09
Level 2	35.42	0.00	49.16
Level 1	19.75	0.00	26.39
		0.00	606.38

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	141.94	55.89	-61.53
Level 6	1	86.10	0.00	126.60	55.89	-61.53
Level 5	1	73.43	0.00	106.86	55.89	-61.53
Level 4	1	60.76	0.00	87.34	55.89	-61.53
Level 3	1	48.09	0.00	68.09	55.89	-61.53
Level 2	1	35.42	0.00	49.16	55.89	-61.53
Level 1	1	19.75	0.00	26.39	55.89	-61.53

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	141.94
Level 6	86.10	0.00	126.60
Level 5	73.43	0.00	106.86
Level 4	60.76	0.00	87.34
Level 3	48.09	0.00	68.09
Level 2	35.42	0.00	49.16
Level 1	19.75	0.00	26.39
		0.00	606.38

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RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

Page 4/7

07/14/21 17:21:05

LOAD CASE: Wind

ASCE 7-16

Exposure: B

Basic Wind Speed (mph): 131.0

Apply Directionality Factor, $K_d = 0.85$ Use Topography Factor, $K_{zt} = 1.00$ Ground Elevation Factor, $K_e = 1.00$

Use Natural Frequency for X-Direction (Hz) = 0.55

Use Natural Frequency for Y-Direction (Hz) = 0.75

Gust Factor for Flexible Structures, G: Use Calculated G for X-Dir.

Gust Factor for Flexible Structures, G: Use Calculated G for Y-Dir.

Damping Ratio for Flexible Structures = 0.05

Mean Roof Height (ft): User Defined = 100.00

Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 0.550

Structure is Flexible

Y-Direction: Natural Frequency = 0.750

Structure is Flexible

CpWindward = 0.80 qLeeward (qh) = 36.90 psf

GCpn (Parapet): Windward = 1.50

Leeward = -1.00

Height ft	Kz	Kzt	qz psf	Gust Factor G		CpLeeward		Pressure (psf)	
				X	Y	X	Y	X	Y
100.35	0.989	1.000	36.940	0.868	0.849	-0.498	-0.500	41.623	40.771
86.10	0.947	1.000	35.358	0.868	0.849	-0.498	-0.500	40.525	39.696
73.43	0.905	1.000	33.786	0.868	0.849	-0.498	-0.500	39.433	38.628
60.76	0.857	1.000	32.006	0.868	0.849	-0.498	-0.500	38.197	37.419
48.09	0.802	1.000	29.938	0.868	0.849	-0.498	-0.500	36.760	36.013
35.42	0.735	1.000	27.433	0.868	0.849	-0.498	-0.500	35.021	34.311
19.75	0.622	1.000	23.216	0.868	0.849	-0.498	-0.500	32.092	31.446
0.00	0.575	1.000	21.461	0.868	0.849	-0.498	-0.500	30.873	30.254

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_X

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	36.99	0.00	62.25	-61.75
Level 6	1	86.10	68.68	0.00	62.25	-61.75
Level 5	1	73.43	62.83	0.00	62.25	-61.75
Level 4	1	60.76	60.84	0.00	62.25	-61.75
Level 3	1	48.09	58.53	0.00	62.25	-61.75
Level 2	1	35.42	61.94	0.00	62.25	-61.75
Level 1	1	19.75	71.98	0.00	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_1_X

Level	Ht	Fx	Fy
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25 of 57

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

Page 5/7

07/14/21 17:21:05

Loads and Applied Forces

	ft	kips	kips
Roof	100.35	36.99	0.00
Level 6	86.10	68.68	0.00
Level 5	73.43	62.83	0.00
Level 4	60.76	60.84	0.00
Level 3	48.09	58.53	0.00
Level 2	35.42	61.94	0.00
Level 1	19.75	71.98	0.00
		<hr/> 421.79	<hr/> 0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	36.52	62.25	-61.75
Level 6	1	86.10	0.00	67.81	62.25	-61.75
Level 5	1	73.43	0.00	62.04	62.25	-61.75
Level 4	1	60.76	0.00	60.08	62.25	-61.75
Level 3	1	48.09	0.00	57.79	62.25	-61.75
Level 2	1	35.42	0.00	61.17	62.25	-61.75
Level 1	1	19.75	0.00	71.09	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_1_Y

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	36.52
Level 6	86.10	0.00	67.81
Level 5	73.43	0.00	62.04
Level 4	60.76	0.00	60.08
Level 3	48.09	0.00	57.79
Level 2	35.42	0.00	61.17
Level 1	19.75	0.00	71.09
		<hr/> 0.00	<hr/> 416.49

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_2_X+E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	27.74	0.00	62.25	43.71
Level 6	1	86.10	51.51	0.00	62.25	-43.71

RAM Structural System

Bentley

Loads and Applied Forces

RAM Frame 17.00.01.09

Page 6/7

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

07/14/21 17:21:05

Level 5	1	73.43	47.12	0.00	62.25	-43.71
Level 4	1	60.76	45.63	0.00	62.25	-43.71
Level 3	1	48.09	43.90	0.00	62.25	-43.71
Level 2	1	35.42	46.46	0.00	62.25	-43.71
Level 1	1	19.75	53.98	0.00	62.25	-43.71

APPLIED STORY FORCES

Type: Wind_ASCE716_2_X+E

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	27.74	0.00
Level 6	86.10	51.51	0.00
Level 5	73.43	47.12	0.00
Level 4	60.76	45.63	0.00
Level 3	48.09	43.90	0.00
Level 2	35.42	46.46	0.00
Level 1	19.75	53.98	0.00
		316.34	0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_2_X-E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	27.74	0.00	62.25	-79.79
Level 6	1	86.10	51.51	0.00	62.25	-79.79
Level 5	1	73.43	47.12	0.00	62.25	-79.79
Level 4	1	60.76	45.63	0.00	62.25	-79.79
Level 3	1	48.09	43.90	0.00	62.25	-79.79
Level 2	1	35.42	46.46	0.00	62.25	-79.79
Level 1	1	19.75	53.98	0.00	62.25	-79.79

APPLIED STORY FORCES

Type: Wind_ASCE716_2_X-E

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	27.74	0.00
Level 6	86.10	51.51	0.00
Level 5	73.43	47.12	0.00
Level 4	60.76	45.63	0.00
Level 3	48.09	43.90	0.00
Level 2	35.42	46.46	0.00
Level 1	19.75	53.98	0.00

Seismic

Site Class info

Site Class D $\rightarrow S_{ws} = 0.261$

$V_{ult} = 131 \text{ MPH}$

Seismic

BF

$R = 3/4$
 $C_s = \frac{S_{ws}}{R} (I_e) = \frac{0.261}{3/4} (1.25) = 0.10$

$W = 100 \text{ psf} (1.5630 \text{ ft}^2) + (9.5 \text{ psf} + 10 \text{ psf}) (1.5630 \text{ ft}^2) 6 = 11410 \text{ lb}$

$V_{Fa} = C_s W = 0.10 (11410 \text{ lb}) = 1150 \text{ lb}$

MF

$R = 3/4$
 $C_s = \frac{S_{ws}}{R} (I_e) = \frac{0.261}{3/4} (1.25) = 0.093$

$W = 100 \text{ psf} (1.5630 \text{ ft}^2) + (9.5 \text{ psf} + 10 \text{ psf}) (1.5630 \text{ ft}^2) 6 = 11410 \text{ lb}$

$V_{Fa} = C_s W = 0.093 (11410 \text{ lb}) = 1065 \text{ lb}$

Check min & max C_s

$T = C_t h^x = 0.28 (100)^{.8} = 1.15 \text{ @ MF}$
 $= \quad \quad = 0.2 (100)^{.75} = 0.63 \text{ @ BF} \quad \left. \vphantom{\begin{matrix} T = C_t h^x \\ = \quad \quad \end{matrix}} \right\} < T_L = 6 \text{ sec}$

$\text{Max } C_s = \frac{S_{ws}}{T(C_t T_L)} = \frac{0.088}{1.15 (2.5/1.15)} = 0.028 \text{ @ MF} \rightarrow V = 0.028(W) = 315 \text{ lb}$
 $= \quad \quad = \frac{0.088}{.63 (2.5/1.15)} = 0.054 \text{ @ BF} \rightarrow V = 0.054(W) = 617 \text{ lb} \rightarrow \text{Seismic controlled @ BF}$

$\text{Min } C_s = 0.044 S_{ws} I_e = 0.044 (0.261) 1.25 = 0.014$

Wind Loads

- Exposure B
- Ultimate wind speed = 121 MPH
- $I_w = 1.0$

$$q_z = .00256 k_z k_{zt} k_d k_e V^2 = .00256 k_z (85) (1.0) 121^2 = 37.7 \text{ psf } k_z$$

Where:

k_{zt} = varies

$$k_{zt} = 1.0$$

$$k_d = 0.85$$

$$k_e = 1.0$$

q_z as a function of height

$$q_{15} \rightarrow k_z = .57 \rightarrow q_{15} = 21.7 \text{ psf}$$

$$q_{20} \rightarrow k_z = 0.62 \rightarrow q_{20} = 23.2 \text{ psf}$$

$$q_{25} \rightarrow k_z = 0.66 \rightarrow q_{25} = 24.6 \text{ psf}$$

$$q_{30} \rightarrow k_z = 0.70 \rightarrow q_{30} = 26.1 \text{ psf}$$

$$q_{40} \rightarrow k_z = 0.76 \rightarrow q_{40} = 28.3 \text{ psf}$$

$$q_{50} \rightarrow k_z = 0.81 \rightarrow q_{50} = 30.2 \text{ psf}$$

$$q_{60} \rightarrow k_z = 0.85 \rightarrow q_{60} = 31.7 \text{ psf}$$

$$q_{70} \rightarrow k_z = 0.89 \rightarrow q_{70} = 33.3 \text{ psf}$$

$$q_{80} \rightarrow k_z = 0.93 \rightarrow q_{80} = 34.7 \text{ psf}$$

$$q_{90} \rightarrow k_z = 0.96 \rightarrow q_{90} = 35.8 \text{ psf}$$

$$q_{100} \rightarrow k_z = 0.99 \rightarrow q_{100} = q_h = 37.0 \text{ psf}$$

Wind Pressure

$$P_e = q_z G C_p - q_h G C_{pe} = G (q_z C_p - q_h C_{pe})$$

Where:

G = Gust Factor $\rightarrow$ see next page $\rightarrow$ ASCE 7-16 26.11

C_p = 0.8 For windward, -0.5 For Leeward

Gust Factor For MF

Natural Frequency

Left = 124' < 4 times h → use approx Eqn 26.11-2

$$\eta = \frac{22.2}{h^{.8}} = \frac{22.2}{100^{.8}} = .55 < 1 \rightarrow \text{Flexible}$$

Determine Gust Factor

$$I_z = C \left(\frac{z}{z_0} \right)^{.6} = .3 \left(\frac{33}{600} \right)^{.6} = .272$$

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{0.4h}{L_z} \right)^{.63}}} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{124 + 100}{390.6} \right)^{.63}}} = 0.83$$

Where:

$$L_z = 1.6 \left(\frac{z}{33} \right)^{.8} = 320 \left(\frac{60}{33} \right)^{.8} = 390.6'$$

$$g_R = g_V = 3.4$$

$$g_R = \sqrt{2 \ln(3600h)} + \frac{0.577}{\sqrt{2 \ln(3600h)}} = 4.04$$

$$R = \sqrt{\frac{1}{2} R_h R_b R_o (.53 + 4.7 R_z)} = \sqrt{\frac{1}{2} (.083) (.207) (.259) (.53 + 4.7 (.087))} = 0.275$$

Where:

$$V_z = \bar{V} \left(\frac{z}{33} \right)^{.08} \left(\frac{88}{60} \right)^{.1}$$

$$= 0.45 \left(\frac{60}{33} \right)^{.4} \left(\frac{88}{60} \right)^{.125} = 95.8$$

$$N_1 = \frac{\eta L_z}{V_z} = \frac{.55 (390.6)}{95.8} = 2.24$$

$$R_h = \frac{7.47 N_1}{(1 + 0.3 N_1)^{.53}} = \frac{7.47 (2.24)}{(1 + 0.3 (2.24))^{.53}} = .083$$

$$\eta_h = 4.6 \eta h / V_z = 4.6 (.55) / 95.8 = 2.64 \rightarrow R_h = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .207$$

$$\eta_b = 4.6 \eta h / V_z = 4.6 (.55) / 95.8 = 3.27 \rightarrow R_b = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .259$$

$$\eta_L = \frac{15.4 \eta L}{V_z} = \frac{15.4 (.55) 124}{95.8} = 10.96 \rightarrow R_L = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .087$$

$$G_F = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_R^2 Q^2 + g_o^2 R^2}}{1 + 1.7 g_o I_z} \right) = 0.925 \left(\frac{1 + 1.7 (.272) \sqrt{3.4^2 (.80)^2 + 4.04^2 (.275)^2}}{1 + 1.7 (3.4) (.272)} \right) = 0.84$$

Gust Factor For DF

Natural Frequency

Left = 124' < 4 times h → use approx Eqn 26.11-2

$$n = \frac{75}{h} = \frac{75}{100} = 0.75 \rightarrow \text{Flexible}$$

Determine Gust Factor

$$I_z = C \left(\frac{z}{z_0} \right)^{1/6} = .3 \left(\frac{33}{600} \right)^{1/6} = .272$$

$$Q = \sqrt{\frac{1}{1 + 0.65 \left(\frac{0.4h}{L_z} \right)^{6.5}}} = \sqrt{\frac{1}{1 + 0.65 \left(\frac{124 + 100}{390.6} \right)^{6.5}}} = 0.83$$

Where:

$$L_z = 1.6 \left(\frac{z}{33} \right)^{.5} = 320 \left(\frac{60}{33} \right)^{.5} = 390.6'$$

$$g_R = g_V = 2.4$$

$$g_R = \sqrt{2 \ln(3600h)} + \frac{0.577}{\sqrt{2 \ln(3600h)}} = 4.12$$

$$R = \sqrt{\frac{1}{R_h R_b R_o} (.53 + 4.7 R_z)} = \sqrt{\frac{1}{.05 (.069) .239 (.199) (.53 + 4.7 (.065))}} = 0.191$$

Where:

$$V_z = \bar{V} \left(\frac{z}{33} \right)^{.5} \left(\frac{88}{60} \right)^{.5} V$$

$$= 0.45 \left(\frac{60}{33} \right)^{.5} \left(\frac{88}{60} \right)^{.5} 125 = 95.8$$

$$N_1 = \frac{n L_z}{V_z} = \frac{.75 (390.6)}{95.8} = 3.06$$

$$R_h = \frac{7.47 N_1}{(1 + 0.3 N_1)^{.53}} = \frac{7.47 (3.06)}{(1 + 0.3 (3.06))^{.53}} = .069$$

$$N_h = 4.6 n h / V_z = 4.6 (.75) 100 / 95.8 = 3.6 \rightarrow R_h = \frac{1}{N} - \frac{1}{2} \frac{1}{N^2} (1 - e^{-N}) = .0239$$

$$N_b = 4.6 n h / V_z = 4.6 (.75) 124 / 95.8 = 4.466 \rightarrow R_b = \frac{1}{N} - \frac{1}{2} \frac{1}{N^2} (1 - e^{-N}) = .0199$$

$$N_L = \frac{15.4 n L}{V_z} = \frac{15.4 (.75) 124}{95.8} = 14.95 \rightarrow R_L = \frac{1}{N} - \frac{1}{2} \frac{1}{N^2} (1 - e^{-N}) = .065$$

$$G_F = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_R^2 Q^2 + g_o^2 R^2}}{1 + 1.7 g_o I_z} \right) = 0.925 \left(\frac{1 + 1.7 (.272) \sqrt{3.4^2 (.80)^2 + 4.12^2 (.191)^2}}{1 + 1.7 (.34) (.272)} \right) = 0.85$$

Effective Wind Pressures

<u>feet</u>	<u>Pressure</u>
0-15	30.2 psf
15-20	31.5 psf
20-25	32.5 psf
25-30	32.5 psf
30-40	35.0 psf
40-50	36.2 psf
50-60	37.7 psf
60-70	38.3 psf
70-80	39.3 psf
80-90	40.1 psf
90-100	40.9 psf

Base Shear

$$\begin{aligned}
 V_L &= B(P_s(15') + 5'(P_{20} + P_{25} + P_{30}) + 10'(P_{40} + P_{50} + P_{60} + P_{70} + P_{80} + P_{90} + P_{100})) \\
 &= 124'-0" (30.2 \text{ psf}(15') + 5'(31.5 \text{ psf} + 32.5 \text{ psf} + 32.5 \text{ psf}) + 10'(35 \text{ psf} + 36.2 \text{ psf} + 37.7 \text{ psf} + 38.3 \text{ psf} + 39.3 \text{ psf} + 40.1 \text{ psf}) \\
 &\quad + 10'(40.9 \text{ psf})) \\
 &= 448k < V_{Fa} \rightarrow \text{Wind Controlled @ MF}
 \end{aligned}$$

Plate

P₁

1 = -74.8 psf

2 = 27.7 psf

P₂

1 = -117.5 psf

2 = 22.7 psf

P₃

1 = -160.4 psf

2 = 22.7 psf

P₄

1 = -62.6 psf

2 = 55.9 psf

P₅

1 = -92.8 psf

2 = 55.9 psf

EAf = .781

Reduction Factor (Positive)

Effective Wind Area	1	2	3	4	5
10	1.0	1.0	1.0	1.0	1.0
20	1.0	.98	.95	.96	.93
50	1.0	.96	.88	.92	.84
100	1.0	.94	.82	.88	.76
500	1.0	.9	.7	.8	.6

C & C Positive Pressure

Effective Wind Area	1	2	3	4	5
10	17.8	17.8	17.8	42.7	42.7
20	17.8	17.4	16.9	41.9	40.6
50	17.8	17.0	15.6	40.2	36.7
100	17.8	16.7	14.6	38.4	32.2
500	17.8	16.0	12.4	34.9	26.2

Plank

<u>P₁</u>	<u>P₂</u>	<u>P₃</u>	<u>P₄</u>	<u>P₅</u>
1 = -74.8 psf	1 = -117.5 psf	1 = -160.4 psf	1 = -60.6 psf	1 = -93.8 psf
2 = 27.7 psf	2 = 22.7 psf	2 = 22.7 psf	2 = 55.9 psf	2 = 55.9 psf
EAF = .781				

Reduction Factors (negative)

<u>Effective Wind Area</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>
10	1.0	1.0	1.0	1.0	1.0
20	.96	.96	.96	.95	.95
50	.92	.92	.92	.88	.88
100	.88	.88	.88	.82	.82
500	.8	.8	.8	.7	.7

C&C Negative Pressures

<u>Effective Wind Area</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>
10	-58.4	-91.8	-125.2	-47.7	-73.7
20	-56.1	-88.1	-119.7	-45.0	-69.6
50	-53.7	-84.5	-115.7	-41.6	-64.5
100	-51.4	-80.7	-110	-38.9	-60.1
500	-46.8	-73.5	-100.2	-33.2	-51.3

BRACED FRAME DESIGN

[illegible]

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Member Code Check

02/12/21 11:43:38

Steel Code: AISC360-16 LRFD

BRACE INFORMATION:

Story Level = Level 1 Frame Number = 1 Brace Number = 2
 Fy (ksi) = 50.00
 Brace Size = HSS8X8X1/2

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	22.25	22.25
Lu for Bending (ft) _____	22.25	22.25
K _____	1.00	1.00

CONTROLLING BRACE FORCES - SHEAR

Load Combination: 1.400 D

Shear	Top	Vux (kip) _____	0.00
		Vuy (kip) _____	0.00
Shear	Bot.	Vux (kip) _____	0.00
		Vuy (kip) _____	0.00

SHEAR CHECK:

Vux (kip) = 0.00	0.90Vnx (kip) = 165.85	Vux/0.90Vnx = 0.000
Vuy (kip) = 0.00	0.90Vny (kip) = 165.85	Vuy/0.90Vny = 0.000

CONTROLLING BRACE FORCES - AXIAL

Load Combination: 1.260 D + 0.500 Lp + 1.000 E4

AXIAL CHECK:

Pu (kip) = 314.28	0.90Pnx (kip) = 345.96	Pu/0.90Pnx = 0.908
	0.90Pny (kip) = 345.96	Pu/0.90Pny = 0.908
	0.90Pn (kip) = 345.96	Pu/0.90Pn = 0.908

CONTROLLING BRACE FORCES - FLEXURE

Load Combination: 1.260 D + 0.500 Lp + 1.000 E4

Axial	Load (kip) _____	314.28
Moment	Top	Mux (kip-ft) _____
		Muy (kip-ft) _____
Moment	Bot.	Mux (kip-ft) _____
		Muy (kip-ft) _____

CALCULATED PARAMETERS:

Pu (kip) = 314.28	0.90Pn (kip) = 345.96
Mux (kip-ft) = 0.00	0.90Mnx (kip-ft) = 140.63
Muy (kip-ft) = 0.00	0.90Mny (kip-ft) = 140.63
KL/Rx = 87.75	KL/Ry = 87.75
Cbx = 1.00	

INTERACTION EQUATION:

Pu/0.90*Pn=0.908

Pr/Pc = 0.908

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RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

02/12/21 11:43:38

Building Code: IBC

Steel Code: AISC341-16 - LRFD

Brace Parameters

Story: Level 1

Frame No: 1

Member No: 2

Fy (ksi): 50.00

Size: HSS8X8X1/2

Frame Type: Ordinary Concentrically Braced Frame

F1.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf = 14.20 Limit = 15.47 OK

Web h/tw = 14.20 Limit = 15.47 OK

F1.6a Diagonal Brace Connections

Maximum Compression force (kip) = 550.64 Seismic Load Combo: 1.260 D + 0.500 Lp + 2.000 E4

Expected Compression force (kip) = 489.90

Maximum Tension force (kip) = 434.58 Seismic Load Combo: 0.840 D - 2.000 E4

Expected Tension force (kip) = 945.00

Bolt Slip Compression (kip) = 314.28 Standard Load Combo: 1.260 D + 0.500 Lp + 1.000 E4

Bolt Slip Tension (kip) = 198.22 Standard Load Combo: 0.840 D - 1.000 E4

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20'-0" Braced Frame

1st DM

$$T_u = \min(-\Omega T, P_u F_y A_g) = 360k$$

$$P_u = 0.3 P_n = 0.3 (64 k) = 186k$$

$$T_L = \min(-\Omega T, P_u F_y A_g) = 44k$$

$$P_L = 0.3 P_n = 0.3 (44k) = 148k$$

Resultant Forces

$$N = (T_u - P_u) \sin(\theta_1) + (P_L - T_L) \sin(\theta_2) = (360k - 186k) \sin(51.5) + (148k - 44k) \sin(62.3) = 106 k \uparrow$$

$$V = (P_u + T_u) \cos(\theta_1) - (P_L + T_L) \cos(\theta_2) = (360k + 186k) \cos(51.5) - (148k + 44k) \cos(62.3) = 77 k$$

$$M_F = \frac{NL}{8} = \frac{106k(20')}{8} = 265 k \cdot ft$$

2nd order Effects

β_1

$$\beta_1 = \frac{C_m}{1 - P_r/P_{e1}} \geq 1 = \frac{1.0}{1 - \frac{130k}{3697k}} = 1.04$$

Where:

$$P_r = 130k$$

$$P_{e1} = \frac{\pi^2 EI_y}{(L_{e1})^2} = \frac{\pi^2 (29,000 ksi) (186 in^4)}{(120')^2} = 3697k$$

β_2

$$\beta_2 = \frac{1}{1 - \frac{P_{strong}}{P_{weak}}} \geq 1 = \frac{1}{1 - \frac{3783k}{63545k}} = 1.06$$

Where:

$$P_{strong} = P_m \frac{HL}{\Delta H} = .85 \frac{411 k (14'-3")}{1.27'} = 63545k$$

$$P_{weak} = (1.2(95 kft) + 1.6(30 kft)) 15620 ft^2 = 3783 k$$

20'-0" Braced Frame

1" DM

Max Load Case

$$P_u = P_r P_E = 1.04 (77k) = 80k$$

$$M_u = D_1 (1+25\%) M_0 + \beta_1 (.5) M_L + \beta_2 M_E = 1.04 (1.06) 32.2 k-ft + 1.04 (.5) 27.2 k-ft + 1.06 (265 k-ft) \\ = 330 k-ft$$

$$\frac{P_u}{\phi P_n} + \left(\frac{M_u}{\phi M_n} \right) = \frac{80 k}{2 (1120 k)} + \left(\frac{330 k-ft}{729 k-ft} \right) = .49 < 1.0$$

Try W16x67

$$\frac{80 k}{2 (74 k)} + \frac{330 k-ft}{474 k-ft} = 0.75 \rightarrow \text{OK}$$

RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

02/12/21 11:43:38

Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Level 1 Frame No: 1 Member No: 22

Fy (ksi): 50.00 Size: W14X120

Frame Type: Ordinary Concentrically Braced Frame

D1.4a Required Strength --- OK

Compression: Max Pu (kip) = 1122.67 --- Combination: 1.260 D + 0.500 Lp + 2.000 E4Max Pu/ ϕ Pn = 0.95 OK*Tension:* Max Pu (kip) = 805.74 --- Combination: 0.840 D - 2.000 E4Max Pu/ ϕ Pn = 0.51 OK

D2.5b Column Splices - Required Strength

*Design strength of column splices must meet or exceed the following forces:**Required tension and compression strength from D1.4a.**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.*

D2.5c Required Shear Strength

	Major	Minor
Mpc (kip-ft)	883.33	425.00

Column Splice Shear Force Required Mpc / H

Where Mpc is the lesser nominal plastic flexural strength of the column sections.

See code for more information on the Required Shear Strength

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HSS 8x8x1/4

$$R_u = 427k$$

Brace to Gusset conn:

Brace weld: $\phi R_n = 1.392(5)(18'') = 445k > 427k \rightarrow 16' \text{ min}$

Net section rupture: $\phi R_n = \phi A_n V F_u = .75(121 \text{ in}^2)(83(62 \text{ ksi})) = 460k > R_u \rightarrow OK$

Where:

$$A_n = 13.5 \text{ in}^2 - (2)(1.34)(0.465'') = 11.87 \text{ in}^2$$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{3''}{16''} = .813$$

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{8^2 + (2)(8^2)}{4(8+8)} = 3'' \quad , \quad L = 16''$$

Shear yield: $\phi R_n = \phi (6) F_y (t) L_w (4) = 1.0 (6)(50 \text{ ksi})(.465'')(16'') (4) = 893k > P_u$

Shear rupture: $\phi R_n = \phi F_u (t) L_w (4) = (0.75)(6)(62 \text{ ksi})(.465'')(16'') (4) = 832k > P_u$

Gusset PL

Buckling: $k_y r = \frac{.7(10'')}{1.5 \sqrt{12}} = 16 < 25$

$$\phi P_n = \phi F_y A_g = .9(36 \text{ ksi})(10'')(1.5'') = 486k$$

HSS 8x8x1/4

Support @ Beam

$$N_{u6} = 494 \text{ k} \cos(62^\circ) = 232 \text{ k}$$

$$M_{u6} = 232 \text{ k} (16/12) = 1856 \text{ k-in}$$

$$N_{u6} = \frac{4(1856 \text{ k-in})}{17.5^\circ} = 424 \text{ k}$$

Tension Yield: $\phi R_n = \phi F_y t(L) = .9(36 \text{ ksi})(1/4)(17.5) = 992 \text{ k} > N_{u6}$

Shear Yield: $\phi R_n = \phi .6 F_y t(L) = 1.0(.6)(36 \text{ ksi})(1/4)(17.5) = 661.5 \text{ k} > V_{u6}$

Bolted Conn. @ Gable

Use 1" A490 Bolts

$$\phi R_n = \phi F_u A_b = .75(68 \text{ ksi})(1")^2 \frac{\pi}{4} = 40 \text{ k}$$

$$n = R_u / \phi R_n / \text{bolt} = 427 / 40 / 60 \text{ k} = 10.7 \rightarrow \text{OK (12) 1" A490 N Bolts}$$

Check 1-1/4 Kurt R 4/9 Bolt

Bearing & Shear

$$R_n = (2.4 d t F_u(n_b) + 1.2 L t F_u(n_s)) = 2.4(1") (1/4) (58 \text{ ksi})(9) + 1.2(1.5") (1/4) (58 \text{ ksi})(3) = 2350 \text{ k}$$

$$\phi R_n = .75(2350 \text{ k}) = 1761 \text{ k} > R_u \rightarrow \text{OK}$$

Tension Yielding: $\phi R_n = \phi F_y A_g = .9(36 \text{ ksi})(1/4)(9) = 428 \text{ k} > R_u$

Tension Rupture: $\phi R_n = \phi F_u A_n = .75(58 \text{ ksi})(1.5)(9 - (3)(1/4)) = 316 \text{ k} < R_u \rightarrow 1-3/4"$

HSS 8x8

Support @ Column

$$\alpha = 97.5^\circ, \beta = 14.5^\circ \left. \begin{array}{l} e_b = 3' \\ e_c = 7' \end{array} \right\} r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = 28.1'$$

$$V_{uc} = \frac{P}{r} P_u = \frac{14.5}{28.1'} (427k) = 220k$$

$$N_{uc} = \frac{e_c}{r} P_u = \frac{7}{28.1'} (427k) = 106k$$

$$R_u = \sqrt{220k^2 + 106k^2} = 244k$$

$$\# bolts = 244k / 40k / bolt = 6.1 \text{ bolts} \rightarrow \text{use } (7) \text{ bolts}$$

$$\text{Bolting} \rightarrow \phi R_n = (10k / in) = 10k (24) = 240k$$

$$\text{Tension} \rightarrow \phi R_n = 76.7 k / in = 76.7 k (24) = 1840k$$

$$\text{Shear} \rightarrow \phi R_n = 406 k / in = 406 k (24) = 9744k$$

$$\text{Tension yield: } \phi R_n = \phi F_y t (L) = .9 (36ksi) (24) (24) = 1555k > N_u$$

$$\text{Shear yield: } \phi R_n = \phi .6 F_y t L = 1.0 (.6) (36ksi) (24) (24) = 1244k > V_{uc}$$

$$\text{Tension rupture: } \phi R_n = \phi F_u A_e = .75 (58ksi) (24 - (3)(14/16)) (1.0) (24) = 442k > T_u$$

$$\text{Shear rupture: } \phi R_n = \phi 0.6 F_u A_n = .75 (6) (58ksi) (24 - 7(14/16)) (1.0) (24) = 265k > V_u$$

Req'd Load @ Single R.

$$R_u = 244k, \theta = \tan^{-1} \left(\frac{106k}{220k} \right) = 25.7^\circ$$

$$D = \frac{R_u}{1.2 + (4)(1 + 5 \sin(\theta))^{1.5}} = \frac{244k}{1.2 + (4)(1 + 5 \sin(25.7^\circ))^{1.5}} = 2.6 \rightarrow \text{use } 1/4"$$

Foundation Calc

$$T_u = 805k$$

$$V_u = 244k$$

Check Shear Wg

$$\text{Conc. Bk} = \phi (.85) f_c A_g = .9 (.85) (344) 6 \frac{1}{2} (8 \frac{1}{2}) = 127k > V_u$$

$$P_{\text{Bending}} = \phi F_y Z_x = \frac{.9 (50ksi) 2 \frac{1}{2} (8 \frac{1}{2})}{4} = 597k-in < 127k (1 \frac{1}{2} + 6 \frac{1}{2} \frac{1}{2}) = 580k-in$$

Column Weld

$$T_u = 805k$$

13 7/8" Weld

$$1.39 (6) 1.5 ((10 \frac{1}{2})^2 + 14 \frac{1}{2} (2) + (14 \frac{1}{2} - 1 \frac{1}{2} (2))^2) = 912k$$

Check (6) 1-1/2" A490 Anchors

$$T_u = .9 (105ksi) (1 \frac{1}{2})^2 \frac{1}{4} (6) = 1002k > T_u$$

Base R Bending

$$T_u / \gamma_{R1} = 270k$$

$$M_u = \frac{P_u}{4} = \frac{270k (12 \frac{1}{2})}{4} = 845k-in$$

$$\phi M_n = .9 F_y Z_x = .9 (50ksi) (12 \frac{1}{2} (2 \frac{1}{2})^2 \frac{1}{4}) = 845k-in$$

MOMENT FRAME DESIGN

1 MOMENT FRAME ELEVATION
S6.01 1/8" = 1'-0"

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

Building Code: IBC

Drift

07/14/21 17:28:00

Steel Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects

Member Force Output: At Face of Joint

P-Delta: Yes Scale Factor: 1.40

Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
W13	Wind DEFL	Wind_ASCE716_1_X
W14	Wind DEFL	Wind_ASCE716_1_Y
W15	Wind DEFL	Wind_ASCE716_2_X+E
W16	Wind DEFL	Wind_ASCE716_2_X-E
W17	Wind DEFL	Wind_ASCE716_2_Y+E
W18	Wind DEFL	Wind_ASCE716_2_Y-E
W19	Wind DEFL	Wind_ASCE716_3_X+Y
W20	Wind DEFL	Wind_ASCE716_3_X-Y
W21	Wind DEFL	Wind_ASCE716_4_X+Y_CW
W22	Wind DEFL	Wind_ASCE716_4_X+Y_CCW
W23	Wind DEFL	Wind_ASCE716_4_X-Y_CW
W24	Wind DEFL	Wind_ASCE716_4_X-Y_CCW
E5	EQ ELF Drift	EQ_ASCE716_X_+E_Drft
E6	EQ ELF Drift	EQ_ASCE716_X_-E_Drft
E7	EQ ELF Drift	EQ_ASCE716_Y_+E_Drft
E8	EQ ELF Drift	EQ_ASCE716_Y_-E_Drft

RESULTS:

Location (ft): (0.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	0.0011	-0.0446	0.0004	-0.0111	0.0000	0.0001
	Lp	0.0009	-0.0387	0.0002	-0.0093	0.0000	0.0001
	W13	1.9463	-0.0094	0.0902	0.0007	0.0005	0.0000
	W14	0.0076	0.4625	0.0004	0.0586	0.0000	0.0003
	W15	1.8698	0.4101	0.0874	0.0204	0.0005	0.0001
	W16	1.0497	-0.4241	0.0480	-0.0193	0.0003	0.0001
	W17	-0.4113	-0.0773	-0.0198	0.0238	0.0001	0.0001
	W18	0.4228	0.7711	0.0203	0.0641	0.0001	0.0004
	W19	1.4655	0.3399	0.0680	0.0445	0.0004	0.0003
	W20	1.4540	-0.3539	0.0674	-0.0434	0.0004	0.0003
	W21	1.7194	0.8859	0.0808	0.0633	0.0005	0.0004

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

Building Code: IBC

Drift

Page 2/4

07/14/21 17:28:00

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W22	0.4788	-0.3761	0.0212	0.0034	0.0001	0.0000
	W23	1.7108	0.3656	0.0804	-0.0026	0.0005	0.0000
	W24	0.4702	-0.8964	0.0207	-0.0625	0.0001	0.0004
	E5	3.3645	0.3067	0.2710	0.0272	0.0016	0.0002
	E6	2.7599	-0.3076	0.2211	-0.0231	0.0013	0.0001
	E7	-0.5350	0.9104	-0.0421	0.1903	0.0002	0.0011
	E8	0.5844	2.0480	0.0460	0.2790	0.0003	0.0016
Level 6	D	0.0007	-0.0334	0.0002	-0.0101	0.0000	0.0001
	Lp	0.0007	-0.0294	0.0002	-0.0083	0.0000	0.0001
	W13	1.8561	-0.0101	0.1560	-0.0001	0.0010	0.0000
	W14	0.0073	0.4039	0.0006	0.0572	0.0000	0.0004
	W15	1.7824	0.3897	0.1500	0.0333	0.0010	0.0002
	W16	1.0017	-0.4049	0.0841	-0.0334	0.0006	0.0002
	W17	-0.3915	-0.1011	-0.0330	0.0089	0.0002	0.0001
	W18	0.4024	0.7070	0.0340	0.0768	0.0002	0.0005
	W19	1.3975	0.2954	0.1175	0.0428	0.0008	0.0003
	W20	1.3866	-0.3105	0.1165	-0.0429	0.0008	0.0003
	W21	1.6386	0.8226	0.1379	0.0826	0.0009	0.0005
	W22	0.4576	-0.3795	0.0383	-0.0184	0.0003	0.0001
	W23	1.6305	0.3682	0.1372	0.0183	0.0009	0.0001
	W24	0.4494	-0.8339	0.0376	-0.0827	0.0002	0.0005
	E5	3.0935	0.2795	0.3724	0.0348	0.0024	0.0002
	E6	2.5388	-0.2845	0.3054	-0.0330	0.0020	0.0002
	E7	-0.4929	0.7202	-0.0572	0.1498	0.0004	0.0010
	E8	0.5385	1.7690	0.0625	0.2711	0.0004	0.0018
Level 5	D	0.0005	-0.0234	0.0001	-0.0065	0.0000	0.0000
	Lp	0.0006	-0.0211	0.0001	-0.0058	0.0000	0.0000
	W13	1.7001	-0.0100	0.2164	-0.0002	0.0014	0.0000
	W14	0.0067	0.3468	0.0008	0.0654	0.0000	0.0004
	W15	1.6324	0.3564	0.2077	0.0458	0.0014	0.0003
	W16	0.9176	-0.3714	0.1170	-0.0461	0.0008	0.0003
	W17	-0.3585	-0.1101	-0.0455	0.0024	0.0003	0.0000
	W18	0.3685	0.6302	0.0468	0.0958	0.0003	0.0006
	W19	1.2800	0.2526	0.1630	0.0490	0.0011	0.0003
	W20	1.2700	-0.2676	0.1617	-0.0492	0.0011	0.0003
	W21	1.5007	0.7400	0.1908	0.1062	0.0013	0.0007
	W22	0.4194	-0.3611	0.0536	-0.0328	0.0004	0.0002
	W23	1.4932	0.3499	0.1899	0.0326	0.0012	0.0002
	W24	0.4119	-0.7512	0.0526	-0.1064	0.0003	0.0007
	E5	2.7211	0.2447	0.4508	0.0420	0.0030	0.0003
	E6	2.2333	-0.2515	0.3702	-0.0396	0.0024	0.0003
	E7	-0.4357	0.5704	-0.0698	0.1522	0.0005	0.0010
	E8	0.4760	1.4980	0.0763	0.3002	0.0005	0.0020

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

Building Code: IBC

Drift

Page 3/4

07/14/21 17:28:00

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
Level 4	D	0.0004	-0.0168	0.0001	-0.0058	0.0000	0.0000
	Lp	0.0004	-0.0153	0.0001	-0.0053	0.0000	0.0000
	W13	1.4836	-0.0098	0.2731	-0.0004	0.0018	0.0000
	W14	0.0058	0.2813	0.0011	0.0633	0.0000	0.0004
	W15	1.4247	0.3106	0.2614	0.0570	0.0017	0.0004
	W16	0.8007	-0.3254	0.1484	-0.0576	0.0010	0.0004
	W17	-0.3130	-0.1124	-0.0567	-0.0108	0.0004	0.0001
	W18	0.3217	0.5344	0.0583	0.1057	0.0004	0.0007
	W19	1.1171	0.2036	0.2056	0.0472	0.0014	0.0003
	W20	1.1084	-0.2184	0.2041	-0.0478	0.0013	0.0003
	W21	1.3098	0.6337	0.2397	0.1220	0.0016	0.0008
	W22	0.3658	-0.3284	0.0688	-0.0513	0.0005	0.0003
	W23	1.3033	0.3173	0.2385	0.0508	0.0016	0.0003
	W24	0.3592	-0.6448	0.0676	-0.1225	0.0004	0.0008
	E5	2.2703	0.2027	0.5140	0.0470	0.0034	0.0003
	E6	1.8631	-0.2119	0.4233	-0.0450	0.0028	0.0003
	E7	-0.3658	0.4182	-0.0797	0.1269	0.0005	0.0008
	E8	0.3997	1.1978	0.0871	0.2959	0.0006	0.0019
Level 3	D	0.0002	-0.0110	0.0001	-0.0047	0.0000	0.0000
	Lp	0.0003	-0.0100	0.0001	-0.0042	0.0000	0.0000
	W13	1.2105	-0.0094	0.2611	-0.0007	0.0017	0.0000
	W14	0.0048	0.2180	0.0010	0.0607	0.0000	0.0004
	W15	1.1634	0.2536	0.2502	0.0547	0.0016	0.0004
	W16	0.6523	-0.2678	0.1414	-0.0558	0.0009	0.0004
	W17	-0.2563	-0.1016	-0.0546	-0.0106	0.0004	0.0001
	W18	0.2635	0.4287	0.0561	0.1017	0.0004	0.0007
	W19	0.9114	0.1564	0.1966	0.0450	0.0013	0.0003
	W20	0.9043	-0.1706	0.1950	-0.0461	0.0013	0.0003
	W21	1.0701	0.5117	0.2298	0.1173	0.0015	0.0008
	W22	0.2970	-0.2771	0.0651	-0.0498	0.0004	0.0003
	W23	1.0648	0.2665	0.2286	0.0490	0.0015	0.0003
	W24	0.2916	-0.5224	0.0639	-0.1181	0.0004	0.0008
	E5	1.7562	0.1558	0.4439	0.0405	0.0029	0.0003
	E6	1.4398	-0.1669	0.3651	-0.0394	0.0024	0.0003
	E7	-0.2862	0.2913	-0.0701	0.1120	0.0005	0.0007
	E8	0.3126	0.9019	0.0765	0.2608	0.0005	0.0017
Level 2	D	0.0002	-0.0063	0.0001	-0.0055	0.0000	0.0000
	Lp	0.0002	-0.0058	0.0001	-0.0050	0.0000	0.0000
	W13	0.9494	-0.0088	0.2741	-0.0009	0.0015	0.0000
	W14	0.0037	0.1573	0.0011	0.0733	0.0000	0.0004
	W15	0.9131	0.1989	0.2651	0.0598	0.0014	0.0003
	W16	0.5109	-0.2120	0.1460	-0.0611	0.0008	0.0003
	W17	-0.2017	-0.0910	-0.0597	-0.0065	0.0003	0.0000

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

Building Code: IBC

Drift

Page 4/4

07/14/21 17:28:00

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W18	0.2073	0.3270	0.0614	0.1164	0.0003	0.0006
	W19	0.7149	0.1114	0.2064	0.0543	0.0011	0.0003
	W20	0.7092	-0.1245	0.2047	-0.0556	0.0011	0.0003
	W21	0.8404	0.3944	0.2449	0.1322	0.0013	0.0007
	W22	0.2319	-0.2273	0.0647	-0.0507	0.0003	0.0003
	W23	0.8362	0.2174	0.2436	0.0497	0.0013	0.0003
	W24	0.2277	-0.4042	0.0635	-0.1332	0.0003	0.0007
	E5	1.3124	0.1153	0.4010	0.0379	0.0021	0.0002
	E6	1.0747	-0.1275	0.3269	-0.0373	0.0017	0.0002
	E7	-0.2161	0.1793	-0.0670	0.1415	0.0004	0.0008
	E8	0.2361	0.6411	0.0732	0.2839	0.0004	0.0015
Level 1	D	0.0001	-0.0008	0.0001	-0.0008	0.0000	0.0000
	Lp	0.0001	-0.0008	0.0001	-0.0008	0.0000	0.0000
	W13	0.6753	-0.0079	0.6753	-0.0079	0.0028	0.0000
	W14	0.0026	0.0840	0.0026	0.0840	0.0000	0.0004
	W15	0.6481	0.1391	0.6481	0.1391	0.0027	0.0006
	W16	0.3649	-0.1509	0.3649	-0.1509	0.0015	0.0006
	W17	-0.1420	-0.0845	-0.1420	-0.0845	0.0006	0.0004
	W18	0.1460	0.2105	0.1460	0.2105	0.0006	0.0009
	W19	0.5085	0.0571	0.5085	0.0571	0.0021	0.0002
	W20	0.5045	-0.0689	0.5045	-0.0689	0.0021	0.0003
	W21	0.5955	0.2622	0.5955	0.2622	0.0025	0.0011
	W22	0.1672	-0.1765	0.1672	-0.1765	0.0007	0.0007
	W23	0.5926	0.1677	0.5926	0.1677	0.0025	0.0007
	W24	0.1642	-0.2711	0.1642	-0.2711	0.0007	0.0011
	E5	0.9114	0.0773	0.9114	0.0773	0.0038	0.0003
	E6	0.7478	-0.0902	0.7478	-0.0902	0.0032	0.0004
	E7	-0.1491	0.0378	-0.1491	0.0378	0.0006	0.0002
	E8	0.1629	0.3573	0.1629	0.3573	0.0007	0.0015

L/400 - Okay

D*3=.011<.015h - Okay

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Member Code Check

02/12/21 14:03:55

Steel Code: AISC360-16 LRFD

BEAM INFORMATION:

Story Level = Level 1 Frame Number = 3 Beam Number = 39
 Fy (ksi) = 50.00
 Beam Size = W33X130

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	32.00	8.00
Lu for Bending (ft) _____	32.00	8.00
K _____	1.00	1.00
Top Flange Continuously Braced _____	No	
Bottom Flange Continuously Braced _____	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.260 D + 0.500 Lp + 1.000 E6

Segment distance (ft) i - end _____ 24.00
 j - end _____ 32.00

SHEAR CHECK:

Vux (kip) = -68.57	1.00Vnx (kip) = 575.94	Vux/1.00Vnx = 0.119
Vuy (kip) = -0.00	0.90Vny (kip) = 530.96	Vuy/0.90Vny = 0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.400 D

Segment distance (ft) i - end _____ 0.00
 j - end _____ 8.00

AXIAL CHECK:

Pu (kip) = 0.00	0.90Pnx (kip) = 1723.50	Pu/0.90Pnx = 0.000
	0.90Pny (kip) = 1723.50	Pu/0.90Pny = 0.000
	0.90Pn (kip) = 1723.50	Pu/0.90Pn = 0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.260 D + 0.500 Lp - 1.000 E6

Segment distance (ft) i - end _____ 0.00
 j - end _____ 8.00

CALCULATED PARAMETERS:

Pu (kip) = 0.00	0.90Pn (kip) = 1723.50
Mux (kip-ft) = -680.19	0.90Mnx (kip-ft) = 1751.25
Muy (kip-ft) = 0.00	0.90Mny (kip-ft) = 223.13
Cbx = 1.36	

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.388

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RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

02/12/21 14:05:56

Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Level 1

Frame No: 3

Member No: 39

Fy (ksi): 50.00

Size: W33X130

Frame Type: Ordinary Moment Resisting Frame

E1.6b Beam-to-Column - FR Moment Connections

FR Moment Connections shall be designed for required flexure strength of (kip-ft) = 2354.46

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RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

Building Code: IBC

Member Code Check

07/14/21 17:49:09

Steel Code: AISC360-16 LRFD

COLUMN INFORMATION:

Story Level = Level 1 Frame Number = 3 Column Number = 2
 Fy (ksi) = 50.00
 Column Size = W14X605

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	19.75	19.75
Lu for Bending (ft) _____	19.75	19.75
K _____	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.252 D + 0.500 Lp + 1.000 E6

Shear	Top	Vux (kip) _____	33.36
		Vuy (kip) _____	-0.16
Shear	Bot.	Vux (kip) _____	33.36
		Vuy (kip) _____	-0.16

SHEAR CHECK:

Vux (kip) = 33.36	1.00Vnx (kip) = 1630.20	Vux/1.00Vnx = 0.020
Vuy (kip) = -0.16	0.90Vny (kip) = 3908.74	Vuy/0.90Vny = 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.200 D + 1.600 Lp

AXIAL CHECK:

Pu (kip) = 963.74	0.90Pnx (kip) = 7485.76	Pu/0.90Pnx = 0.129
	0.90Pny (kip) = 6566.89	Pu/0.90Pny = 0.147
	0.90Pn (kip) = 6566.89	Pu/0.90Pn = 0.147

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.252 D + 0.500 Lp + 1.000 E6

Axial	Load (kip) _____	619.48
Moment	Top	Mux (kip-ft) _____
		Muy (kip-ft) _____
		3.10
Moment	Bot.	Mux (kip-ft) _____
		Muy (kip-ft) _____
		0.00
		0.00

CALCULATED PARAMETERS:

Pu (kip) = 619.48	0.90Pnx (kip) = 7485.76
	0.90Pny (kip) = 6566.89
Mux (kip-ft) = -612.78	0.90Mnx (kip-ft) = 4950.00
Muy (kip-ft) = 3.10	0.90Mny (kip-ft) = 2445.00
	Mcx (kip-ft) = 4912.01
KL/Rx = 30.43	KL/Ry = 52.12
Cbx = 1.67	

INTERACTION EQUATION:

Pu/0.90*Pn=0.083

Eq H1-3: 0.137 + 0.006 = 0.143

Eq H1-1b Per H1.3: 0.041 + 0.124 + 0.000 = 0.165

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RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (6-29-21)

Building Code: IBC

07/14/21 17:43:21

Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Level 1

Frame No: 3

Member No: 2

Fy (ksi): 50.00

Size: W14X605

Frame Type: Ordinary Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pu (kip) = 619.85 --- Combination: 1.252 D + 0.500 Lp - 3.000 E6

Max Pu/ ϕ Pn = 0.09 OK

Tension: No tension on column

D2.5b Column Splices - Required Strength

*Design strength of column splices must meet or exceed the following forces:**Required tension and compression strength from D1.4a.**Shear in major axis (kip) = 99.34 --- Combination: 1.252 D + 0.500 Lp + 3.000 E6**Shear in minor axis (kip) = 4.51 --- Combination: 1.252 D + 0.500 Lp - 3.000 E7**Moment in major axis (kip-ft) = 1824.98 --- Combination: 1.252 D + 0.500 Lp + 3.000 E6**Moment in minor axis (kip-ft) = 89.06 --- Combination: 1.252 D + 0.500 Lp - 3.000 E7**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.*

D2.5c Required Shear Strength

	Major	Minor
Mpc (kip-ft)	5500.00	2716.67

Column Splice Shear Force Required Mpc / H

Where Mpc is the lesser nominal plastic flexural strength of the column sections.

See code for more information on the Required Shear Strength

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STL BM W33x130

$$\begin{aligned} A &= 38.2 \text{ in}^2 & d &= 33.1" & t_w &= 0.58" & b_f &= 11.5" \\ \phi_f &= 0.855" & k_{col} &= 1.56" & k_{beam} &= 2.18" & h_f &= 1.14" \\ Z_x &= 46.7 \text{ in}^3 \end{aligned}$$

STL COL W14x60

$$\begin{aligned} A &= 17.5 \text{ in}^2 & d &= 20.9" & t_w &= 2.6" & b_f &= 17.4" \\ \phi_f &= 4.16" & k_{col} &= 4.76" & k_{beam} &= 5.81" & h_f &= 2.625" \\ Z_x &= 132.0 \text{ in}^3 \end{aligned}$$

Max Mom @ Hinge Zone

$$M_{pr} = 155 \text{ k-ft} = 18684 \text{ k-in}$$

Max Bolt Dia

$$d_b \leq \frac{b_f}{2} \left(1 - \frac{A_b F_b}{R_t F_u} \right) = \frac{11.5}{2} \left(1 - \frac{1.1(50)}{1.1(65)} \right) = 1.22" \rightarrow \text{use } 7/8"$$

Determine Req'd # of Bolts

$$n = \begin{cases} 1.0 F_u A_b = 1.0 (84 \text{ ksi}) (6.01 \text{ in}^2) \rightarrow 505 \text{ k} \rightarrow \text{Control} \\ 2.4 F_u d_b t_f = 2.4 (1.1) (65 \text{ ksi}) (7/8") (0.855") = 128 \text{ k/bolt} \\ 2.4 F_u p d_b t_f = 2.4 (1.1) (65 \text{ ksi}) (7/8") (1.14") = 225 \text{ k/bolt} \end{cases}$$

$$n = \frac{1.25 M_{pr}}{\phi_n (d + t_f)} = \frac{1.25 (18684 \text{ k-in})}{.9 (50.5 \text{ k/bolt}) (20.9" + 1.14")} = 15.7 \text{ bolts} \rightarrow \text{use } 16 \text{ bolts}$$

Determine Plastic Hinge Zone

$$S_h = S + S \left(\frac{1}{2} - 1 \right) = 4.12" + 3" (8 - 1) = 25.12"$$

Determine Force in Flange

$$F_{fr} = \frac{18684 \text{ k-in}}{20.9" + 1.12"} = 540 \text{ kips}$$

$$n = \frac{540 \text{ k}}{.9 (50.5 \text{ k/bolt})} = 12 < 16 \rightarrow \text{OK}$$

Check R Yields

$$\phi R_n = \phi F_y A_g = .9 (50 \text{ ksi}) (1.71' (11.71')) = 776 \text{ k} > 540 \text{ k}$$

Check R Rupture

$$\phi R_n = \phi F_u A_n = .9 (65 \text{ ksi}) (1.71' (11.71' - 2 (1.5/16'))) = 845 \text{ k} > 540 \text{ k}$$

Check BM Block Shear Rupture

$$R_n = \min (.6 F_u A_{nv} + F_u A_{nt}, .6 F_y A_{gv} + F_u A_{nt}) = 1443 \text{ k}$$

$$A_{gv} = (2) (25') (8.5') = 42.5 \text{ in}^2$$

$$A_{nv} = (2) (8.5') (25' - 7/16 (1.5/16')) = 39.7 \text{ in}^2$$

$$A_{nt} = (2) (8.5') (1' - 7/16 (1.5/16')) = 1.76 \text{ in}^2$$

$$\phi R_n = .9 (1443 \text{ k}) = 1300 \text{ k} > 540 \text{ k}$$

Check R Bending

$$L = 4'$$

$$k = 0.65$$

$$k_y = \frac{.65 (4')}{1.5' / \sqrt{12}} = 6 < 25 \rightarrow \phi R_n = \phi F_y A_g$$

$$\phi R_n = \phi F_y A_g = .9 (50 \text{ ksi}) (1.71' (11.71')) = 776 > 540 \text{ k}$$

Check Shear conn.

$$V_n = \frac{2 M_p}{L_h} + V_{gr} = \frac{2 (18684 \text{ k-in})}{31.5'} + 754/2 = 156 \text{ k}$$

Check column Panel zone Shear

$$R_n = \frac{(2) M_p}{d_c + (2) l_p} = \frac{(2) 781 \text{ k-ft}}{20.9' + (2) 1.71'} = 794 \text{ k-in}$$

$$\phi R = \phi (.6) F_y d_c t_w \left(1 + \frac{3 b_f t_w^2}{d_c d_c t_w} \right) = 1.0 (.6) 50 \text{ ksi} (20.9') (2.6') \left(1 + \frac{3 (17.4') (4.16')^2}{32.1' (20.9') (2.6')} \right) = 2400 \text{ k-in} > R_n$$

1987 NBC “Risk Category IV” “Seismic Design Category B”

This section presents representative structural calculations for Building 2 (7-story healthcare facility in Long Island, New York) Design Case C. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v2A.

TABLE OF CONTENTS

DESIGN CRITERIA.....	1
GRAVITY DESIGN.....	4
BEAM DESIGN.....	6
COLUMN DESIGN.....	11
LATERAL DESIGN.....	18
BRACED FRAME DESIGN.....	22
MOMENT FRAME DESIGN.....	41

DESIGN CRITERIA

DESIGN CRITERIA

VERTICAL LOADS

AREA	DESIGN DEAD LOAD	LIVE LOAD (2)	PARTITION LOAD	CONCENTRATED LOADS
OPERATING ROOMS	95 PSF	60 PSF	20 PSF	1000#
PRIVATE ROOMS	95 PSF	40 PSF	20 PSF	1000#
CORRIDORS ABOVE 1ST FLOOR	95 PSF	80 PSF	20 PSF	1000#
ROOF	100 PSF	40 PSF (1)	-	-

(1) DRIFT AND UNBALANCED SNOW LOAD PER NBC, 1987, CHAPTER 4.

(2) LIVE LOAD REDUCTION NOT PERMITTED EXCEPT AS NOTED IN NBC 1987, SECTION 1115.0.

SNOW:

$P_g = 35$ PSF = GROUND SNOW LOAD

$P_f = C_e I P_g$ = FLAT ROOF SNOW LOAD = 29.4 PSF

$P_s = C_s P_f$ = SLOPED ROOF SNOW LOAD

$I = 1.2$, $C_e = 0.7$

LATERAL FORCES

LATERAL FORCES ARE TRANSMITTED BY DIAPHRAGM ACTION OF ROOF AND FLOORS TO BRACED FRAME/MOMENT FRAME. LOADS ARE THEN TRANSFERRED TO FOUNDATION BY BRACED FRAME/MOMENT FRAME ACTION WHERE ULTIMATE DISPLACEMENT IS RESISTED BY PASSIVE PRESSURE OF EARTH AND/OR SLIDING FRICTION. OVERTURNING IS RESISTED BY DEAD LOAD OF THE STRUCTURE.

WIND:

THE BUILDING MEETS THE CRITERIA PER NBC 1987 SECTION 1112.0.

- EXPOSURE CATEGORY = B
- BASIC WIND SPEED, $V = 90$ MPH
- EQUIVALENT RISK CATEGORY PER TABLE 1.5-1 = IV
- PRESSURE COEFFICIENT (ENCLOSED) = 0.8, -0.5
- WIND IMPORTANCE FACTOR $I_w = 1.07$
- DESIGN WIND BASE SHEAR = 390 KIPS

SEISMIC: (NBC 1987) $V = Z I_k C_{SW}$

$Z = 3/8$

SEISMIC IMPORTANCE FACTOR, $I = 1.50$

EQUIVALENT RISK CATEGORY OF BUILDING PER TABLE 1113.1 = IV

$k = 1.0$ AT BRACED FRAME, $k = 0.67$ AT MOMENT FRAME

$W =$ EFFECTIVE SEISMIC WEIGHT OF BUILDING = 12180 KIPS

ANALYSIS PROCEDURE USED = EQUIVALENT LATERAL FORCE PROCEDURE

$C_S = 0.14$

DESIGN BRACE FRAME BASE SHEAR $V = 962$ KIPS

DESIGN MOMENT FRAME BASE SHEAR $V = 645$ KIPS

FOUNDATION DESIGN CRITERIA

SOIL BEARING PRESSURE: 6000 PSF (ASSUMED)

ACTIVE PRESSURE - RESTRAINED: 50 PCF +14H SEISMIC SURCHARGE (ASSUMED)

ACTIVE PRESSURE - UNRESTRAINED: 35 PCF +6H SEISMIC SURCHARGE (ASSUMED)

PASSIVE RESISTANCE: 200 PCF (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

COEFFICIENT OF FRICTION: .35 (INCLUDES F.O.S. ≥ 1.5) (ASSUMED)

*1/3 INCREASE ALLOWED FOR SEISMIC OR WIND LOADING

CONCRETE

CAST-IN-PLACE CONCRETE

ITEM	DESIGN f_c (PSI) (AT 28 DAYS U.N.O.)
FOUNDATIONS	3000
SLABS ON GRADE AND SLABS ON METAL DECK	4000

REINFORCING STEEL

ASTM A615, GRADE 60 TYPICAL UNLESS NOTED OTHERWISE.

WELDED WIRE REINFORCEMENT SHALL CONFORM TO ASTM A185. LAP ONE FULL MESH ON SIDES AND ENDS BUT NOT LESS THAN 8 INCHES. WELDED WIRE REINFORCING SHALL BE SUPPORTED TO WITHSTAND CONCRETE PLACEMENT. PULLING OF MESH INTO PLACE AFTER PLACEMENT IS NOT ALLOWED.

REINFORCING SPLICE AND DEVELOPMENT LENGTH SCHEDULE, $F_y=40$ KSI (UNLESS NOTED OTHERWISE)

BAR SIZE	MINIMUM LAP SPLICE LENGTHS ("Ls")	MINIMUM DEVELOPMENT LENGTHS ("Ld")
#3	1'-6"	1'-3"
#4	2'-0"	1'-7"
#5	2'-7"	2'-0"
#6	3'-1"	2'-4"
#7	4'-6"	3'-6"
#8	5'-2"	3'-11"
#9	5'-10"	4'-6"
#10	6'-6"	5'-0"
#11	7'-3"	5'-7"

STRUCTURAL STEEL

MATERIAL PROPERTIES

WIDE FLANGE SECTIONS: ASTM A36 ($F_y = 36$ KSI)

OTHER SHAPES AND PLATES: ASTM A36 ($F_y = 36$ KSI)

HOLLOW STRUCTURAL SECTIONS: RECTANGULAR & SQUARE - ASTM A500 GRADE B ($F_y = 46$ KSI)

MACHINE BOLTS (M.B.): ASTM A307

HIGH-STRENGTH BOLTS: A325-ASTM F1852, A490-ASTM F2280

ANCHOR BOLTS (A.B.): ASTM A490, GRADE 36, UNLESS OTHERWISE NOTED

GRAVITY DESIGN

Column "C1"

$$L_o = 80 \text{ psf} \left(.25 + \frac{15}{\sqrt{500(4)}} \right) = 80 \text{ psf} (.59) = 46.8 \text{ psf}$$

Column "C2"

$$L_o = 80 \text{ psf} \left(.25 + \frac{15}{\sqrt{972(4)}} \right) = 80 \text{ psf} (.49) = 40 \text{ psf}$$

Flare Beam "FB1"/"FB4"

$$L_o = 80 \text{ psf} \left(.25 + \frac{15}{\sqrt{1212(4)}} \right) = 80 \text{ psf} (.93) = 75 \text{ psf}$$

Flare Beam "FB3"

$$L_o = 80 \text{ psf} \left(.25 + \frac{15}{\sqrt{1212(4)}} \right) = 80 \text{ psf} (.59) = 47.2 \text{ psf}$$

Flare Beam "FB2"

$$L_o = 80 \text{ psf} \left(.25 + \frac{15}{\sqrt{1212(4)}} \right) = 80 \text{ psf} (.59) = 47.2 \text{ psf}$$

BEAM DESIGN

Project ID:	19051
Project Name:	ATC BLDG 2
Engineer:	JAD
Date:	6/19/2020

Beam # = **FB4**

Shape **W 18x40**

Properties	Geometry	
	A	11.8 <i>in.²</i>
	d	17.9 <i>in.</i>
	b_f	6.018 <i>in.</i>
	t_f	0.524 <i>in.</i>
	t_w	0.316 <i>in.</i>
	W(weight)	40 <i>#/ft.</i>
	r_T	1.54
	d/A_f	5.68

Compact Section Criteria	
b_f/2t_f	5.74
F_y	—
d/t_w	56.6
F_y	52.9 <i>ksi</i>
F_v	20.6 <i>ksi</i>

Elastic Properties					
X-X			Y-Y		
I_x	612	<i>in.⁴</i>	I_y	19.1	<i>in.⁴</i>
S_x	68.4	<i>in.³</i>	S_y	6.34	<i>in.³</i>
r_x	7.21	<i>in.</i>	r_y	1.27	<i>in.</i>
Plastic Modulus					
Z_x	78.4	<i>in.³</i>	Z_y	9.94	<i>in.³</i>

Torsional Constant	
J	0.808 <i>in.⁴</i>

Warping Constant	
C_w	1440 <i>in.⁶</i>

Material Property	
E	29000 <i>ksi</i>
F_y	36 <i>ksi</i>

Note: Input/ verify data in yellow box: **36**

Composite Beam AISC 7th Edition

Note: The following calculation is basic on the "Steel Tips by Steel Committee of California" dated Jan 1987 and Example 1 of Composite Design for building construction in AISC 7ed

Assumption:

Item	Ribs Perpendicular	Ribs Parallel
1. Concrete Area below Top of deck	Neglect	Include
2. Stud Reduction Factor	$\frac{0.85}{(Nr)^{1/2}} \left(\frac{wr}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right)$	$0.6 \left(\frac{wr}{hr} \right) * \left(\frac{Hs}{hr} - 1 \right)$
3. Maximum Stud Spacing	32 in.	Not Specified
4. Deck Welding	16 in.	Not Specified
5 Minimum Width of Rib	2 in.	Depends on Nr

Building info:

Steel			Concrete			Deck		
Beam Length	32	ft.	Weight	145	pcf	Depth hr	3	in.
Beam Spacing	8	ft.	Thickness t	7.5	inches	Conc. thickness tc	4.5	in.
Fy	36	ksi	f'c	3000	psi	Stud Dia.	3/4	in.
			E'c	3122	ksi	Stud Height	4.5	in.
			β_1	0.85		Deck Orientation	Perpendicular	

Uniform Load			Deflection Criteria		
Dead Load	115	psf	Dead Load	L/240	1.6 in.
Live Load	75	psf	Live Load	L/360	1.1 in.
Roof Live Load	0	psf			
Snow Load	0	psf			

Uniform Load W/ Combo	
w(D+L+Lr or S)	190 psf

Max Moment:	kips-in
M(max Dead)	1413.12
M(Dead overwrite)	0 <--- input max Dead if you have uniform load only
M(Dead use)	1413.12
M(max uniform)	2334.72
M(max overwrite)	0 <--- input max Uniform if you have uniform load only
M(max use)	2334.72
Max Shear	kips
V(max Dead)	14.72
V(max uniform)	24.32
V(max overwrite)	0 <--- input max Uniform if you have uniform load only
V(max use)	24.32

Solution Summary:

W 18x40

<---

Change W shapes

Partial Composite:

50%

ASD Result:

	DCR	
Concrete Stress	0.147	
Steel Stress	0.889	
Dead Load Δ	0.770	1.2315 in.
Live Load Δ	0.228	0.2434 in.
Shoring?	Need Shoreing	<---Check this
# of studs	17	at each side of max moment
Camber=	0.750	in.
Overall	GOOD	

Result illustration:			
Beam	Stud	Camber	
W 18x40	34	0.750	
o----- Δ			
24.32 Reaction			24.32 Reaction

$n=E/E_c$	9
-----------	---

1. Effective Width :

$L/4$	96	in.
s	96	in.
$16t+bf$	126.0	in.

Govern b_{eff} : 96.0 in.

2. Required Section Moduli

$M(D+L+S), S_{tr}$	98.3	in.^3
$M(D \text{ only}), S_s$	59.5	in.^3

3. Composite design section properties

a. Moment of Inertia

A_{c1}	432	
$A_{c//}$	579.054	
$A_c=b(t_c)$	432	in.^2
$A'_c=A_c/n$	48.00	in.^2
$Y_s=d/2$	8.95	in.
$Y_c=d+hr+t_c/2$	23.15	in.

Section:	A	Y	AY	d	Ad^2	I_o
W 18x40	11.8	8.95	105.6	11.40	1533.0	612.0
Concrete	48	23.15	1111	2.80	376.9	81.0
Sum	59.8		1216.8		1909.8	693.0
					$I_{tr}=$	2602.85 in.^4
$Y_b=$	20.35	in.				

b. Section Moduli

			Partial Composite	
$S_{tr}=$	127.92	in.^3	$S_{eff}=$	110.5 in.^3
$S_t=$	515.21	in.^3	$I_{eff}=$	2019.7 in.^4

4. Check Stress

a. Concrete				
$f_c=M/n \cdot S_t$	0.1988	ksi <	1.35	OK
b. Steel				
Total Load, $S_{tr}=$	110.5	ksi >	98.26262626	OK
Deadl Load, $S_s=$	68.4	ksi >	59.47	OK
Web shear, $F_v=$	4.300	ksi <	20	OK

5. Check Deflection

$\Delta = \frac{5wl^4}{384 \cdot EI} = \frac{ML^2}{1920 I}$				
$\Delta_D=$	1.231477	in. <	1.6	OK
$\Delta_L=$	0.243358	in. <	1.1	OK

6. Check if shore is required

$S_{tr} \max=$	107.95	in.^3 >	127.92	Need Shoring
----------------	--------	---------	--------	--------------

7. Shear Connection, Stud

a. Max Stud dia	1.31	in. >	0.75	OK
b. Horizontal shear				
$V_h=0.85 \cdot f'_c \cdot A_c/2$	550.8	kips	Concrete	
$V_h=A_s \cdot F_y/2$	212.4	kips	Steel	<----- Govern

Reduction Factor

$$\frac{0.85}{(N_r)^{1/2}} \left(\frac{w_r}{h_r} \right) \cdot \left(\frac{H_s}{h_r} - 1 \right) \quad \text{or} \quad 0.6 \left(\frac{w_r}{h_r} \right) \cdot \left(\frac{H_s}{h_r} - 1 \right)$$

$R_{fL}=$	0.567
$R_{f//}=$	1.000
$R_f \text{ use}=$	0.567
$q \text{ per stud}=$	11.5000
$q \cdot R_f=$	6.517

Stud Shear Connector	F'_c (From AISC V7 Table 1.11.4) (ksi)		
0.5	5.1	5.5	5.9
0.625	8	8.6	9.2
0.75	11.5	12.5	13.3
0.875	15.6	16.8	18

Number of Studs= 17 At each side of where max moment is

COLUMN DESIGN

Member # = C2 at FDN

Shape W 14x283

Properties	Geometry	
	A	83.3 in. ²
	d	16.74 in.
	b _f	16.11 in.
	t _f	2.07 in.
	t _w	1.29 in.
	W(weight)	283 #/ft.
	r _T	4.46
	d/A _f	0.5

Compact Section Criteria	
b _f /2t _f	3.9
F _y	— ksi
d/t _w	13
F _y	— ksi
F _y	— ksi

Elastic Properties					
X-X			Y-Y		
I _x	3840	in. ⁴	I _y	1440	in. ⁴
S _x	459	in. ³	S _y	179	in. ³
r _x	6.79	in.	r _y	4.17	in.
Plastic Modulus					
Z _x	542	in. ³	Z _y	274	in. ³

Torsional Constant	
J	104 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	19.25 ft.
Beam Spacing (w) =	0 ft.
Tributary Area to Col=	977 ft.^2
K=	1
Unbraced Length =	19.75 ft.

Loading	
Dead Load (D)=	115 psf
Live Load(L)=	240 psf
Roof Live Load(Lr)=	40 psf
Snow Load(S)=	29.7 psf

Excentricity on Column	
ex =	2 in.
ey =	1 in.

Deflection Criteria		
Total Load	L/360=	0.64 in.

Max Moment About Major Axis:	
MD(Max Dead)=	1543.6 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	1543.6 kips-in
ML(Max Live)=	469 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	469 kips-in
M(Lr or S)(Max Live)=	78.2 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	78.2 kips-in
Mu=	2090.8 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	64.3 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	64.3 kips-in
ML(Max Live)=	19.5 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	19.5 kips-in
M(Lr or S)(Max Live)=	3.3 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	3.3 kips-in
Mu=	87.1 kips-in

Max Shear	
VD(Max Dead)=	0 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	0 kips
VL(Max Live)=	0 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	0 kips
V(Lr or S)(Max Live)=	0 kips
V(Lr or S)(Live Overwrite)=	0 kips
V(Lr or S)(Max Use)=	0 kips
Vu=	0 kips

Max Axial Load	
PD(Max Dead)=	112.355 kips
PD(Dead Overwrite)=	771.8 kips
PD(Max Use)=	771.8 kips
PL(Max Live)=	234.48 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	234.48 kips
P(Lr or S)(Max Live)=	39.08 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	39.08 kips
P(E or W)=	0 kips
Pu=	1045.36 kips

Solution Summary:	
Try	W 14x283 <---- Change W shapes

Member	Major Axis Beam Bending	18%
	Beam Shear	0%
	Minor Axis Beam Bending	2%
	Axial	70%
	Deflection	0%
	Combined Bending and Axial	93%
Overall Check		93%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 19.25 ft
K= 1
KL= 19.25 ft
KL/r= 55.4

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 17.9 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 12.5$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (Lb)= 19.75 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	3.9	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	13	13.1	Good
$l_b \leq 76 b_f \sqrt{F_y}$	237	204.1	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d / A_f) F_y}$	237	1106.7	Good
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	3.9	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	3.9	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$$F_{bx} = \text{N/A} \text{ ksi}$$

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$$F_{bx} = 25.2 \text{ ksi}$$

Shear Stress Equation: $F_v = 0.40 F_y$

$$F_v = 14.4 \text{ ksi}$$

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$$F_{by} = 27 \text{ ksi}$$

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	4.6	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	0	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0.5	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.93	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.78	Less than 1.0, Okay

Column Name: **C2**

All Equations can be found on Page 3-99 of AISC 8th edition

Column= **W 14x283**

Column Properties	
b	16.11 in
d	16.74 in

Material Properties	
Footing f'c=	3 ksi
Fy(Plate)=	36 ksi

Ultimate Axial Load = **1182 kips**

Required Base Plate Area =	$A_{pl} = \frac{P_u}{0.375 f'_c} =$	1050.7	in^2
----------------------------	-------------------------------------	---------------	-------------

Base Plate Design	
B=	22 in.
N=	22 in.

Actual Bearing Pressure =	$f_p = \frac{P_u}{B * N} =$	2.4	ksi
---------------------------	-----------------------------	------------	------------

Plate Thickness		
$m = \frac{N - 0.95d}{2} =$	3.0485	in
$n = \frac{B - 0.8b}{2} =$	4.556	in
$t = m \sqrt{\frac{f_p}{0.25 F_y}} \leq n \sqrt{\frac{f_p}{0.25 F_y}} =$	2.35	in.

- P = Total column load, kips
- $A = B \times N$ = Area of plate, square inches
- F_b = Allowable bending stress in base plate, ksi
- F_p = Allowable bearing pressure on support, ksi
- f_p = Actual bearing pressure
- f'_c = Compressive strength of concrete, ksi or psi
- t = Thickness of plate, inches

LATERAL DESIGN

Seismic Force Back 1987 → Section 11D

$$V = ZIKCSW \rightarrow 1112.4$$

Where:

$$Z = 2/8 \rightarrow \text{Table 11D.4.1}$$

$$I = 1.50 \rightarrow \text{Table 11D.1}$$

$$K = 1.00 @ BF, 0.67 @ MP \rightarrow \text{Table 11D.4.3}$$

$$CS = 0.14 \rightarrow \text{Section 11D.4.4}$$

$$\begin{aligned} V_{BF} &= 2/8 (1.50) 1.00 (0.14) = .079 W = .079 (12180 u) = 962 u \\ V_{MP} &= 2/8 (1.50) .67 (0.14) = .053 W = .053 (12180 u) = 645 u \end{aligned} \left. \vphantom{\begin{aligned} V_{BF} &= 2/8 (1.50) 1.00 (0.14) = .079 W = .079 (12180 u) = 962 u \\ V_{MP} &= 2/8 (1.50) .67 (0.14) = .053 W = .053 (12180 u) = 645 u \end{aligned}} \right\} \text{Control Both Directions}$$

Force Dist.

$$\text{Roof: } W_6 = 15400 u, h_r = 100'$$

$$6^{th}: W_5 = 17730 u, h_6 = 85'-9"$$

$$5^{th}: W_4 = 17730 u, h_5 = 73'-1"$$

$$4^{th}: W_3 = 17730 u, h_4 = 60'-5"$$

$$3^{rd}: W_2 = 17730 u, h_3 = 47'-9"$$

$$2^{nd}: W_1 = 17730 u, h_2 = 35'-1"$$

$$1^{st}: W_0 = 17730 u, h_1 = 18'-5"$$

$$\Sigma W_i = 121800 u \quad \Sigma h_i = 425'-10"$$

$$\Sigma W_i h_i = 7228300$$

Brace Frame

$$F_r = V \frac{W_r h_r}{\Sigma W_i h_i} = 962 u \left(\frac{15400 (100')}{7228300} \right) = 205 u$$

$$F_6 = V \frac{W_6 h_6}{\Sigma W_i h_i} = 962 u \left(\frac{17730 (85'-9")}{7228300} \right) = 202 u$$

$$F_5 = V \frac{W_5 h_5}{\Sigma W_i h_i} = 962 u \left(\frac{17730 (73'-1")}{7228300} \right) = 172 u$$

$$F_4 = V \frac{W_4 h_4}{\Sigma W_i h_i} = 962 u \left(\frac{17730 (60'-5")}{7228300} \right) = 142 u$$

$$F_3 = V \frac{W_3 h_3}{\Sigma W_i h_i} = 962 u \left(\frac{17730 (47'-9")}{7228300} \right) = 112 u$$

$$F_2 = V \frac{W_2 h_2}{\Sigma W_i h_i} = 962 u \left(\frac{17730 (35'-1")}{7228300} \right) = 82.8 u$$

$$F_1 = V \frac{W_1 h_1}{\Sigma W_i h_i} = 962 u \left(\frac{17730 (18'-5")}{7228300} \right) = 44.4 u$$

$$\Sigma F = 962 u \rightarrow 0 u$$

Moment Frame

$$F_r = V \frac{W_r h_r}{\Sigma W_i h_i} = 645 u \left(\frac{15400 (100')}{7228300} \right) = 136 u$$

$$F_6 = V \frac{W_6 h_6}{\Sigma W_i h_i} = 645 u \left(\frac{17730 (85'-9")}{7228300} \right) = 126 u$$

$$F_5 = V \frac{W_5 h_5}{\Sigma W_i h_i} = 645 u \left(\frac{17730 (73'-1")}{7228300} \right) = 115 u$$

$$F_4 = V \frac{W_4 h_4}{\Sigma W_i h_i} = 645 u \left(\frac{17730 (60'-5")}{7228300} \right) = 95 u$$

$$F_3 = V \frac{W_3 h_3}{\Sigma W_i h_i} = 645 u \left(\frac{17730 (47'-9")}{7228300} \right) = 75 u$$

$$F_2 = V \frac{W_2 h_2}{\Sigma W_i h_i} = 645 u \left(\frac{17730 (35'-1")}{7228300} \right) = 55.2 u$$

$$F_1 = V \frac{W_1 h_1}{\Sigma W_i h_i} = 645 u \left(\frac{17730 (18'-5")}{7228300} \right) = 29.5 u$$

$$\Sigma F = 645 u \rightarrow 0 u$$

Wind Loads

- Exposure B
- Basic wind speed = 90MPH
- $I_w = 1.07$

Effective wind pressures

<u>feet</u>	<u>Pressure</u>
0-20	15 PSF
20-40	17 PSF
40-60	21 PSF
60-100	23 PSF
100-150	27 PSF

Wall pressures

$$C_p = 0.8 \rightarrow P_e$$

$$C_p = -0.5 \rightarrow P_h$$

$$P_x = I_w (P_e + P_h)$$

$$P_1 = 1.07 (15 \text{ PSF} (0.8)) = 12.75 \text{ PSF}$$

$$P_2 = 1.07 (17 \text{ PSF} (0.8)) = 15.6 \text{ PSF}$$

$$P_3 = 1.07 (21 \text{ PSF} (0.8)) = 19.2 \text{ PSF}$$

$$P_4 = 1.07 (23 \text{ PSF} (0.8)) = 21.1 \text{ PSF}$$

$$P_w = 1.07 (27 \text{ PSF} (0.5)) = 15.2 \text{ PSF}$$

Base shear

$$V_w = L (P_1 (h_1) + P_2 (h_2) + P_3 (h_3) + P_4 (h_4) + P_5 (h_5))$$

$$= 124' (12.75 \text{ PSF} (18'-9'') + 15.6 \text{ PSF} (1'-3'') + 19.2 \text{ PSF} (20') + 21.1 \text{ PSF} (40') + 15.2 \text{ PSF} (100 - 18.75''))$$

$$= 360 \text{ k}$$

FOR FRAMING NOTES SEE 52.10
SECOND FLOOR FRAMING PLAN
 1/8" = 1'-0"

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Moment Frame

Braced
Frames

Moment Frame

BRACED FRAME DESIGN

Loads: BLC 3, EQx

SK - 1

Nov 23, 2020 at 2:17 PM

30.5' BF Brace Design (11-18-20).r...

HBrace: **M35**Shape: **STS12x12x0.3125_ASD8th**Material: **A36 Gr.36**Length: **24.952 ft**I Joint: **N10**J Joint: **N17**

Envelope

Code Check: **1.556 (LC 2)**

Report Based On 5 Sections

Dz ————— in

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383.759 at 0 ft

69.223 at 0 ft

Vy ————— k

Vz ————— k

T ————— k-ft

Mz ————— k-ft

My ————— k-ft

26.65 at 0 ft

4.807 at 0 ft

f(y) ————— ksi

f(z) ————— ksi

AISC 9th: ASD Code CheckMax Bending Check **1.556 (LC 2)**Location **0 ft**Equation **H1-1**

Max Shear Check

Location **0 ft**Max Defl Ratio **L/10000****NonCompact**

Fy **36 ksi**
 Fa **17.126 ksi**
 Ft **21.6 ksi**
 Fby **21.6 ksi**
 Fbz **21.6 ksi**
 Fvy **14.4 ksi**
 Fvz **14.4 ksi**
 Cb **1.75**

	y-y	z-z
Cm	.6	.6
Lb	24.952 ft	24.952 ft
KL/r	62.281	63.125
Sway	No	No
L Comp Flange	24.952 ft	

Member # = BF 2 1st Floor Brace

Shape ST S 12x12x0.3125

Properties	Geometry	
	A	14.4 in. ²
	d	12 in.
	b	12 in.
	t	0.3125 in.
	W(weight)	48.86 #/ft.

Compact Section Criteria	
b/t	38.4
h/t	38.4

Material Property	
E	29000 ksi
F _y	46 ksi

Elastic Properties					
X-X			Y-Y		
I _x	324	in. ⁴	I _y	324	in. ⁴
S _x	54	in. ³	S _y	54	in. ³
r _x	4.75	in.	r _y	4.75	in.

Non-Composite Beam/Column Design AISC Eighth Edition

Building info:

Steel Properties	
Column Height =	25 ft
Tributary Area to Col=	0 ft^2
K factor =	1

Loading	
Dead Load (D)=	80 psf
Live Load(L)=	60 psf
Roof Live Load(Lr)=	0 psf
Snow Load(S)=	25 psf

Excentricity on Column	
ex =	0 in
ey =	0 in

Max Moment About Major Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	384 kips
Pu=	384 kips

Solution Summary:

Try **ST S 12x12x0.3125** Change W shapes

Member	Major Axis Beam Bending	0%
	Minor Axis Beam Bending	0%
	Axial	97%
	Combined Bending and Axial	97%
Overall Check		97%

Column Design AISC Eighth Edition

Column Height= 25 ft
 K= 1
 KL= 25 ft
 KL/r= 63.2

Column in Compression	
$C_c = \sqrt{2\pi^2 E / F_y} =$	111
$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2
Column in Tension	
$F_a = 0.6F_y$	section 1.5.1.1
Fa = 27.664 ksi	

Calculate Stresses in Column

Axial Stress: fa=Pu/A	26.7	ksi	Column Okay
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Non-Composite Major Axis Beam Design AISC Eighth Edition

can be found in the AISC Seventh Edition Appendix A

Checks for Typical Major Bending Stress Equation		
Equation	Variable Value	Limit Value
$b/t \leq 238 / \sqrt{F_y}$	38.4	35.1
Major Axis Bending Equation: $F_{bx} = 0.6F_y$		Fbx= 27.6 ksi

Non-Composite Minor Axis Beam Design AISC Eighth Edition

Minor Axis Bending Equation: $F_{by} = 0.6F_y$	Fby = 27.6 ksi
--	----------------

Calculate Stresses in Beam

Major Bending Stress: fbx=Mux/Sx=	0	ksi	Column Okay
Minor Bending Stress: fby=Muy/Sy=	0	ksi	Column Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.97	Less than 1.0, Column Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.97	Less than 1.0, Column Okay

Braced Frame # **HSS 12x12**

Brace Size: **ST S 12x12x0.3125**

Brace Properties	
A	14.4 in ²
t	0.3125 in
B	12 in
F _y	46 ksi
Ultimate Axial Load applied to Brace=	383 kips

Weld Properties	
D	10/16"

Gusset Plate Properties	
tp	1.5 in
F _{yp}	36 ksi
E	29000 ksi
K	0.7
L _p	10.5 in
L	14 in
r	0.433 in
KL/r	17
C _c	126

Brace Yielding	$P_a = 0.6F_y A_{net} =$	371.6	kips	Failed
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Required Weld Length	$L_w = \frac{P_a}{0.928D * 4} =$	11	in
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Gusset Plate Yielding	$P_a = 0.6F_{yp} A_p =$	453.6	kips	Okay
-----------------------	-------------------------	-------	------	------

Gusset Plate Buckling	$P_a = \frac{[1 - \frac{(\frac{KL}{r})^2}{2C_c^2}]F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})}{8C_c} - (\frac{KL}{r})^3 / 8C_c^3} A_p =$	436.3	kips	Okay
-----------------------	---	-------	------	------

Greater than the
tension force of 250
applied

Beam: **M15**

Shape: **W18x60_ASD7th**

Material: **A36 Gr.36**

Length: **30.5 ft**

I Joint: **N2**

J Joint: **N18**

Envelope

Code Check: **0.281 (LC 2)**

Report Based On 97 Sections

Dz _____ **in**

Vz _____ **k**

T _____ **k-ft**

My _____ **k-ft**

f(z) _____ **ksi**

AISC 9th: ASD Code Check

Max Bending Check

Location

Equation

0.281 (LC 2)

15.25 ft

H1-3

Max Shear Check

Location

Equation

0.121 (y) (LC 1)

15.25 ft

H1-3

Max Defl Ratio

Location

Span

L/10000

0 ft

NA

Compact

Fy **36 ksi**
Fa **17.709 ksi**
Ft **21.6 ksi**
Fby **27 ksi**
Fbz **21.6 ksi**
Fvy **14.4 ksi**
Fvz **14.4 ksi**
Cb **1**

Cm **.6**
Lb **8 ft**
KL/r **57.061**
Sway **No**
L Comp Flange **8 ft**

z-z **1**
8 ft
12.862
No

Member # = MF 1st Floor Beam

Shape W 18x76

Properties	Geometry	
	A	22.3 in. ²
	d	18.21 in.
	b _f	11.035 in.
	t _f	0.68 in.
	t _w	0.425 in.
	W(weight)	76 #/ft.
	r _T	2.95
	d/A _f	2.43

Compact Section Criteria	
b _f /2t _f	8.1
F _y	64.2 ksi
d/t _w	42.8
F _y	— ksi
F _y	36 ksi

Elastic Properties				
X-X			Y-Y	
I _x	1330	in. ⁴	I _y	152 in. ⁴
S _x	146	in. ³	S _y	27.6 in. ³
r _x	7.73	in	r _y	2.61 in
Plastic Modulus				
Z _x	163	in. ³	Z _y	42.2 in. ³

Torsional Constant	
J	2.83 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties

Total Member Length =	30.5	ft.
Beam Spacing (w) =	8	ft.
Tributary Area to Col=	0	ft.^2
K=	1	
Unbraced Length =	8	ft.

Loading

Dead Load (D)=	115	psf
Live Load(L)=	80	psf
Roof Live Load(Lr)=	0	psf
Snow Load(S)=	0	psf

Excentricity on Column

ex =	0	in.
ey =	0	in.

Deflection Criteria

Total Load	L/360=	1.02	in.
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Max Moment About Major Axis:

MD(Max Dead)=	1283.745	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	1283.745	kips-in
ML(Max Live)=	893.04	kips-in
ML(Live Overwrite)=		kips-in
ML(Max Use)=	893.04	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	2176.785	kips-in

Max Moment About Minor Axis:

MD(Max Dead)=	0	kips-in
MD(Dead Overwrite)=	0	kips-in
MD(Max Use)=	0	kips-in
ML(Max Live)=	0	kips-in
ML(Live Overwrite)=	0	kips-in
ML(Max Use)=	0	kips-in
M(Lr or S)(Max Live)=	0	kips-in
M(Lr or S)(Live Overwrite)=	0	kips-in
M(Lr or S)(Max Use)=	0	kips-in
Mu=	0	kips-in

Max Shear

VD(Max Dead)=	14.03	kips
VD(Dead Overwrite)=	0	kips
VMD(Max Use)=	14.03	kips
VL(Max Live)=	9.76	kips
VL(Live Overwrite)=	0	kips
VL(Max Use)=	9.76	kips
V(Lr or S)(Max Live)=	4.636	kips
V(Lr or S)(Live Overwrite)=	0	kips
V(Lr or S)(Max Use)=	4.636	kips
Vu=	28.426	kips

Max Axial Load

PD(Max Dead)=	0	kips
PD(Dead Overwrite)=	0	kips
PD(Max Use)=	0	kips
PL(Max Live)=	0	kips
PL(Live Overwrite)=	0	kips
PL(Max Use)=	0	kips
P(Lr or S)(Max Live)=	0	kips
P(Lr or S)(Live Overwrite)=	0	kips
P(Lr or S)(Max Use)=	0	kips
P(E or W)=	17	kips
Pu=	17	kips

Solution Summary:

Try **W 18x76** <---- Change W shapes

Member	Major Axis Beam Bending	63%
	Beam Shear	26%
	Minor Axis Beam Bending	0%
	Axial	11%
	Deflection	77%
	Combined Bending and Axial	74%
Overall Check		77%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 30.5 ft
K= 1
KL= 30.5 ft
KL/r= 140.2

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 7.6 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 0.8$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (L_b)= 8 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	8.1	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	42.8	65.1	Good
$l_b \leq 76 b_f / \sqrt{F_y}$	96	139.8	Good
$l_b \leq \frac{20,000}{(d / A_f) F_y}$	96	228.9	Good
Use Typical Bending Stress Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	8.1	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	8.1	15.8	Good
N/A			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$$F_{bx} = 23.76 \text{ ksi}$$

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$$F_{bx} = \text{N/A} \text{ ksi}$$

Shear Stress Equation: $F_v = 0.40 F_y$

$$F_v = 14.4 \text{ ksi}$$

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$$F_{by} = 27 \text{ ksi}$$

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	14.9	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	3.7	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0.79	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.74	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.66	Less than 1.0, Okay

Column: **M8**

Shape: **W14x398 ASD7th**

Material: **A36 Gr.36**

Length: **19.75 ft**

I Joint: **N17**

J Joint: **N18**

Envelope

Code Check: **0.932 (LC 2)**

Report Based On 97 Sections

D_z _____ in

V_y _____ k
-0.816 at 0 ft
-2.511 at 0 ft

V_z _____ k

T _____ k-ft

M_y _____ k-ft

$f(y)$ _____ ksi
.907 at 19.75 ft
-.907 at 19.75 ft

$f(z)$ _____ ksi

AISC 9th: ASD Code Check

Max Bending Check **0.932 (LC 2)**
Location **19.75 ft**
Equation **H1-1**

Max Shear Check **0.005 (y) (LC 2)**
Location **0 ft**
Max Defl Ratio **L/641**

Compact

F_y **36 ksi**
 F_a **17.897 ksi**
 F_t **21.6 ksi**
 F_{by} **27 ksi**
 F_{bz} **21.6 ksi**
 F_{vy} **14.4 ksi**
 F_{vz} **14.4 ksi**
 C_b **1**

	y-y	z-z
C_m	.6	.6
L_b	19.75 ft	19.75 ft
KL/r	55.032	33.068
Sway	No	No
L Comp Flange	19.75 ft	

Member # = BF 2 Col FDN

Shape W 14x370

Properties	Geometry	
	A	109 in. ²
	d	17.92 in.
	b _f	16.475 in.
	t _f	2.66 in.
	t _w	1.655 in.
	W(weight)	370 #/ft.
	r _T	4.57
	d/A _f	0.41

Compact Section Criteria	
b _f /2t _f	3.1
F _y	— ksi
d/t _w	10.8
F _y	— ksi
F _y	— ksi

Elastic Properties					
X-X			Y-Y		
I _x	5440	in. ⁴	I _y	1990	in. ⁴
S _x	607	in. ³	S _y	241	in. ³
r _x	7.07	in	r _y	4.27	in
Plastic Modulus					
Z _x	736	in. ³	Z _y	370	in. ³

Torsional Constant	
J	222 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	19.75 ft.
Beam Spacing (w) =	0 ft.
Tributary Area to Col=	0 ft.^2
K=	1
Unbraced Length =	19.75 ft.

Loading	
Dead Load (D)=	107 psf
Live Load(L)=	80 psf
Roof Live Load(Lr)=	40 psf
Snow Load(S)=	29.7 psf

Excentricity on Column	
ex =	0 in.
ey =	0 in.

Deflection Criteria		
Total Load	L/360=	0.66 in.

Max Moment About Major Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	50 kips-in
ML(Max Use)=	50 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	50 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Shear	
VD(Max Dead)=	0 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	0 kips
VL(Max Live)=	0 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	0 kips
V(Lr or S)(Max Live)=	0 kips
V(Lr or S)(Live Overwrite)=	0 kips
V(Lr or S)(Max Use)=	0 kips
Vu=	0 kips

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	1893 kips
Pu=	1893 kips

Solution Summary:	
Try	W 14x370 <---- Change W shapes

Member	Major Axis Beam Bending	0%
	Beam Shear	0%
	Minor Axis Beam Bending	0%
	Axial	98%
	Deflection	0%
	Combined Bending and Axial	98%
Overall Check		98%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 19.75 ft
K= 1
KL= 19.75 ft
KL/r= 55.5

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 17.8 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 17.4$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (L_b)= 19.75 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	3.1	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	10.8	-8.7	No Good, Use Alternate EQN
$l_b \leq 76 b_f \sqrt{F_y}$	237	208.7	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d / A_f) F_y}$	237	1358.6	Good
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	3.1	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	3.1	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$F_{bx} = \text{N/A}$ ksi

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$F_{bx} = 25.5$ ksi

Shear Stress Equation: $F_v = 0.40 F_y$

$F_v = 14.4$ ksi

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$F_{by} = 27$ ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	0.1	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	0	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.98	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.81	Less than 1.0, Okay

Foundation Calc

$$T_u = 845k$$

$$V_u = 200k$$

Check Shear Wg

$$\phi V_n = \phi (.85)^2 f'_c A_g = .9 (.85)^2 (3ksi) 6k (10') = 126k > V_u$$

$$P_{Bending} = \frac{100k (6\frac{1}{2}'/2)}{(\frac{10' (6\frac{1}{2}')^2}{4}} = 21.8ksi < 1.33(.6) F_y = 1.33(.6) 36ksi = 28.8ksi$$

Column Weld

$$T_u = 845k$$

Try 1/2" weld

$$.928(8) [(15.2' - 1.3'(2))^2 + 15.7'(2) (15.7' - .83')^2] 1.5 = 716k > T_u \rightarrow \text{OK}$$

Check (6) 1-1/2" A490 Anchors

$$T_n = 95.4k (10) = 763k \rightarrow \text{OK}$$

MOMENT FRAME DESIGN

Loads: BLC 3, EQx
Envelope Only Solution

SK - 1

Nov 23, 2020 at 2:31 PM

Moment Frame Design 10-7.r3d

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

Beam: **M61**Shape: **W36x260_ASD8th**Material: **A36 Gr.36**Length: **30.5 ft**I Joint: **N10**J Joint: **N17**

Envelope

Code Check: **0.552 (LC 2)**

Report Based On 5 Sections

Dz in **Vz** k**T** k-ft **My** k-ft

fa ksi

-0.088 at 0 ft

-0.169 at 0 ft

f(z) ksi**AISC 9th: ASD Code Check**

Max Bending Check

0.552 (LC 2)

Location

30.5 ft

Equation

H2-1

Max Shear Check

0.199 (y) (LC 2)

Location

22.875 ft

Max Defl Ratio

L/7207

Location

7.943 ft

Span

1**Compact**

Fy 36 ksi

Fa 20.254 ksi

Ft 21.6 ksi

Fby 27 ksi

Fbz 23.76 ksi

Fvy 14.4 ksi

Fvz 14.4 ksi

Cb 2.3

Cm .6

Lb 8 ft

KL/r 25.432

Sway No

L Comp Flange 8 ft

z-z .85

8 ft

6.384

No

Member # = MF 1st Floor Beam

Shape W 36x260

Properties	Geometry	
	A	76.5 in. ²
	d	36.26 in.
	b _f	16.55 in.
	t _f	1.44 in.
	t _w	0.84 in.
	W(weight)	260 #/ft.
	r _T	4.34
	d/A _f	1.52

Compact Section Criteria	
b _f /2t _f	5.7
F _y	— ksi
d/t _w	43.2
F _y	— ksi
F _y	35.4 ksi

Elastic Properties					
X-X			Y-Y		
I _x	17300	in. ⁴	I _y	1090	in. ⁴
S _x	953	in. ³	S _y	132	in. ³
r _x	15	in	r _y	3.78	in
Plastic Modulus					
Z _x	1080	in. ³	Z _y	204	in. ³

Torsional Constant	
J	41.5 in. ⁴

Warping Constant	
C _w	— in. ⁶

Material Property	
E	29000 ksi
F _y	36 ksi

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	30.5 ft.
Beam Spacing (w) =	0 ft.
Tributary Area to Col=	0 ft.^2
K=	1
Unbraced Length =	30.5 ft.

Loading	
Dead Load (D)=	0 psf
Live Load(L)=	0 psf
Roof Live Load(Lr)=	0 psf
Snow Load(S)=	0 psf

Excentricity on Column	
ex =	0 in.
ey =	0 in.

Deflection Criteria		
Total Load	L/360=	1.02 in.

Max Moment About Major Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	12384 kips-in
M(Lr or S)(Max Use)=	12384 kips-in
Mu=	12384 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Shear	
VD(Max Dead)=	0 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	0 kips
VL(Max Live)=	0 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	0 kips
V(Lr or S)(Max Live)=	0 kips
V(Lr or S)(Live Overwrite)=	88 kips
V(Lr or S)(Max Use)=	88 kips
Vu=	88 kips

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	25 kips
Pu=	25 kips

Solution Summary:	
Try	W 36x260 <---- Change W shapes

Member	Major Axis Beam Bending	53%
	Beam Shear	20%
	Minor Axis Beam Bending	0%
	Axial	2%
	Deflection	0%
	Combined Bending and Axial	55%
Overall Check		55%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 30.5 ft
K= 1
KL= 30.5 ft
KL/r= 96.8

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 13.4 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 0.3$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (L_b)= 30.5 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	5.7	8.7	Good
$d / t_w \leq 412 (1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	43.2	67.3	Good
$l_b / \sqrt{F_y} \leq 76 b_f / \sqrt{F_y}$	366	209.6	No Good, Use Alternate EQN
$l_b / \sqrt{F_y} \leq \frac{20,000}{(d / A_f) F_y}$	366	365.1	No Good, Use Alternate EQN
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	5.7	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	5.7	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66 F_y$

$F_{bx} = \text{N/A}$ ksi

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$F_{bx} = 24.7$ ksi

Shear Stress Equation: $F_v = 0.40 F_y$

$F_v = 14.4$ ksi

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75 F_y$

$F_{by} = 27$ ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	13	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	2.9	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.55	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.54	Less than 1.0, Okay

Column: **M8**Shape: **W14x455_ASD8th**Material: **A36 Gr.36**Length: **19.75 ft**I Joint: **N9**J Joint: **N10**

Envelope

Code Check: **1.000 (LC 2)**

Report Based On 5 Sections

Dz _____ **in**

625.45 at 0 ft

614.526 at 0 ft

Vz _____ **k****T** _____ **k-ft** **My** _____ **k-ft** **f(z)** _____ **ksi****AISC 9th: ASD Code Check**Max Bending Check **1.000 (LC 2)**Location **0 ft**Equation **H1-2**

Max Shear Check

Location **0 ft**Max Defl Ratio **L/315****Compact**

Fy **36 ksi**

Fa **17.972 ksi**

Ft **21.6 ksi**

Fby **27 ksi**

Fbz **21.6 ksi**

Fvy **14.4 ksi**

Fvz **14.4 ksi**

Cb **2.3**

Cm **.6**

Lb **19.75 ft**

KL/r **54.223**

Sway **No**

L Comp Flange **19.75 ft**

z-z

.271

19.75 ft

32.355

No

Member # = Int. MF Col @ FDN

Shape W 14x455

Properties	Geometry	
	A	134 <i>in.²</i>
	d	19.02 <i>in.</i>
	b _f	16.835 <i>in.</i>
	t _f	3.21 <i>in.</i>
	t _w	2.015 <i>in.</i>
	W(weight)	455 <i>#/ft.</i>
	r _T	4.68
	d/A _f	0.35

Compact Section Criteria	
b _f /2t _f	2.6
F _y	— <i>ksi</i>
d/t _w	9.4
F _y	— <i>ksi</i>
F _y	— <i>ksi</i>

Elastic Properties					
X-X			Y-Y		
I _x	7190	<i>in.⁴</i>	I _y	2560	<i>in.⁴</i>
S _x	756	<i>in.³</i>	S _y	304	<i>in.³</i>
r _x	7.33	<i>in.</i>	r _y	4.38	<i>in.</i>
Plastic Modulus					
Z _x	936	<i>in.³</i>	Z _y	468	<i>in.³</i>

Torsional Constant	
J	395 <i>in.⁴</i>

Warping Constant	
C _w	— <i>in.⁶</i>

Material Property	
E	29000 <i>ksi</i>
F _y	36 <i>ksi</i>

Non-Composite Beam/Column Design AISC Seventh Edition

Building info:

Steel Properties	
Total Member Length =	19.25 ft.
Beam Spacing (w) =	0 ft.
Tributary Area to Col=	0 ft.^2
K=	1
Unbraced Length =	19.25 ft.

Loading	
Dead Load (D)=	107 psf
Live Load(L)=	80 psf
Roof Live Load(Lr)=	40 psf
Snow Load(S)=	29.7 psf

Excentricity on Column	
ex =	1.83 in.
ey =	1 in.

Deflection Criteria		
Total Load	L/360=	0.64 in.

Max Moment About Major Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	12840 kips-in
ML(Max Use)=	12840 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	12840 kips-in

Max Moment About Minor Axis:	
MD(Max Dead)=	0 kips-in
MD(Dead Overwrite)=	0 kips-in
MD(Max Use)=	0 kips-in
ML(Max Live)=	0 kips-in
ML(Live Overwrite)=	0 kips-in
ML(Max Use)=	0 kips-in
M(Lr or S)(Max Live)=	0 kips-in
M(Lr or S)(Live Overwrite)=	0 kips-in
M(Lr or S)(Max Use)=	0 kips-in
Mu=	0 kips-in

Max Shear	
VD(Max Dead)=	0 kips
VD(Dead Overwrite)=	0 kips
VMD(Max Use)=	0 kips
VL(Max Live)=	0 kips
VL(Live Overwrite)=	0 kips
VL(Max Use)=	0 kips
V(Lr or S)(Max Live)=	0 kips
V(Lr or S)(Live Overwrite)=	0 kips
V(Lr or S)(Max Use)=	0 kips
Vu=	0 kips

Max Axial Load	
PD(Max Dead)=	0 kips
PD(Dead Overwrite)=	0 kips
PD(Max Use)=	0 kips
PL(Max Live)=	0 kips
PL(Live Overwrite)=	0 kips
PL(Max Use)=	0 kips
P(Lr or S)(Max Live)=	0 kips
P(Lr or S)(Live Overwrite)=	0 kips
P(Lr or S)(Max Use)=	0 kips
P(E or W)=	625 kips
Pu=	625 kips

Solution Summary:	
Try	W 14x455 <---- Change W shapes

Member	Major Axis Beam Bending	66%
	Beam Shear	0%
	Minor Axis Beam Bending	0%
	Axial	26%
	Deflection	0%
	Combined Bending and Axial	95%
Overall Check		95%

Column Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Column Height= 19.25 ft
K= 1
KL= 19.25 ft
KL/r= 52.7

$$C_c = \sqrt{2\pi^2 E / F_y} = 126$$

$\text{if } \frac{KL}{r} \leq C_c \text{ then } F_a = \frac{[1 - (\frac{KL}{r})^2 / 2C_c^2] F_y}{\frac{5}{8} + \frac{3(\frac{KL}{r})^2}{8C_c^2} - (\frac{KL}{r})^3 / 8C_c^3}$	Section 1.5.1.3.1
$\text{if } \frac{KL}{r} \geq C_c \text{ then } F_a = \frac{12\pi^2 E}{23(\frac{KL}{r})^2}$	Section 1.5.1.3.2

$$F_a = 18.1 \text{ ksi}$$

Calculate Stresses in Column

Axial Stress: $f_a = P_u / A = 4.7$ ksi

Okay

Non-Composite Major Axis Beam Design AISC Seventh Edition

Note: All Equations in this section can be found in the AISC Seventh Edition Appendix A

Unbraced length (L_b)= 19.25 ft

Checks for Typical Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$b_f / 2t_f \leq 55.2 / \sqrt{F_y}$	2.6	8.7	Good
$d / t_w \leq 412(1 - 2.33 \frac{f_a}{F_y}) / \sqrt{F_y}$	9.4	47.8	Good
$l_b \leq 76b_f / \sqrt{F_y}$	231	213.2	No Good, Use Alternate EQN
$l_b \leq \frac{20,000}{(d/A_f) F_y}$	231	1578.5	Good
Use Alternate Equation			

Checks for Alternate Major Bending Stress Equation			
Equation From Appendix A	Variable Value	Limit Value	
$55.2 / \sqrt{F_y} < b_f / 2t_f$	2.6	8.7	Good
$b_f / 2t_f \leq 95.0 / \sqrt{F_y}$	2.6	15.8	Good
Use Alternate Bending Stress Equation			

Typical Bending Stress Equation: $F_{xb} = 0.66F_y$

$F_{bx} = \text{N/A}$ ksi

Alternate Bending Stress Equation: $F_{bx} = F_y [0.733 - 0.0014 (\frac{b_f}{t_f}) \sqrt{F_y}]$

$F_{bx} = 25.6$ ksi

Shear Stress Equation: $F_v = 0.40F_y$

$F_v = 14.4$ ksi

Non-Composite Minor Axis Beam Design AISC Seventh Edition

Minor Axis Bending Equation: $F_{by} = 0.75F_y$

$F_{by} = 27$ ksi

Calculate Stresses in Beam

Major Bending Stress:	$f_{bx} = M_{ux}/S_x =$	17	ksi	Okay
Shear Stress:	$f_v = V_u/(t_w \cdot d) =$	0	ksi	Okay
Minor Bending Stress:	$f_{by} = M_{uy}/S_y =$	0	ksi	Okay

Calculate Deflections

Camber:	$C = .8 \frac{5w_d L^4}{384EI} =$	0	in.	
Total Deflection:	$\Delta_T = \frac{5w L^4}{384EI} - C =$	0	in.	Okay

Check Combined Stressed

Formula 1.6-1a:	$\frac{f_a}{F_a} + \frac{f_{bx}}{(1 - \frac{f_a}{F_{ex}})F_{bx}} + \frac{f_{by}}{(1 - \frac{f_a}{F_{ey}})F_{by}} =$	0.95	Less than 1.0, Okay
Formula 1.6-1b:	$\frac{f_a}{0.6F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} =$	0.88	Less than 1.0, Okay

MF: W36 BM

Input :

Please input or verify the data in yellow box	
Beam Size=	W 36x260
Column Size=	W 14x398
Seat Angle Size=	L 8x8x1
Seat Angle Length (in)	16
Bolt Dia.(in)=	1 1/4
Angle to column bolt number	6
Angle to BM flange bolt number	6
Bolt fy (ksi)	36

Beam	db	36.26	in.	Sx	953	in. ^{^3}
	tw	0.84	in.	Ix	17300	in. ^{^4}
	bf	16.55	in.	tf	1.44	in.
	Zx	1080	in. ^{^3}	Fyb	36	ksi

Column	dc	18.29	in.	bf	16.59	in.
	tf	2.845	in.			
	tw	1.77	in.			
	k	3.5	in.	Fyc	36	ksi

Angle	tf	3.5	in.	Bolt Type	A490X	
	b	4.625	in.			
	a	4.625	in.			
	Check if a<1.25b	OK		Fya	36	ksi

Stiffener If Required	tst=	0.75	in	Fy=	36	ksi
	dst=	6.295	in.	Weld Size D	3	/16"
	l=	8.645	in. ^{^4}			

Moment(Service)/(kip-ft)	1175	kip-ft
Reaction(Service)/(kips)	95	kips

ASD Result summary		DCR
Beam	Beam Bending	47%
	Beam Web checked?	Yes, Checked
Angle	Angle to beam	99%
	Angle to column	73%
PryOut	Fastner	73%
	Angle bending	OK
Column	Column checked?	Yes, Checked
	Web shear check	89%
Stiffener	Compression Stiffener	OK, Stiffener Not Required
	Tension Stiffener	OK, Stiffener Not Required

<---Additional check required to determine beam web shear capacity. Use spreadsheet 'shearconnection 1980'

<---Additional check required to determine column bending capacity. Use spreadsheet 'HSS Column_ 1980 or W-Shape Column 1980'

<--- Check Calculation below for detail if stiffener is needed

Check items	Code Ref. AISC 8ed	Result
a. Capacity in bending on beam: $f_b = \frac{M}{S} =$ 11.10 ksi $F_b = 0.66F_y =$ 23.76 ksi b. Beam Web connection Note: beam web connection should be calculation prior to Moment frame detail calculation using spreadsheet "ShearConnection1980"	1.5.1.4.1	OK
c. Flange Connection $T = C = \frac{M}{d} =$ 291.64 kips 1. Angle to beam flange: Single shear per bolt: 49.1 kips Total Shear= 294.6 kips 2. Angle to column Tension per bolt: 66.3 kips Total Tension: 397.8 kips		N.G.
d. Compute pry out force Q $T @ \text{Each Bolt} = T / \text{Bolt \#} =$ 48.6 Kips $p =$ 2.67 inches $b' =$ 4.00 inches $a' =$ 5 1/4 inches $a' + b' =$ 9.25 inches $d' =$ 1.31 inches $\delta = 1 - (d' \div p) =$ 0.507813 $M = p \times t f^2 \times F_y / 8 =$ 147 Kips-in $\alpha = \left(\frac{T b'}{M} - 1 \right) / \delta =$ 0.635367 <1 $B_c = T \left[1 + \frac{\delta * \alpha}{(1 + \delta * \alpha)} \left(\frac{b'}{a'} \right) \right]$ 57.64 Kips $\text{Req'd } t_f =$ $t_f = \left[\frac{8 * B_c * a' * b'}{p * F_y * [a' + \delta \alpha (a' + b')]} \right]^{1/2}$ 3.50 in	Trib Width Per Bolt $b - d / 2$ $a + d / 2$ Bolt Hole Size Ratio of net area and the gross area -Eq (1) of 4-89 Allowable bending moment tributary to tee stub flange - Eq (2) of 4-89 Eq (3) of 4-89 Load per bolt including prying action, Kips -Eq(4) of 4-89 Required thickness of tee-stub flange -Eq(5) of 4-89	OK

Check items	Code Ref. AISC 8ed	Result
Column:		
1. Column check Note: Column calculation prior to Moment frame detail calculation using spreadsheet "Column Design" 2. Column web shear $t_w = \frac{32M}{(A_{bc} * F_y)}$ Where $A_{bc} = d_b * d_c$ 663.20 in.^2 MIN. t_w = 1.575 in. COL. t_w = 1.770 in.	1.5.1.2 Commentary	OK
3. Web crippling check, stiffener needed? Horizontal Force at stiffeners= $P_{bf} = \frac{4}{3} * \frac{M \times 12}{d + \frac{1}{2} * (t_A + t_B)}$ 472.837 Kips Where $t_A = t_B$ 3.5000 inches $A_{st} \geq [P_{bf} - F_{yc} t (t_b + 5k)] \div F_{yst} =$ -24.0356 in^2 At Compression Flange $d_c > \frac{4100 * t_w^3 * \sqrt{F_{yc}}}{P_{bf}} =$ 288.50 At Tension Flange $t_f < 0.4 * \sqrt{\frac{P_{bf}}{F_{yc}}} =$ 1.450	Formula 1.15-1 1.15-2 1.15-3	OK, Stiffener Not Required OK, Stiffener Not Required
4. If Stiffener is need, $w(\text{Stiffener Width}) = b/3 - (t/2) =$ 4.632 in, min $w(\text{use}) =$ 4 in $t_s > t_b/2 =$ 1.75 in Area Furnished, $A_{st} =$ 6 in Width To Thickness Ratio= 5.333333 < $95 / \sqrt{F_y} =$ 15.83333 Stiffener Length= 8 in Min Length= 6.3 in Min Weld Length= $l = \frac{[P_{bf} - F_{yc} t (t_b + 5k)]}{0.928D \times 1.65} =$ -1.890 in		Stiffener not thick enough Stiffener Area OK Stiffener Ratio OK Stiffener Length OK Check Weld Here

MF Base conn

$$M_u = 900k-ft$$

$$T_u = 80k$$

total tension on stud

$$T = M_u / d + T_u / 2 = 900k-ft (12) / 19" + 80k / 2 = 850.8k$$

$$V_n / bolt = 15.6k \text{ for } \frac{3}{8}" \phi \times 7' \text{ Heated stud}$$

$$\# \text{ of Bolts Req'd} = T / V_n = 850.8k / 15.6k / bolt = 54.5 \rightarrow \text{use } 56 \text{ bolts or } 2 \text{ rows of } 28 \text{ bolts}$$

2018 IBC Risk Category IV Seismic Design Category B

This section presents representative structural calculations for Building 2 (7-story healthcare facility in Long Island, New York) Design Case D. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v2A.

TABLE OF CONTENTS

DESIGN CRITERIA.....	1
GRAVITY DESIGN.....	11
BEAM DESIGN.....	12
COLUMN DESIGN.....	17
LATERAL DESIGN.....	21
BRACED FRAME DESIGN.....	34
MOMENT FRAME DESIGN.....	47

DESIGN CRITERIA

Project: ATC BLDG 2 RC4 Job Number: 19-051
 Sheet: _____ of _____ Name: JAD
 Originating Office: Tacoma Date: 07/15/21

DESIGN CRITERIA CHECKLIST

CODE: IBC 2018, ASCE 7-16 LOCATION: Long Island, NY

VERTICAL DESIGN CRITERIA

	DEAD	LIVE	PARTITION	CONCENTRATED
ROOF:	100 PSF	40 PSF		
OPERATING ROOMS	95 PSF	60 PSF	15 PSF	1000 #
LIBRARY:	95 PSF	150 PSF	15 PSF	1000 #
PRIVATE ROOMS	95 PSF	100 PSF	15 PSF	
CORRIDORS ABOVE THE 1st FLOOR	95 PSF	80 PSF	15 PSF	

WIND DESIGN CRITERIA

BASIC WIND SPEED (V) = 140 MPH ▼ (Per ASCE 7-16 Sec. 26.5.1, Fig. 26.5-1A; 1B; 1C & 1D, or as required by Bld'g Dept.)
 RISK CATEGORY: III & IV ▼ (Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)
 EXPOSURE CATEGORY: B ▼ (Per ASCE 7-16 Section 26.7.3)
 TOPOGRAPHIC FACTOR (K_z): 1.00 ▼ (Per ASCE 7-16 Section 26.8.2)
 MEAN ROOF HEIGHT: 30 FT ▼ (See ASCE 7-16 Section 26.2 - Definitions)
 ELEVATION: 0 FT ▼ (See ASCE 7-16 Section 26.9)
 ENCLOSURE CLASSIFICATION: Enclosed ▼ (See ASCE 7-16 Section 26.2 & Table 26.13-1)
 ROOF TYPE: Monoslope ▼ (See ASCE 7-16 Figure 27.3-1)
 ROOF SLOPE (α :12): 0.00:12 (Enter vertical rise in 12 horizontal units)
 θ (degrees): 0.00

SEISMIC DESIGN CRITERIA

RISK CATEGORY: IV ▼ (Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)
 SITE CLASS: D ▼ (Per IBC Section 1613.2.2, Assumed as "D" or per Geotech.)
 IMPORTANCE FACTOR (I_E): 1.5 (Per ASCE 7-16 Table 1.5-2)
 STRUCTURAL SYSTEM (R): 3.5 (Per ASCE 7-16 Table 12.2-1)
 OVERSTRENGTH FACTOR (Ω_o): 3.0 (Per ASCE 7-16 Table 12.2-1)
 INFORMATION BELOW FROM "EARTHQUAKE SPECTRAL RESPONSE ACCELERATION MAPS" PER USGS
 LATITUDE: 40.647 $S_s =$ 0.245 $F_a =$ 1.600
 LONGITUDE: -73.564 $S_1 =$ 0.055 $F_v =$ 2.400

DEFLECTION CRITERIA

FLOOR (LIVE): L/ 480 ▼ ROOF (LIVE): L/ 360 ▼
 FLOOR (TOTAL): L/ 360 ▼ ROOF (TOTAL): L/ 240 ▼
 WALLS: L/ 360 ▼ SPECIAL: L/ ▼

SOIL DESIGN CRITERIA

REPORT: NO **SEE SOILS REPORT FOR ACTIVE, PASSIVE PRESSURES AND FRICTION COEFFICIENT**
 BEARING: 6000 PSF
 ACTIVE: 35 PCF
 PASSIVE: 250 PCF
 COEFFICIENT OF FRICTION: 0.35
 PILE TYPE: NONE
 VERTICAL CAPACITY: N/A
 UPLIFT CAPACITY: N/A
 MINIMUM FOOTING DIMENSIONS:
 CONTINUOUS: 1'-4"
 SPREAD: 1'-6"
 FROST DEPTH: 1'-6"
 LATERAL CAPACITY: N/A
 SIZE: N/A

Project: ATC BLDG 2 RC4

Job Number: 19-051

Sheet: _____ of _____

Name: JAD

Originating Office: Tacoma

Date: 07/15/21

MATERIALS

CONCRETE

Footings/Piles:	<u>3000 PSI</u>	Columns:	<u>4000 PSI</u>
Slabs/Walls:	<u>4000 PSI</u>	Beams:	<u>4000 PSI</u>
-	<u>-</u>	-	<u>-</u>

REINFORCING

Steel Grade = 60 $f_y =$ 60 KSI

STRUCTURAL STEEL

W-Flange Beams	<u>ASTM A992</u>	$f_y =$	<u>50 KSI</u>
Shapes & Plates	<u>ASTM A36</u>	$f_y =$	<u>36 KSI</u>
Pipes	<u>ASTM A53, Grade B</u>	$f_y =$	<u>35 KSI</u>
HSS Rect.	<u>ASTM A500, Grade C</u>	$f_y =$	<u>50 KSI</u>
HSS Round	<u>ASTM A500, Grade C</u>	$f_y =$	<u>46 KSI</u>

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

DESIGN CRITERIA - WIND

FIGURE 27.3-8: Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases per ASCE 7-16

FIGURE 27.3-8 Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases

DESIGN CRITERIA - WIND

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, C_p , for Enclosed and Partially Enclosed Buildings - Walls and Roofs per ASCE 7-16

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, C_p , for Enclosed and Partially Enclosed Buildings—Walls and Roofs

DESIGN CRITERIA - WIND

FIGURE 30.3-1: Components and Cladding [$h \leq 60$ ft]: External Pressure Coefficients, (G_Cp), for Enclosed and Partially Enclosed Buildings - Walls

Diagram

Notation

a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

Exception: For buildings with $\theta = 0^\circ$ to 7° and a least horizontal dimension greater than 300 ft (90 m), dimension a shall be limited to a maximum of $0.8h$.

h = Mean roof height, in ft (m), except that eave height shall be used for $\theta \leq 10^\circ$.

θ = Angle of plane of roof from horizontal, in degrees.

External Pressure Coefficient, (G_Cp) - Walls

Notes

1. Vertical scale denotes (G_Cp) to be used with qh .
2. Horizontal scale denotes effective wind area, in ft^2 (m^2).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (G_Cp) for walls shall be reduced by 10% when $\theta \leq 10^\circ$.

FIGURE 30.3-1 Components and Cladding [$h \leq 60$ ft ($h \leq 18.3$ m)]: External Pressure Coefficients, (G_Cp), for Enclosed and Partially Enclosed Buildings—Walls

DESIGN CRITERIA - SEISMIC

ASCE 7-16 SECTION 12.8 - EQUIVALENT LATERAL FORCE PROCEDURE

RISK CATEGORY:	IV	LATITUDE:	40.647
SITE CLASS:	D	LONGITUDE:	-73.564
IMPORTANCE FACTOR (I_E):	1.5	$S_S =$	0.245
STRUCTURAL SYSTEM (R):	3.5	$S_1 =$	0.055
OVERSTRENGTH FACTOR (Ω_o):	3	$F_a =$	1.600
		$F_v =$	2.400

ASCE 7-16 SECTION 11.4 SEISMIC GROUND MOTION VALUES

Section 11.4.4 - Coefficients and Risk-Targeted Maximum Considered Earthquake (MCER) Spectral Response Acceleration Parameters

$$S_{MS} = F_a * S_S = 0.392 \quad S_{M1} = F_v * S_1 = 0.132$$

Section 11.4.5 - Design Spectral Response Acceleration Parameters

$$S_{DS} = 2/3 * S_{MS} = 0.261 \quad S_{D1} = 2/3 * S_{M1} = 0.088$$

ASCE 7-16 SECTION 11.6 - SEISMIC DESIGN CATEGORY - SECTION 12.8.2 - PERIOD DETERMINATION

ASCE 7-16 TABLE 11.6-1			
SEISMIC DESIGN CATEGORY BASED ON S_{DS}			
	RISK CATEGORY:		
	I & II	III	IV
$< 0.167g$	A	A	A
$< 0.33g$	B	B	C
$< 0.50g$	C	C	D
$\geq 0.50g$	D	D	D
	C		

ASCE 7-16 TABLE 11.6-2			
SEISMIC DESIGN CATEGORY BASED ON S_{D1}			
	RISK CATEGORY:		
	I & II	III	IV
$< 0.067g$	A	A	A
$< 0.133g$	B	B	C
$< 0.20g$	C	C	D
$\geq 0.20g$	D	D	D
	C		

Each building and structure shall be assigned to the most severe Seismic Design Category in accordance with Table 11.6-1 or Table 11.6-2, irrespective of the fundamental period of vibration of the structure.

PERIOD DETERMINATION:

$$C_t = 0.028$$

$$h_n = 100 \text{ FT}$$

$$x = 0.8$$

$$T_a = C_t * h_n^x = 1.115$$

ASCE 7-16 SECTION 12.8.1.1 - SEISMIC RESPONSE COEFFICIENT

GENERAL EQUATION: $C_S = S_{DS}/(R/I) = 0.112$ EQ. 12.8-2

MAXIMUM: $C_S = S_{D1}/(T^*(R/I)) = 0.034$ <--CONTROLS EQ. 12.8-3

MINIMUM: $C_S = 0.044 * S_{DS} * I > 0.01 = 0.017$ EQ. 12.8-5

For structures located where $S_1 > 0.6g$
 $C_S = 0.5 * S_1/(R/I) = 0.000$ EQ. 12.8-6

ASCE 7-10 SECTION 12.8.1 - SEISMIC BASE SHEAR

$$V = C_S * W = 0.034 * W$$

W = the total dead load and applicable portion of other loads as indicated in Section 12.7.2

Search Information

Coordinates: 40.647053, -73.5641258
Elevation: 42 ft
Timestamp: 2021-06-29T20:48:41.993Z
Hazard Type: Seismic
Reference Document: ASCE7-16
Risk Category: IV
Site Class: D-default

MCE_R Horizontal Response Spectrum

Design Horizontal Response Spectrum

Basic Parameters

Name	Value	Description
S _S	0.245	MCE _R ground motion (period=0.2s)
S ₁	0.055	MCE _R ground motion (period=1.0s)
S _{MS}	0.392	Site-modified spectral acceleration value
S _M	0.132	Site-modified spectral acceleration value
S _{DS}	0.261	Numeric seismic design value at 0.2s SA
S _{D1}	0.088	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	C	Seismic design category
F _a	1.6	Site amplification factor at 0.2s
F _v	2.4	Site amplification factor at 1.0s
CR _S	0.943	Coefficient of risk (0.2s)
CR	0.939	Coefficient of risk (1.0s)
PGA	0.146	MCE _G peak ground acceleration
F _{PGA}	1.508	Site amplification factor at PGA
PGA _M	0.22	Site modified peak ground acceleration
T _L	6	Long-period transition period (s)
SsRT	0.245	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.26	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.055	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.059	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)

S1D	0.6	Factored deterministic hazard spectral acceleration (2% probability of exceedance in 50 years)
PGAd	0.5	Factored deterministic acceleration value (PGA)

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

Disclaimer

Hazard loads are provided by the U.S. Geological Survey [Seismic Design Web Services](#).

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This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

Search Information

Coordinates: 40.647053, -73.5641258
Elevation: 42 ft
Timestamp: 2021-06-29T16:29:22.950Z
Hazard Type: Wind

ASCE 7-16

MRI 10-Year	75 mph
MRI 25-Year	84 mph
MRI 50-Year	93 mph
MRI 100-Year	99 mph
Risk Category I	110 mph
Risk Category II	122 mph
Risk Category III	▲ 131 mph

If the structure under consideration is a healthcare facility and you are also within 1 mile of the coastal mean high water line, you are in a wind-borne debris region. If other occupancy, use the Risk Category II basic wind speed contours to determine if you are in a wind-borne debris region.

Risk Category IV ▲ 137 mph

You are in a wind-borne debris region if you are also within 1 mile of the coastal mean high water line.

ASCE 7-10

MRI 10-Year	77 mph
MRI 25-Year	87 mph
MRI 50-Year	94 mph
MRI 100-Year	102 mph
Risk Category I	114 mph
Risk Category II	125 mph
Risk Category III-IV	▲ 135 mph

If the structure under consideration is a healthcare facility and you are also within 1 mile of the coastal mean high water line, you are in a wind-borne debris region. If other occupancy, use the Risk Category II basic wind speed contours to determine if you are in a wind-borne debris region.

ASCE 7-05

ASCE 7-05 Wind Speed ▲ 115 mph

You are in a wind-borne debris region if you are also within 1 mile of the coastal mean high water line.

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

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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GRAVITY DESIGN

BEAM DESIGN

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (10-20-20)

Building Code: IBC

Floor Map

10/20/20 10:06:28

Steel Code: AISC 360-16 LRFD

Floor Type: Floor

Beam Numbers

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Beam Design

02/02/21 12:25:21

Steel Code: AISC 360-16 LRFD

Floor Type: Floor**Beam Number = 98****SPAN INFORMATION (ft): I-End (47.25,-123.50) J-End (47.25,-91.50)**Beam Size (User Selected) = W21X50 Fy = 50.0 ksi

Total Beam Length (ft) = 32.00

COMPOSITE PROPERTIES (Not Shored):

			Left		Right
Deck Label			Deck		Deck
Concrete thickness (in)			4.50		4.50
Unit weight concrete (pcf)			150.00		150.00
f'c (ksi)			5.00		5.00
Decking Orientation			perpendicular		perpendicular
Decking type			VERCO W3 Formlok		VERCO W3 Formlok
beff (in)	=	91.50	Y bar(in)	=	23.01
Mnf (kip-ft)	=	1038.49	Mn (kip-ft)	=	846.60
C (kips)	=	372.16	PNA (in)	=	20.19
Ieff (in4)	=	3120.64	Itr (in4)	=	3986.68
Stud length (in)	=	4.50	Stud diam (in)	=	0.75
Rg Stud Red. Factor	Rg[1] = 1.00		Rg[2] = 0.85		
Rp Stud Red. Factor	Rp[1] = 0.60		Rp[2] = 0.60		
Stud Capacity (kips)	Qn[1] = 17.2		Qn[2] = 14.6		
# of studs:	Max = 64		Partial = 48		Actual = 48
Number of Stud Rows = 2			Percent of Full Composite Action = 50.63		

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.724	0.610	0.610	---	NonR	0.114	0.152
	32.000	0.724	0.610	0.610			0.114	0.152

SHEAR (Ultimate): Max V_u (1.2DL+1.6LL) = 32.45 kips 1.00V_n = 237.12 kips**MOMENTS (Ultimate):**

Span	Cond	LoadCombo	Mu	@	Lb	Cb	Phi	Phi*Mn
			kip-ft	ft	ft			kip-ft
Center	PreCmp+	1.2DL+1.6LL	124.9	16.0	0.0	1.00	0.90	412.50
	Init DL	1.4DL	109.3	16.0	---	---		
	Max +	1.2DL+1.6LL	259.6	16.0	---	---	0.90	761.94
Controlling		1.2DL+1.6LL	259.6	16.0	---	---	0.90	761.94

REACTIONS (kips):

	Left	Right
Initial reaction	12.20	12.20
DL reaction	11.59	11.59
Max +LL reaction	11.59	11.59
Max +total reaction (factored)	32.45	32.45

DEFLECTIONS:

			Ratio
Initial load (in)	at 16.00 ft =	-0.504	L/D = 761
Live load (in)	at 16.00 ft =	-0.189	L/D = 2033 > 360 ^{14 of 58} 0.18
Post Comp load (in)	at 16.00 ft =	-0.219	L/D = 1756 > 240 0.14

RAM Structural System

Gravity Beam Design

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Page 2/2

02/02/21 12:25:21

Steel Code: AISC 360-16 LRFD

Net Total load (in)	at 16.00 ft	=	-0.723	L/D =	531	>	240	0.45
---------------------	-------------	---	--------	-------	-----	---	-----	------

Project ID:
Project # :
Bay ID : 1-Floor: B-C , 5-4
Model : ATC BLDG 2 ASCE RC 3 (10-20-20)

VIBRATION ANALYSIS:

Equipment: User Defined
Evaluation Criterion: Sensitive Equipment
Reference: Murray, T.M., Allen, D.E. and Ungar, E.E.,
"Floor Vibrations Due to Human Activity",
AISC Design Guide #11, 1997

Vibrational Velocity: V= 6000. μ -in/sec (User Defined)
Beams are not continuous at girders
Girders are not continuous at columns

EVALUATION:

Walking Speed (Steps/min)	V (μ -in/sec)	Evaluation V(limit) (μ -in/sec)	Criterion
100	26932	> 6000	Not Satisfied
75	5985	<= 6000	Satisfied
50	1609	<= 6000	Satisfied

PARAMETER SUMMARY:

Parameter	Beam	Left Girder	Right Girder	
Member	W21X50	W24X68	W24X68	
Uniform Load, plf	751.5	1644.9	2646.8	
Effective I, in ⁴	4150.8	6410.2	7206.5	
Frequency, Hz	9.21	8.52	7.12	5.63

FRAMING: (User Modified Bay)

Girder Span = 30.50 ft
Beam Spans:
Left = 0.00 ft
Center = 32.00 ft
Right = 20.33 ft

Girders/Walls:
Left -W24X68
Right -W24X68

Beam -W21X50
4 spaces at 91.50 in

Concrete: dc = 7.50 in
f'c = 5.00 ksi
wt. = 150.0 pcf
Deck Height = 3.00 in

Loading: Dead = 4.00 psf
Live = 11.00 psf
Collateral = 0.00 psf

Warning: The bay configuration and/or properties analyzed here have been modified by the user and differ from the actual configuration and/or properties of the bay in the RAM Structural System.

COLUMN DESIGN

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Column Design

02/12/21 11:17:28

Steel Code: AISC360-16 LRFD

Story level Level 2, Column Line B-4, Column # 7

Fy (ksi) = 50.00

Column Size

= W14X176

Orientation (deg.) = 0.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	15.67	15.67
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	10.10	10.35
Bottom _____	10.10	10.35

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	545.17	488.28	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 1435.44	0.90Pnx (kip) = 2189.60	Pu/0.90Pnx = 0.656
	0.90Pny (kip) = 1986.72	Pu/0.90Pny = 0.723
	0.90Pn (kip) = 1986.72	Pu/0.90Pn = 0.723

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 3:

	Dead	Live	Roof
Axial (kip) _____	545.17	488.28	0.00
Moments Top Mx (kip-ft) _____	-1.64	-1.64	0.00
My (kip-ft) _____	0.21	0.21	0.00
Bot Mx (kip-ft) _____	-2.05	-16.72	0.00
My (kip-ft) _____	0.27	5.71	0.00

Reverse curvature about X-Axis

Reverse curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 1435.44	0.90*Pn (kip) = 1986.72
Mux (kip-ft) = -29.21	0.90*Mnx (kip-ft) = 1200.00
Muy (kip-ft) = 9.46	0.90*Mny (kip-ft) = 611.25
Rm = 1.00	
Cbx = 1.86	
Cmx = 0.54	Cmy = 0.57
Pex (kip) = 17322.52	Pey (kip) = 6783.30
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.723

Eq H1-1a: $0.723 + 8/9(0.024 + 0.015) = 0.758$ This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Column Design

Page 2/2

02/12/21 11:17:28

Steel Code: AISC360-16 LRFD

Story level Level 1, Column Line B-4, Column # 7

Fy (ksi) = 50.00

Column Size

= W14X176

Orientation (deg.) = 0.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	19.75	19.75
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	10.10	10.35
Bottom _____	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	635.24	578.35	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 1687.64	0.90Pnx (kip) = 2110.42	Pu/0.90Pnx = 0.800
	0.90Pny (kip) = 1808.38	Pu/0.90Pny = 0.933
	0.90Pn (kip) = 1808.38	Pu/0.90Pn = 0.933

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 6:

	Dead	Live	Roof
Axial (kip) _____	635.24	567.03	0.00
Moments Top Mx (kip-ft) _____	-1.63	-1.63	0.00
My (kip-ft) _____	0.21	4.53	0.00
Bot Mx (kip-ft) _____	0.00	0.00	0.00
My (kip-ft) _____	0.00	0.00	0.00

Single curvature about X-Axis

Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 1669.53	0.90*Pn (kip) = 1808.38
Mux (kip-ft) = -4.56	0.90*Mnx (kip-ft) = 1200.00
Muy (kip-ft) = 7.50	0.90*Mny (kip-ft) = 611.25
Rm = 1.00	
Cbx = 1.67	
Cmx = 0.60	Cmy = 0.60
Pex (kip) = 10904.73	Pey (kip) = 4270.17
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.923

Eq H1-1a: $0.923 + 8/9(0.004 + 0.012) = 0.938$ This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

LATERAL

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

07/15/21 16:54:09

LOAD CASE: EQ ELF

Seismic ASCE 7-16 Equivalent Lateral Force

Importance Factor: 1.50 TL: 6.00 s

Site Class D: Stiff Soil, Default

Ss: 0.245 g S1: 0.055 g

Use Specified: SDs: 0.261 g SD1: 0.088 g

Risk Category: IV Seismic Design Category: C

Provisions for: Force

Ground Level: Base

Dir	Eccent	R	Ta Equation				Building Period-T		
X	+ And -	3.5	Std,Ct=0.028,x=0.80				User Defined		
Y	+ And -	3.3	Std,Ct=0.020,x=0.75				User Defined		
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
X	1.118	1.700	1.150	1.150	0.112	0.033	0.017	0.033	1.325
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
Y	0.634	1.700	0.630	0.630	0.120	0.064	0.017	0.064	1.065

Total Building Weight (kips) = 11286.99

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	95.56	0.00	62.23	-55.23
Level 6	1	86.10	81.91	0.00	62.23	-55.23
Level 5	1	73.43	66.33	0.00	62.23	-55.23
Level 4	1	60.76	51.61	0.00	62.23	-55.23
Level 3	1	48.09	37.86	0.00	62.23	-55.23
Level 2	1	35.42	25.25	0.00	62.23	-55.23
Level 1	1	19.75	11.64	0.00	62.23	-55.23

APPLIED STORY FORCES

Type: EQ_ASCE716_X+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	95.56	0.00
Level 6	86.10	81.91	0.00
Level 5	73.43	66.33	0.00
Level 4	60.76	51.61	0.00
Level 3	48.09	37.86	0.00

RAM Structural System

Bentley DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

Loads and Applied Forces

Page 2/5

07/15/21 16:54:09

Level 2	35.42	25.25	0.00
Level 1	19.75	11.64	0.00
		<hr/> 370.16	<hr/> 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	100.35	95.56	0.00	62.23	-67.82
Level 6	1	86.10	81.91	0.00	62.23	-67.82
Level 5	1	73.43	66.33	0.00	62.23	-67.82
Level 4	1	60.76	51.61	0.00	62.23	-67.82
Level 3	1	48.09	37.86	0.00	62.23	-67.82
Level 2	1	35.42	25.25	0.00	62.23	-67.82
Level 1	1	19.75	11.64	0.00	62.23	-67.82

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	95.56	0.00
Level 6	86.10	81.91	0.00
Level 5	73.43	66.33	0.00
Level 4	60.76	51.61	0.00
Level 3	48.09	37.86	0.00
Level 2	35.42	25.25	0.00
Level 1	19.75	11.64	0.00
		<hr/> 370.16	<hr/> 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	100.35	0.00	170.32	68.57	-61.53
Level 6	1	86.10	0.00	151.92	68.57	-61.53
Level 5	1	73.43	0.00	128.23	68.57	-61.53
Level 4	1	60.76	0.00	104.81	68.57	-61.53
Level 3	1	48.09	0.00	81.70	68.57	-61.53
Level 2	1	35.42	0.00	58.99	68.57	-61.53
Level 1	1	19.75	0.00	31.67	68.57	-61.53

RAM Structural System

DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

Loads and Applied Forces

Page 3/5

07/15/21 16:54:09

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	170.32
Level 6	86.10	0.00	151.92
Level 5	73.43	0.00	128.23
Level 4	60.76	0.00	104.81
Level 3	48.09	0.00	81.70
Level 2	35.42	0.00	58.99
Level 1	19.75	0.00	31.67
		0.00	727.66

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	170.32	55.89	-61.53
Level 6	1	86.10	0.00	151.92	55.89	-61.53
Level 5	1	73.43	0.00	128.23	55.89	-61.53
Level 4	1	60.76	0.00	104.81	55.89	-61.53
Level 3	1	48.09	0.00	81.70	55.89	-61.53
Level 2	1	35.42	0.00	58.99	55.89	-61.53
Level 1	1	19.75	0.00	31.67	55.89	-61.53

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	170.32
Level 6	86.10	0.00	151.92
Level 5	73.43	0.00	128.23
Level 4	60.76	0.00	104.81
Level 3	48.09	0.00	81.70
Level 2	35.42	0.00	58.99
Level 1	19.75	0.00	31.67
		0.00	727.66

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RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

Loads and Applied Forces

Page 4/5

07/15/21 16:54:09

LOAD CASE: Wind

ASCE 7-16

Exposure: B

Basic Wind Speed (mph): 137.0

Apply Directionality Factor, $K_d = 0.85$ Use Topography Factor, $K_{zt} = 1.00$ Ground Elevation Factor, $K_e = 1.00$

Use Natural Frequency for X-Direction (Hz) = 0.55

Use Natural Frequency for Y-Direction (Hz) = 0.75

Gust Factor for Flexible Structures, G: Use Calculated G for X-Dir.

Gust Factor for Flexible Structures, G: Use Calculated G for Y-Dir.

Damping Ratio for Flexible Structures = 0.05

Mean Roof Height (ft): Top Story Height = 100.35

Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 0.550

Structure is Flexible

Y-Direction: Natural Frequency = 0.750

Structure is Flexible

CpWindward = 0.80 qLeeward (qh) = 40.40 psf

GCpn (Parapet): Windward = 1.50

Leeward = -1.00

Height ft	Kz	Kzt	qz psf	Gust Factor G		CpLeeward		Pressure (psf)	
				X	Y	X	Y	X	Y
100.35	0.989	1.000	40.401	0.872	0.851	-0.498	-0.500	45.740	44.719
86.10	0.947	1.000	38.671	0.872	0.851	-0.498	-0.500	44.534	43.541
73.43	0.905	1.000	36.952	0.872	0.851	-0.498	-0.500	43.334	42.370
60.76	0.857	1.000	35.005	0.872	0.851	-0.498	-0.500	41.976	41.044
48.09	0.802	1.000	32.743	0.872	0.851	-0.498	-0.500	40.398	39.503
35.42	0.735	1.000	30.004	0.872	0.851	-0.498	-0.500	38.487	37.637
19.75	0.622	1.000	25.392	0.872	0.851	-0.498	-0.500	35.270	34.495
0.00	0.575	1.000	23.472	0.872	0.851	-0.498	-0.500	33.931	33.188

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_X

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	40.65	0.00	62.25	-61.75
Level 6	1	86.10	75.47	0.00	62.25	-61.75
Level 5	1	73.43	69.05	0.00	62.25	-61.75
Level 4	1	60.76	66.86	0.00	62.25	-61.75
Level 3	1	48.09	64.32	0.00	62.25	-61.75
Level 2	1	35.42	68.08	0.00	62.25	-61.75
Level 1	1	19.75	79.10	0.00	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_1_X

Level	Ht	Fx	Fy
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25 of 58

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

Page 5/5

07/15/21 16:54:09

Loads and Applied Forces

	ft	kips	kips
Roof	100.35	40.65	0.00
Level 6	86.10	75.47	0.00
Level 5	73.43	69.05	0.00
Level 4	60.76	66.86	0.00
Level 3	48.09	64.32	0.00
Level 2	35.42	68.08	0.00
Level 1	19.75	79.10	0.00
		<hr/> 463.53	<hr/> 0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	40.06	62.25	-61.75
Level 6	1	86.10	0.00	74.37	62.25	-61.75
Level 5	1	73.43	0.00	68.05	62.25	-61.75
Level 4	1	60.76	0.00	65.90	62.25	-61.75
Level 3	1	48.09	0.00	63.39	62.25	-61.75
Level 2	1	35.42	0.00	67.10	62.25	-61.75
Level 1	1	19.75	0.00	77.98	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_1_Y

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	40.06
Level 6	86.10	0.00	74.37
Level 5	73.43	0.00	68.05
Level 4	60.76	0.00	65.90
Level 3	48.09	0.00	63.39
Level 2	35.42	0.00	67.10
Level 1	19.75	0.00	77.98
		<hr/> 0.00	<hr/> 456.85

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Seismic

Site Class info

Site Class D $\rightarrow S_{DS} = .261$

$V_{ult} = 157 \text{ MPH}$

Seismic

BF

$R = 3/4$
 $C_s = \frac{S_{DS}}{R} (I_e) = \frac{0.261}{3/4} (1.50) = 0.12$

$W = 100 \text{ psf} (15630 \text{ ft}^2) + (95 \text{ psf} + 10 \text{ psf}) (15630 \text{ ft}^2) 6 = 11410 \text{ k}$
 $V_{Fa} = C_s W = 0.12 (11410 \text{ k}) = 1370 \text{ k}$

MF

$R = 3/4$
 $C_s = \frac{S_{DS}}{R} (I_e) = \frac{0.261}{3/4} (1.50) = 0.11$

$W = 100 \text{ psf} (15630 \text{ ft}^2) + (95 \text{ psf} + 10 \text{ psf}) (15630 \text{ ft}^2) 6 = 11410 \text{ k}$
 $V_{Fa} = C_s W = 0.11 (11410 \text{ k}) = 1280 \text{ k}$

Check min & max C_s

$T = C_t h^x = .028 (100)^{.8} = 1.15 \text{ @ MF}$
 $= .02 (100)^{.75} = 0.67 \text{ @ BF} \quad \left. \vphantom{\begin{matrix} T = C_t h^x \\ = .028 (100)^{.8} \\ = .02 (100)^{.75} \end{matrix}} \right\} < T_L = 6 \text{ sec}$

$\text{Max } C_s = \frac{S_{DS}}{T(C_{Te})} = \frac{0.088}{1.15 (3/4)} = .033 \text{ @ MF} \rightarrow V = .033 (W) = 375 \text{ k}$
 $= \frac{0.088}{.67 (3/4)} = 0.07 \text{ @ BF} \rightarrow V = .07 (W) = 800 \text{ k} \rightarrow \text{Seismic controlled @ BF}$

$\text{Min } C_s = .044 S_{DS} I_e = .044 (.305) 1.50 = .02$

Wind Loads

- Exposure B
- Ultimate wind speed = 137 mph
- $I_w = 1.0$

$$q_z = .00256 k_z k_{zt} k_d k_e V^2 = .00256 k_z (85) (1.0) 137^2 = 40.8 \text{ psf } k_z$$

Where:

k_{zt} = varies

$$k_{zt} = 1.0$$

$$k_d = 0.85$$

$$k_e = 1.0$$

q_z as a function of height

$$q_{15} \rightarrow k_z = .57 \rightarrow q_{15} = 23.2 \text{ psf}$$

$$q_{20} \rightarrow k_z = 0.62 \rightarrow q_{20} = 25.4 \text{ psf}$$

$$q_{25} \rightarrow k_z = 0.66 \rightarrow q_{25} = 27.0 \text{ psf}$$

$$q_{30} \rightarrow k_z = 0.70 \rightarrow q_{30} = 28.5 \text{ psf}$$

$$q_{40} \rightarrow k_z = 0.76 \rightarrow q_{40} = 31.0 \text{ psf}$$

$$q_{50} \rightarrow k_z = 0.81 \rightarrow q_{50} = 33.0 \text{ psf}$$

$$q_{60} \rightarrow k_z = 0.85 \rightarrow q_{60} = 34.7 \text{ psf}$$

$$q_{70} \rightarrow k_z = 0.89 \rightarrow q_{70} = 36.3 \text{ psf}$$

$$q_{80} \rightarrow k_z = 0.93 \rightarrow q_{80} = 37.7 \text{ psf}$$

$$q_{90} \rightarrow k_z = 0.96 \rightarrow q_{90} = 38.0 \text{ psf}$$

$$q_{100} \rightarrow k_z = 0.99 \rightarrow q_{100} = q_h = 40.4 \text{ psf}$$

Wind Pressure

$$P_e = q_z G C_p - q_h G C_{pe} = G (q_z C_p - q_h C_{pe})$$

Where:

G = Gust Factor $\rightarrow$ see next page $\rightarrow$ ASCE 7-16 26.11

C_p = 0.8 For windward, -0.5 For Leeward

Gust Factor For MF
Natural Frequency

Left = 124' < 4 times h → use approx Eqn 26.11-2

$$\eta = \frac{22.2}{h^{.8}} = \frac{22.2}{100^{.8}} = .55 < 1 \rightarrow \text{Flexible}$$

Determine Gust Factor

$$I_z = C \left(\frac{z}{z_0} \right)^{.6} = .3 \left(\frac{33}{600} \right)^{.6} = .272$$

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{0.4h}{L_z} \right)^{.63}}} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{124 + 100}{390.6} \right)^{.63}}} = 0.83$$

Where:

$$L_z = 1.6 \left(\frac{z}{33} \right)^{.98} = 320 \left(\frac{60}{33} \right)^{.98} = 390.6'$$

$$g_R = g_V = 3.4$$

$$g_R = \sqrt{2 \ln(3600h)} + \frac{0.577}{\sqrt{2 \ln(3600h)}} = 4.04$$

$$R = \sqrt{\frac{1}{2} R_h R_b R_o (.53 + 4.7 R_z)} = \sqrt{\frac{1}{2} (.083) (.207) (.259) (.53 + 4.7 (.087))} = 0.275$$

Where:

$$V_z = \bar{V} \left(\frac{z}{33} \right)^{.68} \left(\frac{88}{60} \right)^{.125} V$$

$$= 0.45 \left(\frac{60}{33} \right)^{.68} \left(\frac{88}{60} \right)^{.125} = 95.8$$

$$N_1 = \frac{\eta L_z}{V_z} = \frac{.55 (390.6)}{95.8} = 2.24$$

$$R_h = \frac{7.47 N_1}{(1 + 0.3 N_1)^{.53}} = \frac{7.47 (2.24)}{(1 + 0.3 (2.24))^{.53}} = .083$$

$$\eta_h = 4.6 nh / V_z = 4.6 (.55)^{.100} / 95.8 = 2.64 \rightarrow R_h = \frac{1}{2} \eta \left(1 - e^{-\eta} \right) = .207$$

$$\eta_b = 4.6 nh / V_z = 4.6 (.55)^{.124} / 95.8 = 3.27 \rightarrow R_b = \frac{1}{2} \eta \left(1 - e^{-\eta} \right) = .259$$

$$\eta_L = \frac{15.4 n L}{V_z} = \frac{15.4 (.55) 124}{95.8} = 10.96 \rightarrow R_L = \frac{1}{2} \eta \left(1 - e^{-\eta} \right) = .087$$

$$G_F = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_R^2 Q^2 + g_o^2 L^2}}{1 + 1.7 g_o L} \right) = 0.925 \left(\frac{1 + 1.7 (.272) \sqrt{3.4^2 (.80)^2 + 4.04^2 (.275)^2}}{1 + 1.7 (3.4) (.272)} \right) = 0.84$$

Gust Factor For DF

Natural Frequency

Left = 124' < 4 times h → use approx Eqn 26.11-2

$$n = \frac{75}{h} = \frac{75}{100} = 0.75 \rightarrow \text{Flexible}$$

Determine Gust Factor

$$I_z = C \left(\frac{z}{z_0} \right)^{1/6} = .3 \left(\frac{33}{600} \right)^{1/6} = .272$$

$$Q = \sqrt{\frac{1}{1 + 0.65 \left(\frac{0.4h}{L_z} \right)^{6.5}}} = \sqrt{\frac{1}{1 + 0.65 \left(\frac{124 + 100}{390.6} \right)^{6.5}}} = 0.83$$

Where:

$$L_z = 1.6 \left(\frac{z}{33} \right)^{.5} = 320 \left(\frac{60}{33} \right)^{.5} = 390.6'$$

$$g_R = g_V = 3.4$$

$$g_R = \sqrt{2 \ln(3600h)} + \frac{0.577}{\sqrt{2 \ln(3600h)}} = 4.12$$

$$R = \sqrt{\frac{1}{R_h R_b R_o} (.53 + 4.7 R_o)} = \sqrt{\frac{1}{.05 (.069) .239 (.199) (.53 + 4.7 (.065))}} = 0.191$$

Where:

$$V_z = \bar{V} \left(\frac{z}{33} \right)^{.5} \left(\frac{88}{60} \right)^{.5} V$$

$$= 0.45 \left(\frac{60}{33} \right)^{.5} \left(\frac{88}{60} \right)^{.5} 125 = 95.8$$

$$N_1 = \frac{n L_z}{V_z} = \frac{.75 (390.6)}{95.8} = 3.06$$

$$R_h = \frac{7.47 N_1}{(1 + 0.3 N_1)^{.53}} = \frac{7.47 (3.06)}{(1 + 0.3 (3.06))^{.53}} = .069$$

$$N_h = 4.6 n h / V_z = 4.6 (.75) 100 / 95.8 = 3.6 \rightarrow R_h = \frac{1}{N_h} \left(1 - e^{-N_h} \right) = .0239$$

$$N_b = 4.6 n h / V_z = 4.6 (.75) 124 / 95.8 = 4.466 \rightarrow R_b = \frac{1}{N_b} \left(1 - e^{-N_b} \right) = .0199$$

$$N_L = \frac{15.4 n L}{V_z} = \frac{15.4 (.75) 124}{95.8} = 14.95 \rightarrow R_L = \frac{1}{N_L} \left(1 - e^{-N_L} \right) = .065$$

$$G_F = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_R^2 Q^2 + g_o^2 R^2}}{1 + 1.7 g_o I_o} \right) = 0.925 \left(\frac{1 + 1.7 (.272) \sqrt{3.4^2 (.80)^2 + 4.12^2 (.191)^2}}{1 + 1.7 (.34) .272} \right) = 0.85$$

Effective Wind Pressures

<u>feet</u>	<u>Pressure</u>
0-15	32.9 psf
15-20	32.9 psf
20-25	34.4 psf
25-30	35.5 psf
30-40	36.5 psf
40-50	38.2 psf
50-60	39.6 psf
60-70	40.8 psf
70-80	41.8 psf
80-90	43.0 psf
90-100	42.9 psf

Base Shear

$$\begin{aligned}
 V_L &= B(P_{s1}(15') + 5'(P_{20} + P_{25} + P_{30}) + 10'(P_{40} + P_{50} + P_{60} + P_{70} + P_{80} + P_{90} + P_{100})) \\
 &= 124'-0" (32.9 \text{ psf}(15') + 5'(32.9 \text{ psf} + 34.4 \text{ psf} + 35.5 \text{ psf}) + 10'(36.5 \text{ psf} + 38.2 \text{ psf} + 39.6 \text{ psf} + 40.8 \text{ psf} + 41.8 \text{ psf} + 43.0 \text{ psf} + 42.9 \text{ psf})) \\
 &\quad + 10'(43.9 \text{ psf}) \\
 &= 477k < V_{Fa} \rightarrow \text{Wind Controlled @ MF}
 \end{aligned}$$

Plate

P₁

1 = -82.9 PSI

2 = 25.2 PSI

P₂

1 = 130.2 PSI

2 = 25.2 PSI

P₃

1 = -177.4 PSI

2 = 25.2 PSI

P₄

1 = -67.2 PSI

2 = 61.9 PSI

P₅

1 = -104 PSI

2 = 61.9 PSI

EMF = .781

Reduction Factor (Positive)

Effective Width Area	1	2	3	4	5
10	1.0	1.0	1.0	1.0	1.0
20	1.0	.98	.95	.96	.93
50	1.0	.96	.88	.92	.84
100	1.0	.94	.82	.88	.76
500	1.0	.9	.7	.8	.6

C & C Positive Pressure

Effective Width Area	1	2	3	4	5
10	19.6	19.6	19.6	48.4	48.4
20	19.6	19.3	18.6	46.5	45.0
50	19.6	18.8	17.3	44.4	40.6
100	19.6	18.5	16.1	42.5	36.8
500	19.6	17.7	13.8	38.7	29.0

Plate

<u>P₁</u>	<u>P₂</u>	<u>P₃</u>	<u>P₄</u>	<u>P₅</u>
1 = -82.9 psf	1 = 120.2 psf	1 = -177.4 psf	1 = -67.2 psf	1 = -104 psf
2 = 25.2 psf	2 = 25.2 psf	2 = 25.2 psf	2 = 61.9 psf	2 = 61.9 psf

$EAF = .791$

Reduction Factor (negative)

<u>Effective Wind Area</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>
10	1.0	1.0	1.0	1.0	1.0
20	.96	.96	.96	.95	.95
50	.92	.92	.92	.88	.88
100	.88	.88	.88	.82	.82
500	.8	.8	.8	.7	.7

C&C Negative Pressures

<u>Effective Wind Area</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>
10	-64.7	-101.6	-128.6	-52.4	-81.2
20	-62.1	-97.6	-122	-49.9	-77.1
50	-59.6	-93.5	-127	-46.2	-71.5
100	-57.0	-89.4	-121.8	-43.0	-66.5
500	-51.8	-81.2	-110.9	-36.8	-56.8

BRACED FRAME DESIGN

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (01-27-21)

Building Code: IBC

Member Code Check

02/12/21 15:01:05

Steel Code: AISC360-16 LRFD

BRACE INFORMATION:

Story Level = Level 1 Frame Number = 1 Brace Number = 2
 Fy (ksi) = 50.00
 Brace Size = HSS8X8X5/8

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	22.25	22.25
Lu for Bending (ft) _____	22.25	22.25
K _____	1.00	1.00

CONTROLLING BRACE FORCES - SHEAR

Load Combination: 1.400 D

Shear	Top	Vux (kip) _____	-0.00
		Vuy (kip) _____	0.00
Shear	Bot.	Vux (kip) _____	-0.00
		Vuy (kip) _____	0.00

SHEAR CHECK:

Vux (kip) = -0.00	0.90Vnx (kip) = 196.31	Vux/0.90Vnx = 0.000
Vuy (kip) = 0.00	0.90Vny (kip) = 196.31	Vuy/0.90Vny = 0.000

CONTROLLING BRACE FORCES - AXIAL

Load Combination: 1.260 D + 0.500 Lp + 1.000 E3

AXIAL CHECK:

Pu (kip) = 379.22	0.90Pnx (kip) = 410.90	Pu/0.90Pnx = 0.923
	0.90Pny (kip) = 410.90	Pu/0.90Pny = 0.923
	0.90Pn (kip) = 410.90	Pu/0.90Pn = 0.923

CONTROLLING BRACE FORCES - FLEXURE

Load Combination: 1.260 D + 0.500 Lp + 1.000 E3

Axial	Load (kip) _____	379.22
Moment	Top	Mux (kip-ft) _____
		Muy (kip-ft) _____
Moment	Bot.	Mux (kip-ft) _____
		Muy (kip-ft) _____

CALCULATED PARAMETERS:

Pu (kip) = 379.22	0.90Pn (kip) = 410.90
Mux (kip-ft) = 0.00	0.90Mnx (kip-ft) = 167.63
Muy (kip-ft) = 0.00	0.90Mny (kip-ft) = 167.63
KL/Rx = 89.49	KL/Ry = 89.49
Cbx = 1.00	

INTERACTION EQUATION:

Pu/0.90*Pn=0.923

Pr/Pc = 0.923

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RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (01-27-21)

02/12/21 15:01:05

Building Code: IBC

Steel Code: AISC341-16 - LRFD

Brace Parameters

Story: Level 1

Frame No: 1

Member No: 2

Fy (ksi): 50.00

Size: HSS8X8X5/8

Frame Type: Ordinary Concentrically Braced Frame

F1.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf = 10.77 Limit = 15.47 OK

Web h/tw = 10.77 Limit = 15.47 OK

F1.6a Diagonal Brace Connections

Maximum Compression force (kip) = 672.02 Seismic Load Combo: 1.260 D + 0.500 Lp + 2.000 E3

Expected Compression force (kip) = 576.63

Maximum Tension force (kip) = 543.31 Seismic Load Combo: 0.840 D - 2.000 E3

Expected Tension force (kip) = 1148.00

Bolt Slip Compression (kip) = 379.22 Standard Load Combo: 1.260 D + 0.500 Lp + 1.000 E3

Bolt Slip Tension (kip) = 250.51 Standard Load Combo: 0.840 D - 1.000 E3

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20'-0" Braced Frame

1st DM

$$T_u = \min(-\Omega T, P_u F_y A_g) = 458k$$

$$P_u = 0.9 P_n = 0.9 (739k) = 665k$$

$$T_L = \min(-\Omega T, P_u F_y A_g) = 526k$$

$$P_L = 0.9 P_n = 0.9 (588k) = 529k$$

Resultant Forces

$$N = (T_u - P_u) \sin(\theta_1) + (P_L - T_L) \sin(\theta_2) = (458k - 665k) \sin(51.5) + (529k - 526k) \sin(62.3) = 126k$$

$$V = (P_u + T_u) \cos(\theta_1) - (P_L + T_L) \cos(\theta_2) = (665k + 458k) \cos(51.5) - (529k + 526k) \cos(62.3) = 97k$$

$$M_F = \frac{NL}{8} = \frac{126k(20')}{8} = 315k \cdot ft$$

2nd order Effects

β_1

$$\beta_1 = \frac{C_m}{1 - P_r/P_{e1}} \geq 1 = \frac{1.0}{1 - \frac{130k}{3697k}} = 1.04$$

Where:

$$P_r = 130k$$

$$P_{e1} = \frac{\pi^2 EI_y}{(L_{e1})^2} = \frac{\pi^2 (29,000 ksi) (186 in^4)}{(120')^2} = 3697k$$

β_2

$$\beta_2 = \frac{1}{1 - \frac{P_{strong}}{P_{weak}}} \geq 1 = \frac{1}{1 - \frac{3783k}{63545k}} = 1.06$$

Where:

$$P_{strong} = P_m \frac{HL}{\Delta H} = .85 \frac{411k(19'-3'')}{1.27'} = 63545k$$

$$P_{weak} = (1.2(95 kft) + 1.6(30 kft)) 15620 ft^2 = 3783k$$

20'-0" Braced Frame

1" DM

Max Load Case

$$P_u = P_r P_E = 1.04 (97 \text{ k}) = 101 \text{ k}$$

$$M_u = D_1 (1 + \alpha) M_0 + \beta_1 (.5) M_L + \beta_2 M_E = 1.04 (1.06) 322 \text{ k-ft} + 1.04 (.5) 272 \text{ k-ft} + 1.06 (215 \text{ k-ft})$$

$$= 720 \text{ k-ft}$$

$$\frac{P_u}{\phi P_n} + \left(\frac{M_u}{\phi M_n} \right) = \frac{101 \text{ k}}{2 (1120 \text{ k})} + \left(\frac{215 \text{ k-ft}}{720 \text{ k-ft}} \right) = .48 < 1.0$$

Try W16x67

$$\frac{101 \text{ k}}{2 (74 \text{ k})} + \frac{215 \text{ k-ft}}{474 \text{ k-ft}} = 0.73 \rightarrow \text{OK}$$

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (01-27-21)

Building Code: IBC

Member Code Check

02/12/21 15:01:05

Steel Code: AISC360-16 LRFD

COLUMN INFORMATION:

Story Level = Level 1 Frame Number = 1 Column Number = 22
 Fy (ksi) = 50.00
 Column Size = W14X132

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	19.75	19.75
Lu for Bending (ft) _____	19.75	19.75
K _____	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.260 D + 0.500 Lp + 1.000 E3

Shear	Top	Vux (kip) _____	4.52
		Vuy (kip) _____	1.20
Shear	Bot.	Vux (kip) _____	4.52
		Vuy (kip) _____	1.20

SHEAR CHECK:

Vux (kip) = 4.52	1.00Vnx (kip) = 284.44	Vux/1.00Vnx = 0.016
Vuy (kip) = 1.20	0.90Vny (kip) = 817.61	Vuy/0.90Vny = 0.001

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.260 D + 0.500 Lp + 1.000 E4

AXIAL CHECK:

Pu (kip) = 732.45	0.90Pnx (kip) = 1573.30	Pu/0.90Pnx = 0.466
	0.90Pny (kip) = 1305.45	Pu/0.90Pny = 0.561
	0.90Pn (kip) = 1305.45	Pu/0.90Pn = 0.561

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.260 D + 0.500 Lp + 1.000 E4

Axial	Load (kip) _____	732.45
Moment	Top	Mux (kip-ft) _____
		Muy (kip-ft) _____
		8.31
Moment	Bot.	Mux (kip-ft) _____
		Muy (kip-ft) _____
		-17.62

CALCULATED PARAMETERS:

Pu (kip) = 732.45	0.90Pn (kip) = 1305.45
Mux (kip-ft) = 62.46	0.90Mnx (kip-ft) = 877.50
Muy (kip-ft) = -17.62	0.90Mny (kip-ft) = 423.75
KL/Rx = 37.74	KL/Ry = 63.06
Cbx = 2.15	

INTERACTION EQUATION:

Pu/0.90*Pn=0.561

Eq H1-1a: 0.561 + 8/9(0.071 + 0.042) = 0.661

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HSS 8x8x 5/8

$$T_u = 476k$$

$$P_u = 646k$$

Brace to Gussier Conn:

Brace Weld: $\phi R_n = 1.25(6)4(20) = 600k > P_u \rightarrow 17' \text{ min}$

Net section rupture: $\phi R_n = \phi A_n F_u = .75(14.1 \text{ in}^2)(62ksi) = 557k > P_u \rightarrow OK$

where:

$$A_n = 16.4 \text{ in}^2 - (2) 2" (0.581) = 14.08 \text{ in}^2$$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{2"}{20"} = 0.85$$

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{8^2 + (2)8^2}{4(8+8)} = 3" \quad , \quad L = 16'$$

Shear Yield: $\phi R_n = \phi (6) F_y (b) L_w (u) = 1.0 (6) 50ksi (.581) 20 (4) = 1395k > T_u$

Shear Rupture: $\phi R_n = \phi .6 F_u (b) L_w (u) = (0.75) .6 (62ksi) .581 (20) 4 = 1257k > T_u$

Gusset PL

Buckling: $K_y r = \frac{.7(10")}{2" \sqrt{12}} = 12.1 < 25$

$$\phi P_n = \phi F_c A_g = .9 (36 \text{ ksi}) 2" (10) = 648k > P_u \rightarrow OK$$

Gusset @ col

$$N_{uL} = 646k \sin(63) = 575k$$

$$M_{uL} = 575k (14\frac{1}{2}) = 4030 \text{ k-in}$$

$$N_{uB} = \frac{4(4030 \text{ k-in})}{40} = 403k$$

Tension Yield: $\phi R_n = \phi F_y A_g = .9 (36ksi) 2" (40") = 2592k$

Shear Yield: $\phi R_n = \phi .6 F_y t L = 1.0 (.6) 36ksi (2") 40 = 1560k > V_{uL}$

Check Weld: $L = 28'$, $u = 0.6$, $a = .15 \rightarrow C = 2.43 \rightarrow \text{Table 8-10a}$

$$\phi R_n = \phi D L = .75 (2.43) 5 (28")^2 = 720k > P_u \rightarrow \underline{5/16" \text{ Weld on}}$$

HSS 8x8x 5/8

Support @ Beam

$$N_{ub} = 646 \text{ k} \cos(62^\circ) = 293 \text{ k}$$

$$M_{ub} = 293 \text{ k} (7'') = 2053 \text{ k-in}$$

$$N_{ub} = \frac{4(2053 \text{ k-in})}{17''} = 483 \text{ k}$$

Tension Yield: $\phi R_n = \phi F_y t(L) = .9(36 \text{ ksi})(2'')(17'') = 1102 \text{ k} > N_{ub}$

Shear Yield: $\phi R_n = \phi .6 F_y t(L) = 1.0(.6)(36 \text{ ksi})(2'')(17'') = 661 \text{ k} > V_{ub}$

HSS 8x8x 5/8

$$T_u = 500k$$

$$P_u = 582k$$

Brace to Gusset Conn:

Brace weld: $\phi R_n = 1.25(6)(4)(20) = 600k > P_u \rightarrow 17' \text{ min}$

Net section rupture: $\phi R_n = \phi A_n F_u = .75(14.1 \text{ in}^2)(62ksi) = 557k > P_u \rightarrow OK$

where:

$$A_n = 16.4 \text{ in}^2 - (2)(2")(.581) = 14.08 \text{ in}^2$$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{2"}{20"} = .88$$

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{8^2 + (2)(8^2)}{4(8+8)} = 3" \quad , \quad L = 16"$$

Shear Yield: $\phi R_n = \phi (6) F_y (6) L_w (4) = 10 (6)(50ksi)(.581)(20)(4) = 1395k > T_u$

Shear Rupture: $\phi R_n = \phi .6 F_u (6) L_w (4) = (.75) .6 (62ksi) .581 (20)(4) = 1257k > T_u$

Gusset PL

Buckling: $K_y r = \frac{.7(10")}{1.5 \sqrt{A_{T2}}} = 162 < 25$

$$\phi P_n = \phi F_y A_g = .9(36 \text{ ksi})(17.1 \text{ in}^2) = 608k > P_u \rightarrow OK$$

HSS 8x8x 5/8

Gusset @ Beam

$$N_{UG} = 646 \text{ k} \cos(63^\circ) = 293 \text{ k}$$

$$M_{UG} = 293 \text{ k} (7^\circ) = 2053 \text{ k-in}$$

$$N_{UG} = \frac{4(2053 \text{ k-in})}{17^\circ} = 483 \text{ k}$$

Tension Yield: $\phi R_n = \phi F_y t(L) = .9(36 \text{ ksi})(2^\circ)(17^\circ) = 1104 \text{ k} > N_{UG}$

Shear Yield: $\phi R_n = \phi .6 F_y tL = 1.0(.6)(36 \text{ ksi})(2^\circ)(17^\circ) = 661 \text{ k} > V_{UG}$

Bolted Conn. @ Gusset

Use 1" A490 Bolts

$$\phi R_n = \phi F_u A_b = .75(65 \text{ ksi})(1^\circ)^2 \frac{\pi}{4} = 40 \text{ k}$$

$$n = \frac{R_u}{\phi R_n / \text{bolt}} = \frac{582 \text{ k}}{40 \text{ k/bolt}} = 14.6 \rightarrow \text{use } (15) \text{ 1" A490 N Bolts}$$

Check 1-1/2" kerf R 4 (15) Bolts

Bearing @ gusset

$$R_n = (2.4 \alpha_6 F_u n_b) + 1.2 L_c t F_u (n_r) = 2.4(1^\circ) 1 \frac{1}{2} (65 \text{ ksi})(15) + 1.2(1.5^\circ) 1 \frac{1}{2} (65 \text{ ksi}) 3 = 4570 \text{ k}$$

$$\phi R_n = .75(4570 \text{ k}) = 3428 \text{ k} > R_n \rightarrow \text{OK}$$

Tension Yielding: $\phi R_n = \phi F_y A_g = .9(50 \text{ ksi})(1 \frac{1}{2}^\circ)(9^\circ) = 608 \text{ k} > T_n$

Tensile Rupture: $\phi R_n = \phi F_u A_n = .75(65 \text{ ksi}) 1 \frac{1}{2}^\circ ((9^\circ - (3^\circ)(1 \frac{1}{2}^\circ)) = 426 \text{ k} < T_n \rightarrow \text{use } 1-2 \frac{1}{4}^\circ$

$$\alpha = 9.7^\circ, \beta = 18^\circ \quad \left. \begin{array}{l} e_b = 3' \\ e_c = 7' \end{array} \right\} r = \sqrt{(\alpha + e_b)^2 + (\beta + e_c)^2} = 30.9'$$

$$V_{uc} = \frac{P}{r} P_u = \frac{18'}{30.9'} (582k) = 339k$$

$$N_{uc} = \frac{e_c}{r} P_u = \frac{7'}{30.9'} (582k) = 131.8k$$

$$R_u = \sqrt{339k^2 + 132k^2} = 363k$$

$$\# Bolts = 363k / 40k / 80k = 9.1 \text{ bolts} \rightarrow \text{use (10) bolts}$$

Bearing $\rightarrow \phi R_n = (0.6 A_{tn}) = (0.6 A_{tn}) = 76k$
Tension $\rightarrow \phi R_n = 76.7 k \text{ in } 7/8" = 76.7 k (7/8) = 58k$

$\left. \begin{array}{l} \phi R_n = 76.7 k \text{ in } 7/8" > V_{uc} \\ \phi R_n = 58 k \text{ in } 7/8" > N_{uc} \end{array} \right\}$

Tension Yield: $\phi R_n = \phi F_y A_g = .9(36ksi)(7/8)(30) = 729k > N_u$

Shear Yield: $\phi R_n = \phi .6 F_y A_g = 1.0(.6)(36ksi)(7/8)(30) = 486k > V_{uc}$

Tension Rupture: $\phi R_n = \phi F_u A_e = .75(58ksi)(30 - (10)(1/4)) = 632k > T_u$

Shear Rupture: $\phi R_n = \phi 0.6 F_u A_{nv} = .75(.6)(58ksi)(30 - 10(1/4)) = 380k > V_u$

Req'd weld @ Single R.

$$R_u = 363k, \quad \theta = \tan^{-1} \left(\frac{132k}{339k} \right) = 21.3^\circ$$

$$D = \frac{R_u}{1.2(2)(1 + 5 \sin(\theta)^{1.5})} = \frac{363k}{1.2(2)(2)(1 + 5 \sin(21.3^\circ)^{1.5})} = 3.9 \rightarrow \text{use } 1/4"$$

Foundation Cals

$$T_u = 805k$$

$$V_u = 244k$$

Check Shear Wdg

$$\text{Conc. Bldg} = \phi (.85) f_c A_g = .9 (.85) (344) 6 \frac{1}{2} (8 \frac{1}{2}) = 127k > V_{u, req}$$

$$P_{\text{Bending}} = \phi F_y Z_x = \frac{.9 (50ksi) 2 \frac{1}{2} (8 \frac{1}{2})}{4} = 597k < 127k (1 \frac{1}{2} + 6 \frac{1}{2} \frac{1}{2}) = 580k$$

Column Weld

$$T_u = 805k$$

13 7/8" Weld

$$1.39 (6) 1.5 ((10 \frac{1}{2})^2 + 14 \frac{1}{2} (2) + (14 \frac{1}{2} - 1 \frac{1}{2} (2))^2) = 912k$$

Check (6) 1-1/2" A490 Anchors

$$T_u = 0.9 (105ksi) (1 \frac{1}{2})^2 \frac{1}{4} (6) = 1002k > T_u$$

Base R Bending

$$T_u / \gamma_{R1} = 270k$$

$$M_u = \frac{P_u}{4} = \frac{270k (12 \frac{1}{2})}{4} = 845k$$

$$\phi M_n = .9 F_y Z_x = .9 (50ksi) (12 \frac{1}{2} (2 \frac{1}{2})^2 \frac{1}{4}) = 845k$$

MOMENT FRAME DESIGN

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

Building Code: IBC

Drift

07/15/21 17:06:12

Steel Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects
 Member Force Output: At Face of Joint
 P-Delta: No
 Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
W13	Wind DEFL	Wind_ASCE716_1_X
W14	Wind DEFL	Wind_ASCE716_1_Y
W15	Wind DEFL	Wind_ASCE716_2_X+E
W16	Wind DEFL	Wind_ASCE716_2_X-E
W17	Wind DEFL	Wind_ASCE716_2_Y+E
W18	Wind DEFL	Wind_ASCE716_2_Y-E
W19	Wind DEFL	Wind_ASCE716_3_X+Y
W20	Wind DEFL	Wind_ASCE716_3_X-Y
W21	Wind DEFL	Wind_ASCE716_4_X+Y_CW
W22	Wind DEFL	Wind_ASCE716_4_X+Y_CCW
W23	Wind DEFL	Wind_ASCE716_4_X-Y_CW
W24	Wind DEFL	Wind_ASCE716_4_X-Y_CCW
E5	EQ ELF Drift	EQ_ASCE716_X_+E_Drft
E6	EQ ELF Drift	EQ_ASCE716_X_-E_Drft
E7	EQ ELF Drift	EQ_ASCE716_Y_+E_Drft
E8	EQ ELF Drift	EQ_ASCE716_Y_-E_Drft

RESULTS:

Location (ft): (0.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	0.0010	-0.0564	0.0004	-0.0148	0.0000	0.0001
	Lp	0.0009	-0.0485	0.0002	-0.0123	0.0000	0.0001
	W13	1.6360	-0.0054	0.0828	0.0006	0.0005	0.0000
	W14	0.0065	0.4130	0.0003	0.0537	0.0000	0.0003
	W15	1.5888	0.3634	0.0807	0.0192	0.0005	0.0001
	W16	0.8651	-0.3714	0.0435	-0.0183	0.0003	0.0001
	W17	-0.3631	-0.0639	-0.0187	0.0212	0.0001	0.0001
	W18	0.3729	0.6834	0.0192	0.0593	0.0001	0.0003
	W19	1.2319	0.3057	0.0624	0.0408	0.0004	0.0002
	W20	1.2221	-0.3138	0.0619	-0.0398	0.0004	0.0002
	W21	1.4713	0.7851	0.0750	0.0589	0.0004	0.0003

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

Building Code: IBC

Drift

Page 2/4

07/15/21 17:06:12

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W22	0.3765	-0.3265	0.0186	0.0022	0.0001	0.0000
	W23	1.4640	0.3205	0.0746	-0.0015	0.0004	0.0000
	W24	0.3692	-0.7911	0.0182	-0.0582	0.0001	0.0003
	E5	2.8349	0.2708	0.2488	0.0252	0.0015	0.0001
	E6	2.3081	-0.2636	0.2023	-0.0217	0.0012	0.0001
	E7	-0.4814	0.8309	-0.0404	0.1769	0.0002	0.0010
	E8	0.5246	1.8516	0.0441	0.2621	0.0003	0.0015
Level 6	D	0.0006	-0.0415	0.0002	-0.0140	0.0000	0.0001
	Lp	0.0007	-0.0361	0.0002	-0.0116	0.0000	0.0001
	W13	1.5531	-0.0060	0.1410	-0.0001	0.0009	0.0000
	W14	0.0062	0.3593	0.0006	0.0516	0.0000	0.0003
	W15	1.5081	0.3442	0.1366	0.0311	0.0009	0.0002
	W16	0.8216	-0.3532	0.0749	-0.0313	0.0005	0.0002
	W17	-0.3444	-0.0852	-0.0309	0.0069	0.0002	0.0000
	W18	0.3537	0.6241	0.0318	0.0705	0.0002	0.0005
	W19	1.1695	0.2650	0.1062	0.0386	0.0007	0.0003
	W20	1.1602	-0.2740	0.1053	-0.0388	0.0007	0.0003
	W21	1.3963	0.7262	0.1263	0.0762	0.0008	0.0005
	W22	0.3579	-0.3287	0.0330	-0.0183	0.0002	0.0001
	W23	1.3894	0.3220	0.1256	0.0182	0.0008	0.0001
	W24	0.3510	-0.7329	0.0323	-0.0764	0.0002	0.0005
	E5	2.5861	0.2456	0.3344	0.0319	0.0022	0.0002
	E6	2.1058	-0.2420	0.2729	-0.0304	0.0018	0.0002
	E7	-0.4410	0.6539	-0.0542	0.1349	0.0004	0.0009
	E8	0.4805	1.5894	0.0590	0.2496	0.0004	0.0016
Level 5	D	0.0004	-0.0275	0.0001	-0.0096	0.0000	0.0001
	Lp	0.0005	-0.0245	0.0001	-0.0084	0.0000	0.0001
	W13	1.4122	-0.0059	0.1921	-0.0002	0.0013	0.0000
	W14	0.0056	0.3077	0.0008	0.0591	0.0000	0.0004
	W15	1.3715	0.3131	0.1861	0.0424	0.0012	0.0003
	W16	0.7467	-0.3218	0.1021	-0.0427	0.0007	0.0003
	W17	-0.3135	-0.0921	-0.0421	0.0011	0.0003	0.0000
	W18	0.3219	0.5536	0.0432	0.0876	0.0003	0.0006
	W19	1.0633	0.2264	0.1447	0.0442	0.0010	0.0003
	W20	1.0549	-0.2352	0.1435	-0.0445	0.0009	0.0003
	W21	1.2701	0.6500	0.1720	0.0975	0.0011	0.0006
	W22	0.3249	-0.3104	0.0450	-0.0312	0.0003	0.0002
	W23	1.2637	0.3039	0.1711	0.0309	0.0011	0.0002
	W24	0.3186	-0.6566	0.0442	-0.0977	0.0003	0.0006
	E5	2.2517	0.2137	0.3965	0.0380	0.0026	0.0002
	E6	1.8329	-0.2116	0.3236	-0.0359	0.0021	0.0002
	E7	-0.3868	0.5190	-0.0653	0.1370	0.0004	0.0009
	E8	0.4215	1.3398	0.0711	0.2751	0.0005	0.0018

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

Building Code: IBC

Drift

Page 3/4

07/15/21 17:06:12

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
Level 4	D	0.0003	-0.0179	0.0002	-0.0084	0.0000	0.0001
	Lp	0.0004	-0.0161	0.0002	-0.0074	0.0000	0.0000
	W13	1.2200	-0.0057	0.2393	0.0001	0.0016	0.0000
	W14	0.0049	0.2486	0.0009	0.0587	0.0000	0.0004
	W15	1.1854	0.2707	0.2313	0.0525	0.0015	0.0003
	W16	0.6446	-0.2792	0.1277	-0.0523	0.0008	0.0003
	W17	-0.2714	-0.0932	-0.0520	-0.0093	0.0003	0.0001
	W18	0.2787	0.4660	0.0534	0.0974	0.0004	0.0006
	W19	0.9187	0.1822	0.1802	0.0441	0.0012	0.0003
	W20	0.9114	-0.1907	0.1788	-0.0440	0.0012	0.0003
	W21	1.0981	0.5525	0.2135	0.1124	0.0014	0.0007
	W22	0.2799	-0.2793	0.0567	-0.0462	0.0004	0.0003
	W23	1.0926	0.2729	0.2125	0.0463	0.0014	0.0003
	W24	0.2744	-0.5589	0.0557	-0.1123	0.0004	0.0007
	E5	1.8553	0.1757	0.4452	0.0428	0.0029	0.0003
	E6	1.5094	-0.1757	0.3639	-0.0394	0.0024	0.0003
	E7	-0.3216	0.3821	-0.0738	0.1198	0.0005	0.0008
	E8	0.3504	1.0647	0.0804	0.2757	0.0005	0.0018
Level 3	D	0.0002	-0.0096	0.0001	-0.0044	0.0000	0.0000
	Lp	0.0002	-0.0087	0.0001	-0.0039	0.0000	0.0000
	W13	0.9807	-0.0058	0.2284	0.0004	0.0015	0.0000
	W14	0.0039	0.1898	0.0009	0.0564	0.0000	0.0004
	W15	0.9541	0.2182	0.2211	0.0506	0.0015	0.0003
	W16	0.5169	-0.2268	0.1215	-0.0501	0.0008	0.0003
	W17	-0.2194	-0.0839	-0.0500	-0.0089	0.0003	0.0001
	W18	0.2253	0.3687	0.0514	0.0935	0.0003	0.0006
	W19	0.7385	0.1380	0.1720	0.0425	0.0011	0.0003
	W20	0.7326	-0.1467	0.1706	-0.0420	0.0011	0.0003
	W21	0.8846	0.4401	0.2044	0.1081	0.0013	0.0007
	W22	0.2232	-0.2331	0.0536	-0.0442	0.0004	0.0003
	W23	0.8801	0.2266	0.2034	0.0446	0.0013	0.0003
	W24	0.2187	-0.4466	0.0526	-0.1077	0.0003	0.0007
	E5	1.4101	0.1329	0.3829	0.0374	0.0025	0.0002
	E6	1.1455	-0.1363	0.3127	-0.0335	0.0021	0.0002
	E7	-0.2478	0.2623	-0.0646	0.1059	0.0004	0.0007
	E8	0.2701	0.7891	0.0704	0.2423	0.0005	0.0016
Level 2	D	0.0001	-0.0052	0.0001	-0.0046	0.0000	0.0000
	Lp	0.0001	-0.0048	0.0001	-0.0042	0.0000	0.0000
	W13	0.7523	-0.0061	0.2293	-0.0007	0.0012	0.0000
	W14	0.0030	0.1334	0.0009	0.0629	0.0000	0.0003
	W15	0.7330	0.1676	0.2242	0.0525	0.0012	0.0003
	W16	0.3955	-0.1768	0.1197	-0.0535	0.0006	0.0003
	W17	-0.1694	-0.0750	-0.0524	-0.0068	0.0003	0.0000

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

Building Code: IBC

Drift

Page 4/4

07/15/21 17:06:12

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W18	0.1739	0.2752	0.0539	0.1011	0.0003	0.0005
	W19	0.5665	0.0955	0.1727	0.0467	0.0009	0.0002
	W20	0.5620	-0.1047	0.1712	-0.0477	0.0009	0.0003
	W21	0.6802	0.3321	0.2086	0.1152	0.0011	0.0006
	W22	0.1696	-0.1888	0.0504	-0.0452	0.0003	0.0002
	W23	0.6768	0.1819	0.2075	0.0445	0.0011	0.0002
	W24	0.1662	-0.3390	0.0494	-0.1160	0.0003	0.0006
	E5	1.0272	0.0955	0.3311	0.0325	0.0018	0.0002
	E6	0.8328	-0.1028	0.2677	-0.0317	0.0014	0.0002
	E7	-0.1832	0.1564	-0.0594	0.1216	0.0003	0.0006
	E8	0.1996	0.5468	0.0647	0.2475	0.0003	0.0013
Level 1	D	0.0001	-0.0006	0.0001	-0.0006	0.0000	0.0000
	Lp	0.0001	-0.0006	0.0001	-0.0006	0.0000	0.0000
	W13	0.5230	-0.0055	0.5230	-0.0055	0.0022	0.0000
	W14	0.0021	0.0705	0.0021	0.0705	0.0000	0.0003
	W15	0.5088	0.1151	0.5088	0.1151	0.0021	0.0005
	W16	0.2758	-0.1232	0.2758	-0.1232	0.0012	0.0005
	W17	-0.1169	-0.0683	-0.1169	-0.0683	0.0005	0.0003
	W18	0.1201	0.1741	0.1201	0.1741	0.0005	0.0007
	W19	0.3939	0.0488	0.3939	0.0488	0.0017	0.0002
	W20	0.3907	-0.0570	0.3907	-0.0570	0.0016	0.0002
	W21	0.4716	0.2169	0.4716	0.2169	0.0020	0.0009
	W22	0.1191	-0.1436	0.1191	-0.1436	0.0005	0.0006
	W23	0.4693	0.1375	0.4693	0.1375	0.0020	0.0006
	W24	0.1168	-0.2230	0.1168	-0.2230	0.0005	0.0009
	E5	0.6962	0.0630	0.6962	0.0630	0.0029	0.0003
	E6	0.5651	-0.0711	0.5651	-0.0711	0.0024	0.0003
	E7	-0.1238	0.0348	-0.1238	0.0348	0.0005	0.0001
	E8	0.1349	0.2993	0.1349	0.2993	0.0006	0.0013

0.0029*3=0.0087<.01 OK

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (01-27-21)

Building Code: IBC

Member Code Check

02/12/21 15:01:05

Steel Code: AISC360-16 LRFD

BEAM INFORMATION:

Story Level = Level 1 Frame Number = 3 Beam Number = 39
 Fy (ksi) = 50.00
 Beam Size = W33X130

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	32.00	8.00
Lu for Bending (ft) _____	32.00	8.00
K _____	1.00	1.00
Top Flange Continuously Braced _____	No	
Bottom Flange Continuously Braced _____	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.260 D + 0.500 Lp + 1.000 E2

Segment distance (ft) i - end _____ 24.00
 j - end _____ 32.00

SHEAR CHECK:

Vux (kip) = -77.03	1.00Vnx (kip) = 575.94	Vux/1.00Vnx = 0.134
Vuy (kip) = -0.00	0.90Vny (kip) = 530.96	Vuy/0.90Vny = 0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.400 D

Segment distance (ft) i - end _____ 0.00
 j - end _____ 8.00

AXIAL CHECK:

Pu (kip) = 0.00	0.90Pnx (kip) = 1723.50	Pu/0.90Pnx = 0.000
	0.90Pny (kip) = 1723.50	Pu/0.90Pny = 0.000
	0.90Pn (kip) = 1723.50	Pu/0.90Pn = 0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.260 D + 0.500 Lp - 1.000 E2

Segment distance (ft) i - end _____ 0.00
 j - end _____ 8.00

CALCULATED PARAMETERS:

Pu (kip) = 0.00	0.90Pn (kip) = 1723.50
Mux (kip-ft) = -809.20	0.90Mnx (kip-ft) = 1751.25
Muy (kip-ft) = 0.00	0.90Mny (kip-ft) = 223.13
Cbx = 1.34	

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.462

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RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (01-27-21)

Building Code: IBC

02/12/21 15:01:05

Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Level 1

Frame No: 3

Member No: 39

Fy (ksi): 50.00

Size: W33X130

Frame Type: Ordinary Moment Resisting Frame

E1.6b Beam-to-Column - FR Moment Connections

FR Moment Connections shall be designed for required flexure strength of (kip-ft) = 2354.46

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RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

Building Code: IBC

Member Code Check

07/15/21 17:11:04

Steel Code: AISC360-16 LRFD

COLUMN INFORMATION:

Story Level = Level 1 Frame Number = 3 Column Number = 2
 Fy (ksi) = 50.00
 Column Size = W14X665

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	19.75	19.75
Lu for Bending (ft) _____	19.75	19.75
K _____	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.200 D + 0.500 Lp + 1.000 W1

Shear	Top	Vux (kip) _____	45.48
		Vuy (kip) _____	0.06
Shear	Bot.	Vux (kip) _____	45.48
		Vuy (kip) _____	0.06

SHEAR CHECK:

Vux (kip) = 45.48	1.00Vnx (kip) = 1833.84	Vux/1.00Vnx = 0.025
Vuy (kip) = 0.06	0.90Vny (kip) = 4320.22	Vuy/0.90Vny = 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.200 D + 1.600 Lp

AXIAL CHECK:

Pu (kip) = 963.37	0.90Pnx (kip) = 8265.63	Pu/0.90Pnx = 0.117
	0.90Pny (kip) = 7271.67	Pu/0.90Pny = 0.132
	0.90Pn (kip) = 7271.67	Pu/0.90Pn = 0.132

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.200 D + 0.500 Lp + 1.000 W1

Axial	Load (kip) _____	599.90
Moment	Top	Mux (kip-ft) _____
		Muy (kip-ft) _____
		-1.21
Moment	Bot.	Mux (kip-ft) _____
		Muy (kip-ft) _____
		0.00
		0.00

CALCULATED PARAMETERS:

Pu (kip) = 599.90	0.90Pnx (kip) = 8265.63
	0.90Pny (kip) = 7271.67
Mux (kip-ft) = -835.43	0.90Mnx (kip-ft) = 5550.00
Muy (kip-ft) = -1.21	0.90Mny (kip-ft) = 2737.50
	Mcx (kip-ft) = 5513.07
KL/Rx = 29.80	KL/Ry = 51.38
Cbx = 1.67	

INTERACTION EQUATION:

Pu/0.90*Pn=0.073

Eq H1-3: 0.120 + 0.008 = 0.129

Eq H1-1b Per H1.3: 0.036 + 0.151 + 0.000 = 0.187

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RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 (06-29-21)

Building Code: IBC

07/15/21 17:12:02

Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Level 1

Frame No: 3

Member No: 2

Fy (ksi): 50.00

Size: W14X665

Frame Type: Ordinary Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pu (kip) = 621.01 --- Combination: 1.252 D + 0.500 Lp - 3.000 E2Max Pu/ ϕ Pn = 0.09 OK*Tension:* No tension on column

D2.5b Column Splices - Required Strength

*Design strength of column splices must meet or exceed the following forces:**Required tension and compression strength from D1.4a.**Shear in major axis (kip) = 119.39 --- Combination: 1.252 D + 0.500 Lp + 3.000 E2**Shear in minor axis (kip) = 6.44 --- Combination: 1.252 D + 0.500 Lp - 3.000 E3**Moment in major axis (kip-ft) = 2193.31 --- Combination: 1.252 D + 0.500 Lp + 3.000 E2**Moment in minor axis (kip-ft) = 127.25 --- Combination: 1.252 D + 0.500 Lp - 3.000 E3**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.*

D2.5c Required Shear Strength

	Major	Minor
Mpc (kip-ft)	6166.67	3041.67

Column Splice Shear Force Required Mpc / H

Where Mpc is the lesser nominal plastic flexural strength of the column sections.

See code for more information on the Required Shear Strength

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STE BM W33x130

$$\begin{aligned} A &= 38.2 \text{ in}^2 & d &= 33.1" & t_w &= 0.58" & b_f &= 11.5" \\ \phi_f &= 0.855" & k_{col} &= 1.56" & k_{end} &= 2\frac{1}{8}" & h &= 1\frac{1}{4}" \\ Z_x &= 467 \text{ in}^3 \end{aligned}$$

STE COL W14x60

$$\begin{aligned} A &= 17.5 \text{ in}^2 & d &= 20.9" & t_w &= 2.6" & b_f &= 17.4" \\ \phi_f &= 4.16" & k_{col} &= 4.76" & k_{end} &= 5.81" & h &= 2.625" \\ Z_x &= 1320 \text{ in}^3 \end{aligned}$$

Max Mom @ Hinge Zone

$$M_{pr} = 155 \text{ k-ft} = 18684 \text{ k-in}$$

Max Bolt Dia

$$d_b \leq \frac{b_f}{2} \left(1 - \frac{A_b F_u}{R_t F_n} \right) = \frac{11.5}{2} \left(1 - \frac{1.1(50)}{1.1(65)} \right) = 1.22" \rightarrow \text{use } 7/8"$$

Determine Req'd # of Bolts

$$n = \begin{cases} 1.0 F_{nv} A_b = 1.0 (84 \text{ ksi}) (6.01 \text{ in}^2) \rightarrow 505 \text{ k/bolt} \rightarrow \text{controls} \\ 2.4 F_u b d_{btf} = 2.4 (1.1) (65 \text{ ksi}) (7/8") (8.55") = 128 \text{ k/bolt} \\ 2.4 F_u p d_{btf} = 2.4 (1.1) (65 \text{ ksi}) (7/8") (1\frac{1}{4}") = 225 \text{ k/bolt} \end{cases}$$

$$n = \frac{1.25 M_{pr}}{\phi_n (d + r)} = \frac{1.25 (18684 \text{ k-in})}{.9 (505 \text{ k/bolt}) (21.5" + 1\frac{1}{4}")} = 15.7 \text{ bolts} \rightarrow \text{use } (16) \text{ bolts}$$

Determine Plastic Hinge Zone

$$S_h = S + S(\frac{1}{2} - 1) = 4\frac{1}{2}" + 0(8 - 1) = 2\frac{1}{2}"$$

Determine Force in Flange

$$F_{fr} = \frac{18684 \text{ k-in}}{21.5" + 1\frac{1}{2}"} = 540 \text{ kips}$$

$$n = \frac{540 \text{ k}}{.9 (505 \text{ k/bolt})} = 12 < 16 \rightarrow \text{OK}$$

Check R Yields

$$\phi R_n = \phi F_y A_g = .9 (50 \text{ ksi}) (1.71' (11.71')) = 776 \text{ k} > 540 \text{ k}$$

Check R Rupture

$$\phi R_n = \phi F_u A_n = .9 (65 \text{ ksi}) (1.71' (11.71' - 2 (1.5/16'))) = 845 \text{ k} > 540 \text{ k}$$

Check BM Block Shear Rupture

$$R_n = \min (.6 F_u A_{nv} + F_u A_{nt}, .6 F_y A_{gv} + F_u A_{nt}) = 1443 \text{ k}$$

$$A_g = (2) (25') (8.5') = 42.5 \text{ in}^2$$

$$A_{nv} = (2) (8.5') (25' - 7/2 (1.5/16')) = 39.7 \text{ in}^2$$

$$A_{nt} = (2) (8.5') (1' - 7/2 (1.5/16')) = 1.76 \text{ in}^2$$

$$\phi R_n = .9 (1443 \text{ k}) = 1300 \text{ k} > 540 \text{ k}$$

Check R Bending

$$L = 4'$$

$$k = 0.65$$

$$k_y = \frac{.65 (4')}{1.5' / \sqrt{12}} = 6 < 25 \rightarrow \phi R_n = \phi F_y A_g$$

$$\phi R_n = \phi F_y A_g = .9 (50 \text{ ksi}) (1.71' (11.71')) = 776 > 540 \text{ k}$$

Check Shear conn.

$$V_n = \frac{2 M_p}{L_h} + V_{gr} = \frac{2 (18684 \text{ k-in})}{31.5'} + 754/2 = 156 \text{ k}$$

Check column Panel zone Shear

$$R_n = \frac{(2) M_p}{d_c + (2) l_p} = \frac{(2) 781 \text{ k-ft}}{20.9' + (2) 1.71'} = 794 \text{ k-in}$$

$$\phi R = \phi (.6) F_y d_c t_w \left(1 + \frac{3 b_f t_w^2}{d_c d_c t_w} \right) = 1.0 (.6) 50 \text{ ksi} (20.9') (2.6') \left(1 + \frac{3 (17.4') (4.16')^2}{32.1' (20.9') (2.6')} \right) = 2400 \text{ k-in} > R_n$$

2018 IBC Risk Category IV Seismic Design Category D

This section presents representative structural calculations for Building 2 (7-story healthcare facility in Long Island, New York) Design Case E. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v2A.

TABLE OF CONTENTS

DESIGN CRITERIA.....	1
GRAVITY DESIGN.....	11
BEAM DESIGN.....	12
COLUMN DESIGN.....	17
LATERAL DESIGN.....	21
BRACED FRAME DESIGN.....	44
MOMENT FRAME DESIGN.....	51

DESIGN CRITERIA

Project: ATC BLDG 2 RC4 SDC D

Job Number: 19-051

Sheet: _____ of _____

Name: JAD

Originating Office: Tacoma

Date: 08/25/21

DESIGN CRITERIA CHECKLIST

CODE: IBC 2018, ASCE 7-16

LOCATION: Long Island, NY

VERTICAL DESIGN CRITERIA

	DEAD	LIVE	PARTITION	CONCENTRATED
ROOF:	100 PSF	40 PSF		
OPERATING ROOMS	95 PSF	60 PSF		1000 #
LIBRARY:	95 PSF	150 PSF		1000 #
PRIVATE ROOMS	95 PSF	100 PSF		
CORRIDORS ABOVE THE 1st FLOOR	95 PSF	80 PSF		

WIND DESIGN CRITERIA

BASIC WIND SPEED (V) = 140 MPH	▼	(Per ASCE 7-16 Sec. 26.5.1, Fig. 26.5-1A; 1B; 1C & 1D, or as required by Bld'g Dept.)
RISK CATEGORY: III & IV	▼	(Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)
EXPOSURE CATEGORY: B	▼	(Per ASCE 7-16 Section 26.7.3)
TOPOGRAPHIC FACTOR (K_{zt}): <u>1.00</u>	▼	(Per ASCE 7-16 Section 26.8.2)
MEAN ROOF HEIGHT: 30 FT	▼	(See ASCE 7-16 Section 26.2 - Definitions)
ELEVATION: <u>0 FT</u>	▼	(See ASCE 7-16 Section 26.9)
ENCLOSURE CLASSIFICATION: Enclosed	▼	(See ASCE 7-16 Section 26.2 & Table 26.13-1)
ROOF TYPE: Monoslope	▼	(See ASCE 7-16 Figure 27.3-1)
ROOF SLOPE (α :12): <u>0.00:12</u>		(Enter vertical rise in 12 horizontal units)
θ (degrees): <u>0.00</u>		

SEISMIC DESIGN CRITERIA

RISK CATEGORY: IV	▼	(Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)
SITE CLASS: D	▼	(Per IBC Section 1613.2.2, Assumed as "D" or per Geotech.)
IMPORTANCE FACTOR (I_E): <u>1.5</u>		(Per ASCE 7-16 Table 1.5-2)
STRUCTURAL SYSTEM (R): <u>8</u>		(Per ASCE 7-16 Table 12.2-1)
OVERSTRENGTH FACTOR (Ω_o): <u>3.0</u>		(Per ASCE 7-16 Table 12.2-1)
INFORMATION BELOW FROM "EARTHQUAKE SPECTRAL RESPONSE ACCELERATION MAPS" PER USGS		
LATITUDE: <u>40.647</u>	$S_s =$ <u>0.245</u>	$F_a =$ <u>2.400</u>
LONGITUDE: <u>-73.564</u>	$S_1 =$ <u>0.055</u>	$F_v =$ <u>4.200</u>

DEFLECTION CRITERIA

FLOOR (LIVE):	L/ 480	▼	ROOF (LIVE):	L/ 360	▼
FLOOR (TOTAL):	L/ 360	▼	ROOF (TOTAL):	L/ 240	▼
WALLS:	L/ 360	▼	SPECIAL:	L/	▼

SOIL DESIGN CRITERIA

REPORT: <u>NO</u>	SEE SOILS REPORT FOR ACTIVE, PASSIVE PRESSURES AND FRICTION COEFFICIENT	
BEARING: <u>6000 PSF</u>	MINIMUM FOOTING DIMENSIONS:	
ACTIVE: <u>35 PCF</u>		
PASSIVE: <u>250 PCF</u>		
COEFFICIENT OF FRICTION: <u>0.35</u>	CONTINUOUS:	<u>1'-4"</u>
PILE TYPE: <u>NONE</u>	SPREAD:	<u>1'-6"</u>
VERTICAL CAPACITY: <u>N/A</u>	FROST DEPTH:	<u>1'-6"</u>
UPLIFT CAPACITY: <u>N/A</u>	LATERAL CAPACITY:	<u>N/A</u>
	SIZE:	<u>N/A</u>

Project: ATC BLDG 2 RC4 SDC D

Job Number: 19-051

Sheet: of

Name: JAD

Originating Office: Tacoma

Date: 08/25/21

MATERIALS

CONCRETE

Footings/Piles:	3000 PSI	Columns:	4000 PSI
Slabs/Walls:	4000 PSI	Beams:	4000 PSI
-	-	-	-

REINFORCING

Steel Grade = 60 $f_y = 60$ KSI

STRUCTURAL STEEL

W-Flange Beams	ASTM A992	$f_y = 50$ KSI
Shapes & Plates	ASTM A36	$f_y = 36$ KSI
Pipes	ASTM A53, Grade B	$f_y = 35$ KSI
HSS Rect.	ASTM A500, Grade C	$f_y = 50$ KSI
HSS Round	ASTM A500, Grade C	$f_y = 46$ KSI

MASONRY

ASTM C90 $f'_m = 1900$ PSI

SOLID GROUTED

GLULAM BEAMS

Simple Spans

24F-V4
 $1.80E+06$ PSI
2400 PSI
1850 PSI
240 PSI

Grade =
E =
 $F_{b(BOTTOM)} =$
 $F_{b(TOP)} =$
 $F_v =$

Cantilevers

24F-V8
 $1.80E+06$ PSI
2400 PSI
2400 PSI
240 PSI

SCL PRODUCTS

	2x SCL	1½" SCL	3½, 5¼ SCL
E =	$1.30E+06$ PSI	$1.80E+06$ PSI	$2.00E+06$ PSI
$F_b =$	1700 PSI	2600 PSI	2900 PSI
$F_v =$	285 PSI	285 PSI	285 PSI
$F_c =$	1400 PSI	2400 PSI	2600 PSI

FRAMING LUMBER

<u>Joists & Studs</u>	2x DF #2	2x HF #1	-
E =	$1.60E+06$ PSI	$1.50E+06$ PSI	-
$F_b =$	900 PSI	975 PSI	-
$F_v =$	180 PSI	150 PSI	-
$F_c =$	1350 PSI	1350 PSI	-
<u>Beams & Headers</u>	4x DF #2	4x HF #1	6x DF #1
E =	$1.60E+06$ PSI	$1.50E+06$ PSI	$1.60E+06$ PSI
$F_b =$	900 PSI	975 PSI	1350 PSI
$F_v =$	180 PSI	150 PSI	170 PSI
<u>Posts & Timbers</u>	6x DF #1	-	-
E =	$1.60E+06$ PSI	-	-
$F_c =$	1000 PSI	-	-

DESIGN CRITERIA - WIND

FIGURE 27.3-8: Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases per ASCE 7-16

FIGURE 27.3-8 Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases

DESIGN CRITERIA - WIND

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, C_p , for Enclosed and Partially Enclosed Buildings - Walls and Roofs per ASCE 7-16

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, C_p , for Enclosed and Partially Enclosed Buildings—Walls and Roofs

DESIGN CRITERIA - WIND

FIGURE 30.3-1: Components and Cladding [$h \leq 60$ ft]: External Pressure Coefficients, (G_Cp), for Enclosed and Partially Enclosed Buildings - Walls

Diagram

Notation

a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

Exception: For buildings with $\theta = 0^\circ$ to 7° and a least horizontal dimension greater than 300 ft (90 m), dimension a shall be limited to a maximum of $0.8h$.

h = Mean roof height, in ft (m), except that eave height shall be used for $\theta \leq 10^\circ$.

θ = Angle of plane of roof from horizontal, in degrees.

External Pressure Coefficient, (G_Cp) - Walls

Notes

1. Vertical scale denotes (G_Cp) to be used with qh .
2. Horizontal scale denotes effective wind area, in ft^2 (m^2).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (G_Cp) for walls shall be reduced by 10% when $\theta \leq 10^\circ$.

FIGURE 30.3-1 Components and Cladding [$h \leq 60$ ft ($h \leq 18.3$ m)]: External Pressure Coefficients, (G_Cp), for Enclosed and Partially Enclosed Buildings—Walls

DESIGN CRITERIA - SEISMIC

ASCE 7-16 SECTION 12.8 - EQUIVALENT LATERAL FORCE PROCEDURE

RISK CATEGORY:	IV	LATITUDE:	40.647
SITE CLASS:	D	LONGITUDE:	-73.564
IMPORTANCE FACTOR (I_E):	1.5	$S_S =$	0.245
STRUCTURAL SYSTEM (R):	8	$S_1 =$	0.055
OVERSTRENGTH FACTOR (Ω_o):	3	$F_a =$	2.400
		$F_v =$	4.200

ASCE 7-16 SECTION 11.4 SEISMIC GROUND MOTION VALUES

Section 11.4.4 - Coefficients and Risk-Targeted Maximum Considered Earthquake (MCER) Spectral Response Acceleration Parameters

$$S_{MS} = F_a * S_S = 0.588 \quad S_{M1} = F_v * S_1 = 0.231$$

Section 11.4.5 - Design Spectral Response Acceleration Parameters

$$S_{DS} = 2/3 * S_{MS} = 0.392 \quad S_{D1} = 2/3 * S_{M1} = 0.154$$

ASCE 7-16 SECTION 11.6 - SEISMIC DESIGN CATEGORY - SECTION 12.8.2 - PERIOD DETERMINATION

ASCE 7-16 TABLE 11.6-1			
SEISMIC DESIGN CATEGORY BASED ON S_{DS}			
	RISK CATEGORY:		
	I & II	III	IV
$< 0.167g$	A	A	A
$< 0.33g$	B	B	C
$< 0.50g$	C	C	D
$\geq 0.50g$	D	D	D
	D		

ASCE 7-16 TABLE 11.6-2			
SEISMIC DESIGN CATEGORY BASED ON S_{D1}			
	RISK CATEGORY:		
	I & II	III	IV
$< 0.067g$	A	A	A
$< 0.133g$	B	B	C
$< 0.20g$	C	C	D
$\geq 0.20g$	D	D	D
	D		

Each building and structure shall be assigned to the most severe Seismic Design Category in accordance with Table 11.6-1 or Table 11.6-2, irrespective of the fundamental period of vibration of the structure.

PERIOD DETERMINATION:

$$C_t = 0.028$$

$$h_n = 100 \text{ FT}$$

$$x = 0.8$$

$$T_a = C_t * h_n^x = 1.115$$

ASCE 7-16 SECTION 12.8.1.1 - SEISMIC RESPONSE COEFFICIENT

GENERAL EQUATION: $C_S = S_{DS}/(R/I) = 0.074$ EQ. 12.8-2

MAXIMUM: $C_S = S_{D1}/(T*(R/I)) = 0.026$ <--CONTROLS EQ. 12.8-3

MINIMUM: $C_S = 0.044 * S_{DS} * I > 0.01 = 0.026$ EQ. 12.8-5

For structures located where $S_1 > 0.6g$
 $C_S = 0.5 * S_1 / (R/I) = 0.000$ EQ. 12.8-6

ASCE 7-10 SECTION 12.8.1 - SEISMIC BASE SHEAR

$$V = C_S * W = 0.026 * W$$

W = the total dead load and applicable portion of other loads as indicated in Section 12.7.2

Search Information

Address: Long Island City, Queens, NY, USA

Coordinates: 40.744679, -73.9485424

Elevation: 12 ft

Timestamp: 2020-11-20T22:34:24.331Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: IV

Site Class: E

MCE_R Horizontal Response Spectrum

Design Horizontal Response Spectrum

Basic Parameters

Name	Value	Description
S _S	0.289	MCE _R ground motion (period=0.2s)
S ₁	0.06	MCE _R ground motion (period=1.0s)
S _{MS}	0.661	Site-modified spectral acceleration value
S _{M1}	0.251	Site-modified spectral acceleration value
S _{DS}	0.441	Numeric seismic design value at 0.2s SA
S _{D1}	0.167	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F _a	2.292	Site amplification factor at 0.2s
F _v	4.2	Site amplification factor at 1.0s

CR_S	0.935	Coefficient of risk (0.2s)
CR_1	0.944	Coefficient of risk (1.0s)
PGA	0.178	MCE_G peak ground acceleration
F_{PGA}	2.01	Site amplification factor at PGA
PGA_M	0.358	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
Ss_{RT}	0.289	Probabilistic risk-targeted ground motion (0.2s)
Ss_{UH}	0.309	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
Ss_D	1.5	Factored deterministic acceleration value (0.2s)
$S1_{RT}$	0.06	Probabilistic risk-targeted ground motion (1.0s)
$S1_{UH}$	0.063	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
$S1_D$	0.6	Factored deterministic acceleration value (1.0s)
PGA_d	0.5	Factored deterministic acceleration value (PGA)

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

Disclaimer

Hazard loads are provided by the U.S. Geological Survey [Seismic Design Web Services](https://seismic.designweb.org/).

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Search Information

Coordinates: 40.647053, -73.5641258
Elevation: 42 ft
Timestamp: 2021-06-29T16:29:22.950Z
Hazard Type: Wind

ASCE 7-16

MRI 10-Year	75 mph
MRI 25-Year	84 mph
MRI 50-Year	93 mph
MRI 100-Year	99 mph
Risk Category I	110 mph
Risk Category II	122 mph
Risk Category III	▲ 131 mph

If the structure under consideration is a healthcare facility and you are also within 1 mile of the coastal mean high water line, you are in a wind-borne debris region. If other occupancy, use the Risk Category II basic wind speed contours to determine if you are in a wind-borne debris region.

Risk Category IV ▲ 137 mph

You are in a wind-borne debris region if you are also within 1 mile of the coastal mean high water line.

ASCE 7-10

MRI 10-Year	77 mph
MRI 25-Year	87 mph
MRI 50-Year	94 mph
MRI 100-Year	102 mph
Risk Category I	114 mph
Risk Category II	125 mph
Risk Category III-IV	▲ 135 mph

If the structure under consideration is a healthcare facility and you are also within 1 mile of the coastal mean high water line, you are in a wind-borne debris region. If other occupancy, use the Risk Category II basic wind speed contours to determine if you are in a wind-borne debris region.

ASCE 7-05

ASCE 7-05 Wind Speed ▲ 115 mph

You are in a wind-borne debris region if you are also within 1 mile of the coastal mean high water line.

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

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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GRAVITY DESIGN

BEAM DESIGN

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (10-20-20)

Building Code: IBC

Floor Map

10/20/20 10:06:28

Steel Code: AISC 360-16 LRFD

Floor Type: Floor

Beam Numbers

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Beam Design

02/02/21 12:25:21

Steel Code: AISC 360-16 LRFD

Floor Type: Floor**Beam Number = 98****SPAN INFORMATION (ft): I-End (47.25,-123.50) J-End (47.25,-91.50)**Beam Size (User Selected) = W21X50 Fy = 50.0 ksi

Total Beam Length (ft) = 32.00

COMPOSITE PROPERTIES (Not Shored):

		Left		Right
Deck Label		Deck		Deck
Concrete thickness (in)		4.50		4.50
Unit weight concrete (pcf)		150.00		150.00
f _c (ksi)		5.00		5.00
Decking Orientation		perpendicular		perpendicular
Decking type		VERCO W3 Formlok		VERCO W3 Formlok
beff (in)	=	91.50	Y bar(in)	= 23.01
M _{nf} (kip-ft)	=	1038.49	M _n (kip-ft)	= 846.60
C (kips)	=	372.16	PNA (in)	= 20.19
I _{eff} (in ⁴)	=	3120.64	I _{tr} (in ⁴)	= 3986.68
Stud length (in)	=	4.50	Stud diam (in)	= 0.75
R _g Stud Red. Factor	R _g [1] = 1.00		R _g [2] = 0.85	
R _p Stud Red. Factor	R _p [1] = 0.60		R _p [2] = 0.60	
Stud Capacity (kips)	Q _n [1] = 17.2		Q _n [2] = 14.6	
# of studs:	Max = 64	Partial = 48	Actual = 48	
Number of Stud Rows = 2		Percent of Full Composite Action = 50.63		

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.724	0.610	0.610	---	NonR	0.114	0.152
	32.000	0.724	0.610	0.610			0.114	0.152

SHEAR (Ultimate): Max V_u (1.2DL+1.6LL) = 32.45 kips 1.00V_n = 237.12 kips**MOMENTS (Ultimate):**

Span	Cond	LoadCombo	Mu	@	Lb	Cb	Phi	Phi*Mn
			kip-ft	ft	ft			kip-ft
Center	PreCmp+	1.2DL+1.6LL	124.9	16.0	0.0	1.00	0.90	412.50
	Init DL	1.4DL	109.3	16.0	---	---		
	Max +	1.2DL+1.6LL	259.6	16.0	---	---	0.90	761.94
Controlling		1.2DL+1.6LL	259.6	16.0	---	---	0.90	761.94

REACTIONS (kips):

	Left	Right
Initial reaction	12.20	12.20
DL reaction	11.59	11.59
Max +LL reaction	11.59	11.59
Max +total reaction (factored)	32.45	32.45

DEFLECTIONS:

			Ratio
Initial load (in)	at 16.00 ft =	-0.504	L/D = 761
Live load (in)	at 16.00 ft =	-0.189	L/D = 2033 > 560
Post Comp load (in)	at 16.00 ft =	-0.219	L/D = 1756 > 240

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

RAM Structural System

Gravity Beam Design

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Page 2/2

02/02/21 12:25:21

Steel Code: AISC 360-16 LRFD

Net Total load (in)	at 16.00 ft =	-0.723	L/D =	531 >	240	0.45
---------------------	---------------	--------	-------	-------	-----	------

Project ID:
Project # :
Bay ID : 1-Floor: B-C , 5-4
Model : ATC BLDG 2 ASCE RC 3 (10-20-20)

VIBRATION ANALYSIS:

Equipment: User Defined
Evaluation Criterion: Sensitive Equipment
Reference: Murray, T.M., Allen, D.E. and Ungar, E.E.,
"Floor Vibrations Due to Human Activity",
AISC Design Guide #11, 1997

Vibrational Velocity: V= 6000. μ -in/sec (User Defined)
Beams are not continuous at girders
Girders are not continuous at columns

EVALUATION:

Walking Speed (Steps/min)	V (μ -in/sec)	Evaluation V(limit) (μ -in/sec)	Criterion
100	26932	> 6000	Not Satisfied
75	5985	<= 6000	Satisfied
50	1609	<= 6000	Satisfied

PARAMETER SUMMARY:

Parameter	Beam	Left Girder	Right Girder	
Member	W21X50	W24X68	W24X68	
Uniform Load, plf	751.5	1644.9	2646.8	
Effective I, in ⁴	4150.8	6410.2	7206.5	
Frequency, Hz	9.21	8.52	7.12	5.63

FRAMING: (User Modified Bay)

Girder Span = 30.50 ft
Beam Spans:
Left = 0.00 ft
Center = 32.00 ft
Right = 20.33 ft

Girders/Walls:
Left -W24X68
Right -W24X68

Beam -W21X50
4 spaces at 91.50 in

Concrete: dc = 7.50 in
f'c = 5.00 ksi
wt. = 150.0 pcf
Deck Height = 3.00 in

Loading: Dead = 4.00 psf
Live = 11.00 psf
Collateral = 0.00 psf

Warning: The bay configuration and/or properties analyzed here have been modified by the user and differ from the actual configuration and/or properties of the bay in the RAM Structural System.

COLUMN DESIGN

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Column Design

02/12/21 11:17:28

Steel Code: AISC360-16 LRFD

Story level Level 2, Column Line B-4, Column # 7

Fy (ksi) = 50.00

Column Size

= W14X176

Orientation (deg.) = 0.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	15.67	15.67
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	10.10	10.35
Bottom _____	10.10	10.35

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	545.17	488.28	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 1435.44	0.90Pnx (kip) = 2189.60	Pu/0.90Pnx = 0.656
	0.90Pny (kip) = 1986.72	Pu/0.90Pny = 0.723
	0.90Pn (kip) = 1986.72	Pu/0.90Pn = 0.723

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 3:

	Dead	Live	Roof
Axial (kip) _____	545.17	488.28	0.00
Moments Top Mx (kip-ft) _____	-1.64	-1.64	0.00
My (kip-ft) _____	0.21	0.21	0.00
Bot Mx (kip-ft) _____	-2.05	-16.72	0.00
My (kip-ft) _____	0.27	5.71	0.00

Reverse curvature about X-Axis

Reverse curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 1435.44	0.90*Pn (kip) = 1986.72
Mux (kip-ft) = -29.21	0.90*Mnx (kip-ft) = 1200.00
Muy (kip-ft) = 9.46	0.90*Mny (kip-ft) = 611.25
Rm = 1.00	
Cbx = 1.86	
Cmx = 0.54	Cmy = 0.57
Pex (kip) = 17322.52	Pey (kip) = 6783.30
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.723

Eq H1-1a: $0.723 + 8/9(0.024 + 0.015) = 0.758$ This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Column Design

Page 2/2

02/12/21 11:17:28

Steel Code: AISC360-16 LRFD

Story level Level 1, Column Line B-4, Column # 7

Fy (ksi) = 50.00

Column Size

= W14X176

Orientation (deg.) = 0.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	19.75	19.75
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	10.10	10.35
Bottom _____	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	635.24	578.35	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 1687.64	0.90Pnx (kip) = 2110.42	Pu/0.90Pnx = 0.800
	0.90Pny (kip) = 1808.38	Pu/0.90Pny = 0.933
	0.90Pn (kip) = 1808.38	Pu/0.90Pn = 0.933

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 6:

	Dead	Live	Roof
Axial (kip) _____	635.24	567.03	0.00
Moments Top Mx (kip-ft) _____	-1.63	-1.63	0.00
My (kip-ft) _____	0.21	4.53	0.00
Bot Mx (kip-ft) _____	0.00	0.00	0.00
My (kip-ft) _____	0.00	0.00	0.00

Single curvature about X-Axis

Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 1669.53	0.90*Pn (kip) = 1808.38
Mux (kip-ft) = -4.56	0.90*Mnx (kip-ft) = 1200.00
Muy (kip-ft) = 7.50	0.90*Mny (kip-ft) = 611.25
Rm = 1.00	
Cbx = 1.67	
Cmx = 0.60	Cmy = 0.60
Pex (kip) = 10904.73	Pey (kip) = 4270.17
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.923

Eq H1-1a: $0.923 + 8/9(0.004 + 0.012) = 0.938$ This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

LATERAL

Seismic

Site Class info

Site Class E $\rightarrow S_{DS} = 0.392$

$V_{ult} = 137 \text{ MPH}$

Seismic

BF

$R = 6$

$C_s = \frac{S_{DS}}{R} (I_e) = \frac{0.392}{6} (1.50) = 0.098$

$W = 100 \text{ psf} (1.5630 \text{ ft}^2) + (95 \text{ psf} + 100 \text{ psf}) (1.5630 \text{ ft}^2) 6 = 11410 \text{ lb}$

$V_{Fa} = C_s W = 0.098 (11410 \text{ lb}) = 1120 \text{ lb}$

MF

$R = 8$

$C_s = \frac{S_{DS}}{R} (I_e) = \frac{0.392}{8} (1.50) = 0.074$

$W = 100 \text{ psf} (1.5630 \text{ ft}^2) + (95 \text{ psf} + 100 \text{ psf}) (1.5630 \text{ ft}^2) 6 = 11410 \text{ lb}$

$V_{Fa} = C_s W = 0.074 (11410 \text{ lb}) = 840 \text{ lb}$

Check min & max C_s

$T = C_t h^x = 0.28 (100)^x = 1.15 @ \text{MF}$
 $= \quad \quad = 0.28 (100)^{0.75} = 0.63 @ \text{BF}$ } $T_L = 6 \text{ sec}$

$\text{Max } C_s = \frac{S_{D1}}{T C_{Te}} = \frac{0.154}{1.15 (8/1.5)} = 0.025 @ \text{MF} \rightarrow V = 0.025 (W) = 287 \text{ lb}$

$= \quad \quad = \frac{0.154}{0.63 (8/1.5)} = 0.061 @ \text{BF} \rightarrow V = 0.061 (W) = 700 \text{ lb} \rightarrow \text{Seismic controlled @ BF}$

$\text{Min } C_s = 0.044 S_{D1} I_e = 0.044 (0.392) 1.50 = 0.025$

Wind Loads

- Exposure B
- Ultimate Wind Speed = 137 mph
- $I_w = 1.0$

$$q_z = .00256 k_z k_{zt} k_d k_e V^2 = .00256 k_z (85) (10) 129^2 = 36.2 \text{ psf } k_z$$

Where:

k_z = varies
 $k_{zt} = 1.0$
 $k_d = 0.85$
 $k_e = 1.0$

q_z as a function of height

$q_{15} \rightarrow k_z = .57 \rightarrow q_{15} = 23.2 \text{ psf}$
 $q_{20} \rightarrow k_z = 0.62 \rightarrow q_{20} = 25.4 \text{ psf}$
 $q_{25} \rightarrow k_z = 0.66 \rightarrow q_{25} = 27.0 \text{ psf}$
 $q_{30} \rightarrow k_z = 0.70 \rightarrow q_{30} = 28.5 \text{ psf}$
 $q_{40} \rightarrow k_z = 0.76 \rightarrow q_{40} = 31.0 \text{ psf}$
 $q_{50} \rightarrow k_z = 0.81 \rightarrow q_{50} = 33.0 \text{ psf}$
 $q_{60} \rightarrow k_z = 0.85 \rightarrow q_{60} = 34.7 \text{ psf}$
 $q_{70} \rightarrow k_z = 0.89 \rightarrow q_{70} = 36 \text{ psf}$
 $q_{80} \rightarrow k_z = 0.93 \rightarrow q_{80} = 37.7 \text{ psf}$
 $q_{90} \rightarrow k_z = 0.96 \rightarrow q_{90} = 38.0 \text{ psf}$
 $q_{100} \rightarrow k_z = 0.99 \rightarrow q_{100} = q_h = 40.4 \text{ psf}$

Wind Pressure

$$P_e = q_z G C_p - q_h G C_{pe} = G (q_z C_p - q_h C_{pe})$$

Where:

G = Gust Factor $\rightarrow$ see next page $\rightarrow$ ASCE 7-16 26.11
 C_p = 0.8 For windward, -0.5 For Leeward

Gust Factor For MF
Natural Frequency

Left = 124' < 4 times h → use approx Eqn 26.11-2

$$\eta = \frac{22.2}{h^{.8}} = \frac{22.2}{100^{.8}} = .55 < 1 \rightarrow \text{Flexible}$$

Determine Gust Factor

$$I_z = C \left(\frac{z}{z_0} \right)^{.6} = .3 \left(\frac{33}{600} \right)^{.6} = .272$$

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{0.4h}{L_z} \right)^{.63}}} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{124 + 100}{390.6} \right)^{.63}}} = 0.83$$

Where:

$$L_z = 1.6 \left(\frac{z}{33} \right)^{.8} = 320 \left(\frac{60}{33} \right)^{.8} = 390.6'$$

$$g_R = g_V = 3.4$$

$$g_R = \sqrt{2 \ln(3600h)} + \frac{0.577}{\sqrt{2 \ln(3600h)}} = 4.04$$

$$R = \sqrt{\frac{1}{2} R_h R_b R_o (.53 + 4.7 R_o)} = \sqrt{\frac{1}{2} (.083) (.207) (.259) (.53 + 4.7 (.083))} = 0.275$$

Where:

$$V_z = \bar{V} \left(\frac{z}{33} \right)^{.08} \left(\frac{88}{60} \right)^{.1} V$$

$$= 0.45 \left(\frac{60}{33} \right)^{.4} \left(\frac{88}{60} \right)^{.125} = 95.8$$

$$N_1 = \frac{\eta L_z}{V_z} = \frac{.55 (390.6)}{95.8} = 2.24$$

$$R_h = \frac{7.47 N_1}{(1 + 0.3 N_1)^{.53}} = \frac{7.47 (2.24)}{(1 + 0.3 (2.24))^{.53}} = .083$$

$$\eta_h = 4.6 n h / V_z = 4.6 (.55)^{100} / 95.8 = 2.64 \rightarrow R_h = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .207$$

$$\eta_b = 4.6 n h / V_z = 4.6 (.55)^{124} / 95.8 = 3.27 \rightarrow R_b = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .259$$

$$\eta_L = \frac{15.4 n L}{V_z} = \frac{15.4 (.55) 124}{95.8} = 10.96 \rightarrow R_L = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .087$$

$$G_F = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_R^2 Q^2 + g_o^2 L^2}}{1 + 1.7 g_o L} \right) = 0.925 \left(\frac{1 + 1.7 (.272) \sqrt{3.4^2 (.8)^2 + 4.04^2 (.275)^2}}{1 + 1.7 (3.4) (.272)} \right) = 0.84$$

Gust Factor For DF

Natural Frequency

Left = 124' < 4 times h → use approx Eqn 26.11-2

$$n = \frac{75}{h} = \frac{75}{100} = 0.75 \rightarrow \text{Flexible}$$

Determine Gust Factor

$$I_z = C \left(\frac{z}{z_0} \right)^{1/6} = .3 \left(\frac{33}{600} \right)^{1/6} = .272$$

$$Q = \sqrt{\frac{1}{1 + 0.65 \left(\frac{0.4h}{L_z} \right)^{6.5}}} = \sqrt{\frac{1}{1 + 0.65 \left(\frac{124 + 100}{390.6} \right)^{6.5}}} = 0.83$$

Where:

$$L_z = 1.6 \left(\frac{z}{33} \right)^{.5} = 320 \left(\frac{60}{33} \right)^{.5} = 390.6'$$

$$g_R = g_V = 3.4$$

$$g_R = \sqrt{2 \ln(3600h)} + \frac{0.577}{\sqrt{2 \ln(3600h)}} = 4.12$$

$$R = \sqrt{\frac{1}{R_h R_b R_o} (.53 + 4.7 R_z)} = \sqrt{\frac{1}{.05 (.069) .239 (.199) (.53 + 4.7 (.065))}} = 0.191$$

Where:

$$V_z = \bar{V} \left(\frac{z}{33} \right)^{.5} \left(\frac{88}{60} \right)^{.5} V$$

$$= 0.45 \left(\frac{60}{33} \right)^{.5} \left(\frac{88}{60} \right)^{.5} 125 = 95.8$$

$$N_1 = \frac{n L_z}{V_z} = \frac{.75 (390.6)}{95.8} = 3.06$$

$$R_h = \frac{7.47 N_1}{(1 + 0.3 N_1)^{.53}} = \frac{7.47 (3.06)}{(1 + 0.3 (3.06))^{.53}} = .069$$

$$R_b = 4.6 n h / V_z = 4.6 (.75) 100 / 95.8 = 3.6 \rightarrow R_b = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .239$$

$$R_o = 4.6 n h / V_z = 4.6 (.75) 124 / 95.8 = 4.66 \rightarrow R_o = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .199$$

$$\eta_L = \frac{15.4 n L}{V_z} = \frac{15.4 (.75) 124}{95.8} = 14.95 \rightarrow R_L = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .065$$

$$G_F = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_R^2 Q^2 + g_o^2 R^2}}{1 + 1.7 g_o I_o} \right) = 0.925 \left(\frac{1 + 1.7 (.272) \sqrt{3.4^2 (.80)^2 + 4.12^2 (.191)^2}}{1 + 1.7 (.34) .272} \right) = 0.85$$

Effective Wind Pressures

<u>feet</u>	<u>Pressure</u>
0-15	32.9 psf
15-20	32.9 psf
20-25	34.4 psf
25-30	35.5 psf
30-40	36.5 psf
40-50	38.2 psf
50-60	39.6 psf
60-70	40.8 psf
70-80	41.8 psf
80-90	43.0 psf
90-100	42.9 psf

Base Shear

$$\begin{aligned}
 V_L &= B(P_s(15') + 5'(P_{20} + P_{25} + P_{30}) + 10'(P_{40} + P_{50} + P_{60} + P_{70} + P_{80} + P_{90} + P_{100})) \\
 &= 124'-0" (32.9 \text{ psf}(15') + 5'(32.9 \text{ psf} + 34.4 \text{ psf} + 35.5 \text{ psf}) + 10'(36.5 \text{ psf} + 38.2 \text{ psf} + 39.6 \text{ psf} + 40.8 \text{ psf} + 41.8 \text{ psf} + 43.0 \text{ psf}) \\
 &\quad + 10'(42.9 \text{ psf})) \\
 &= 477k < V_{Fa} \rightarrow \text{Wind Controlled @ MF}
 \end{aligned}$$

RAM Structural System

Bentley DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

06/30/21 11:56:27

Loads and Applied Forces

LOAD CASE: EQ ELF

Seismic ASCE 7-16 Equivalent Lateral Force

Importance Factor: 1.50 TL: 6.00 s

Site Class E: Soft Clay Soil

Ss: 0.245 g S1: 0.055 g

Use Specified: SDs: 0.392 g SD1: 0.154 g

Use Specified Seismic Design Category: D

Provisions for: Force

Ground Level: Base

Dir	Eccent	R	Ta Equation			Building Period-T			
X	+ And -	8.0	Std,Ct=0.028,x=0.80			User Defined			
Y	+ And -	6.0	Std,Ct=0.020,x=0.75			User Defined			
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
X	1.118	1.592	1.150	1.150	0.073	0.025	0.026	0.026	1.325
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
Y	0.634	1.592	0.630	0.630	0.098	0.061	0.026	0.061	1.065

Total Building Weight (kips) = 11286.99

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X+E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	100.35	75.39	0.00	62.23	-55.23
Level 6	1	86.10	64.62	0.00	62.23	-55.23
Level 5	1	73.43	52.33	0.00	62.23	-55.23
Level 4	1	60.76	40.72	0.00	62.23	-55.23
Level 3	1	48.09	29.87	0.00	62.23	-55.23
Level 2	1	35.42	19.92	0.00	62.23	-55.23
Level 1	1	19.75	9.19	0.00	62.23	-55.23

APPLIED STORY FORCES

Type: EQ_ASCE716_X+E_F

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	100.35	75.39	0.00
Level 6	86.10	64.62	0.00
Level 5	73.43	52.33	0.00
Level 4	60.76	40.72	0.00
Level 3	48.09	29.87	0.00

RAM Structural System

Loads and Applied Forces

RAM Frame 17.00.01.09

Page 2/11

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

06/30/21 11:56:27

Level 2	35.42	19.92	0.00
Level 1	19.75	9.19	0.00
		<hr/> 292.02	<hr/> 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	100.35	75.39	0.00	62.23	-67.82
Level 6	1	86.10	64.62	0.00	62.23	-67.82
Level 5	1	73.43	52.33	0.00	62.23	-67.82
Level 4	1	60.76	40.72	0.00	62.23	-67.82
Level 3	1	48.09	29.87	0.00	62.23	-67.82
Level 2	1	35.42	19.92	0.00	62.23	-67.82
Level 1	1	19.75	9.19	0.00	62.23	-67.82

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	75.39	0.00
Level 6	86.10	64.62	0.00
Level 5	73.43	52.33	0.00
Level 4	60.76	40.72	0.00
Level 3	48.09	29.87	0.00
Level 2	35.42	19.92	0.00
Level 1	19.75	9.19	0.00
		<hr/> 292.02	<hr/> 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	100.35	0.00	161.45	68.57	-61.53
Level 6	1	86.10	0.00	144.01	68.57	-61.53
Level 5	1	73.43	0.00	121.56	68.57	-61.53
Level 4	1	60.76	0.00	99.35	68.57	-61.53
Level 3	1	48.09	0.00	77.45	68.57	-61.53
Level 2	1	35.42	0.00	55.92	68.57	-61.53
Level 1	1	19.75	0.00	30.02	68.57	-61.53

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Page 3/11

06/30/21 11:56:27

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	161.45
Level 6	86.10	0.00	144.01
Level 5	73.43	0.00	121.56
Level 4	60.76	0.00	99.35
Level 3	48.09	0.00	77.45
Level 2	35.42	0.00	55.92
Level 1	19.75	0.00	30.02
		0.00	689.76

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	161.45	55.89	-61.53
Level 6	1	86.10	0.00	144.01	55.89	-61.53
Level 5	1	73.43	0.00	121.56	55.89	-61.53
Level 4	1	60.76	0.00	99.35	55.89	-61.53
Level 3	1	48.09	0.00	77.45	55.89	-61.53
Level 2	1	35.42	0.00	55.92	55.89	-61.53
Level 1	1	19.75	0.00	30.02	55.89	-61.53

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	161.45
Level 6	86.10	0.00	144.01
Level 5	73.43	0.00	121.56
Level 4	60.76	0.00	99.35
Level 3	48.09	0.00	77.45
Level 2	35.42	0.00	55.92
Level 1	19.75	0.00	30.02
		0.00	689.76

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RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Page 4/11

06/30/21 11:56:27

LOAD CASE: Wind

ASCE 7-16

Exposure: B

Basic Wind Speed (mph): 137.0

Apply Directionality Factor, $K_d = 0.85$ Use Topography Factor, $K_{zt} = 1.00$ Ground Elevation Factor, $K_e = 1.00$

Use Natural Frequency for X-Direction (Hz) = 0.55

Use Natural Frequency for Y-Direction (Hz) = 0.75

Gust Factor for Flexible Structures, G: Use Calculated G for X-Dir.

Gust Factor for Flexible Structures, G: Use Calculated G for Y-Dir.

Damping Ratio for Flexible Structures = 0.05

Mean Roof Height (ft): Top Story Height = 100.35

Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 0.550

Structure is Flexible

Y-Direction: Natural Frequency = 0.750

Structure is Flexible

CpWindward = 0.80 qLeeward (qh) = 40.40 psf

GCpn (Parapet): Windward = 1.50

Leeward = -1.00

Height ft	Kz	Kzt	qz psf	Gust Factor G		CpLeeward		Pressure (psf)	
				X	Y	X	Y	X	Y
100.35	0.989	1.000	40.401	0.872	0.851	-0.498	-0.500	45.740	44.719
86.10	0.947	1.000	38.671	0.872	0.851	-0.498	-0.500	44.534	43.541
73.43	0.905	1.000	36.952	0.872	0.851	-0.498	-0.500	43.334	42.370
60.76	0.857	1.000	35.005	0.872	0.851	-0.498	-0.500	41.976	41.044
48.09	0.802	1.000	32.743	0.872	0.851	-0.498	-0.500	40.398	39.503
35.42	0.735	1.000	30.004	0.872	0.851	-0.498	-0.500	38.487	37.637
19.75	0.622	1.000	25.392	0.872	0.851	-0.498	-0.500	35.270	34.495
0.00	0.575	1.000	23.472	0.872	0.851	-0.498	-0.500	33.931	33.188

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_X

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	40.65	0.00	62.25	-61.75
Level 6	1	86.10	75.47	0.00	62.25	-61.75
Level 5	1	73.43	69.05	0.00	62.25	-61.75
Level 4	1	60.76	66.86	0.00	62.25	-61.75
Level 3	1	48.09	64.32	0.00	62.25	-61.75
Level 2	1	35.42	68.08	0.00	62.25	-61.75
Level 1	1	19.75	79.10	0.00	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_1_X

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

RAM Structural System

Loads and Applied Forces

RAM Frame 17.00.01.09

Page 5/11

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

06/30/21 11:56:27

	ft	kips	kips
Roof	100.35	40.65	0.00
Level 6	86.10	75.47	0.00
Level 5	73.43	69.05	0.00
Level 4	60.76	66.86	0.00
Level 3	48.09	64.32	0.00
Level 2	35.42	68.08	0.00
Level 1	19.75	79.10	0.00
		<hr/> 463.53	<hr/> 0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	40.06	62.25	-61.75
Level 6	1	86.10	0.00	74.37	62.25	-61.75
Level 5	1	73.43	0.00	68.05	62.25	-61.75
Level 4	1	60.76	0.00	65.90	62.25	-61.75
Level 3	1	48.09	0.00	63.39	62.25	-61.75
Level 2	1	35.42	0.00	67.10	62.25	-61.75
Level 1	1	19.75	0.00	77.98	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_1_Y

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	40.06
Level 6	86.10	0.00	74.37
Level 5	73.43	0.00	68.05
Level 4	60.76	0.00	65.90
Level 3	48.09	0.00	63.39
Level 2	35.42	0.00	67.10
Level 1	19.75	0.00	77.98
		<hr/> 0.00	<hr/> 456.85

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_2_X+E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	30.49	0.00	62.25	43.79
Level 6	1	86.10	56.60	0.00	62.25	-43.79

RAM Structural System

Loads and Applied Forces

RAM Frame 17.00.01.09

Page 6/11

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

06/30/21 11:56:27

Level 5	1	73.43	51.78	0.00	62.25	-43.79
Level 4	1	60.76	50.15	0.00	62.25	-43.79
Level 3	1	48.09	48.24	0.00	62.25	-43.79
Level 2	1	35.42	51.06	0.00	62.25	-43.79
Level 1	1	19.75	59.33	0.00	62.25	-43.79

APPLIED STORY FORCES

Type: Wind_ASCE716_2_X+E

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	30.49	0.00
Level 6	86.10	56.60	0.00
Level 5	73.43	51.78	0.00
Level 4	60.76	50.15	0.00
Level 3	48.09	48.24	0.00
Level 2	35.42	51.06	0.00
Level 1	19.75	59.33	0.00
		347.64	0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_2_X-E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	30.49	0.00	62.25	-79.71
Level 6	1	86.10	56.60	0.00	62.25	-79.71
Level 5	1	73.43	51.78	0.00	62.25	-79.71
Level 4	1	60.76	50.15	0.00	62.25	-79.71
Level 3	1	48.09	48.24	0.00	62.25	-79.71
Level 2	1	35.42	51.06	0.00	62.25	-79.71
Level 1	1	19.75	59.33	0.00	62.25	-79.71

APPLIED STORY FORCES

Type: Wind_ASCE716_2_X-E

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	30.49	0.00
Level 6	86.10	56.60	0.00
Level 5	73.43	51.78	0.00
Level 4	60.76	50.15	0.00
Level 3	48.09	48.24	0.00
Level 2	35.42	51.06	0.00
Level 1	19.75	59.33	0.00

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Loads and Applied Forces

Page 7/11

06/30/21 11:56:27

347.64 0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_2_Y+E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	30.04	80.79	-61.75
Level 6	1	86.10	0.00	55.78	80.79	-61.75
Level 5	1	73.43	0.00	51.03	80.79	-61.75
Level 4	1	60.76	0.00	49.42	80.79	-61.75
Level 3	1	48.09	0.00	47.54	80.79	-61.75
Level 2	1	35.42	0.00	50.33	80.79	-61.75
Level 1	1	19.75	0.00	58.48	80.79	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_2_Y+E

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	30.04
Level 6	86.10	0.00	55.78
Level 5	73.43	0.00	51.03
Level 4	60.76	0.00	49.42
Level 3	48.09	0.00	47.54
Level 2	35.42	0.00	50.33
Level 1	19.75	0.00	58.48
		0.00	342.64

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_2_Y-E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	30.04	43.71	-61.75
Level 6	1	86.10	0.00	55.78	43.71	-61.75
Level 5	1	73.43	0.00	51.03	43.71	-61.75
Level 4	1	60.76	0.00	49.42	43.71	-61.75
Level 3	1	48.09	0.00	47.54	43.71	-61.75
Level 2	1	35.42	0.00	50.33	43.71	-61.75
Level 1	1	19.75	0.00	58.48	43.71	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_2_Y-E

Level	Ht	Fx	Fy
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Page 33 of 63

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Page 8/11

06/30/21 11:56:27

	ft	kips	kips
Roof	100.35	0.00	30.04
Level 6	86.10	0.00	55.78
Level 5	73.43	0.00	51.03
Level 4	60.76	0.00	49.42
Level 3	48.09	0.00	47.54
Level 2	35.42	0.00	50.33
Level 1	19.75	0.00	58.48
		0.00	342.64

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_3_X+Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	30.49	30.04	62.25	-61.75
Level 6	1	86.10	56.60	55.78	62.25	-61.75
Level 5	1	73.43	51.78	51.03	62.25	-61.75
Level 4	1	60.76	50.15	49.42	62.25	-61.75
Level 3	1	48.09	48.24	47.54	62.25	-61.75
Level 2	1	35.42	51.06	50.33	62.25	-61.75
Level 1	1	19.75	59.33	58.48	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_3_X+Y

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	30.49	30.04
Level 6	86.10	56.60	55.78
Level 5	73.43	51.78	51.03
Level 4	60.76	50.15	49.42
Level 3	48.09	48.24	47.54
Level 2	35.42	51.06	50.33
Level 1	19.75	59.33	58.48
		347.64	342.64

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_3_X-Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	30.49	-30.04	62.25	-61.75
Level 6	1	86.10	56.60	-55.78	62.25	-61.75

RAM Structural System

Loads and Applied Forces

RAM Frame 17.00.01.09

Page 9/11

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

06/30/21 11:56:27

Level 5	1	73.43	51.78	-51.03	62.25	-61.75
Level 4	1	60.76	50.15	-49.42	62.25	-61.75
Level 3	1	48.09	48.24	-47.54	62.25	-61.75
Level 2	1	35.42	51.06	-50.33	62.25	-61.75
Level 1	1	19.75	59.33	-58.48	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_3_X-Y

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	30.49	-30.04
Level 6	86.10	56.60	-55.78
Level 5	73.43	51.78	-51.03
Level 4	60.76	50.15	-49.42
Level 3	48.09	48.24	-47.54
Level 2	35.42	51.06	-50.33
Level 1	19.75	59.33	-58.48
		347.64	-342.64

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_4_X+Y_CW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	22.86	22.53	43.71	-43.79
Level 6	1	86.10	42.45	41.83	43.71	-43.79
Level 5	1	73.43	38.84	38.28	43.71	-43.79
Level 4	1	60.76	37.61	37.07	43.71	-43.79
Level 3	1	48.09	36.18	35.66	43.71	-43.79
Level 2	1	35.42	38.29	37.74	43.71	-43.79
Level 1	1	19.75	44.50	43.86	43.71	-43.79

APPLIED STORY FORCES

Type: Wind_ASCE716_4_X+Y_CW

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	22.86	22.53
Level 6	86.10	42.45	41.83
Level 5	73.43	38.84	38.28
Level 4	60.76	37.61	37.07
Level 3	48.09	36.18	35.66
Level 2	35.42	38.29	37.74
Level 1	19.75	44.50	43.86

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Loads and Applied ForcesPage 10/11
06/30/21 11:56:27

260.73 256.98

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_4_X+Y_CCW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	22.86	22.53	80.79	-79.71
Level 6	1	86.10	42.45	41.83	80.79	-79.71
Level 5	1	73.43	38.84	38.28	80.79	-79.71
Level 4	1	60.76	37.61	37.07	80.79	-79.71
Level 3	1	48.09	36.18	35.66	80.79	-79.71
Level 2	1	35.42	38.29	37.74	80.79	-79.71
Level 1	1	19.75	44.50	43.86	80.79	-79.71

APPLIED STORY FORCES

Type: Wind_ASCE716_4_X+Y_CCW

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	22.86	22.53
Level 6	86.10	42.45	41.83
Level 5	73.43	38.84	38.28
Level 4	60.76	37.61	37.07
Level 3	48.09	36.18	35.66
Level 2	35.42	38.29	37.74
Level 1	19.75	44.50	43.86

260.73 256.98

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_4_X-Y_CW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	22.86	-22.53	80.79	-43.79
Level 6	1	86.10	42.45	-41.83	80.79	-43.79
Level 5	1	73.43	38.84	-38.28	80.79	-43.79
Level 4	1	60.76	37.61	-37.07	80.79	-43.79
Level 3	1	48.09	36.18	-35.66	80.79	-43.79
Level 2	1	35.42	38.29	-37.74	80.79	-43.79
Level 1	1	19.75	44.50	-43.86	80.79	-43.79

APPLIED STORY FORCES

Type: Wind_ASCE716_4_X-Y_CW

Level	Ht	Fx	Fy
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Page 36 of 63

RAM Structural System

Loads and Applied Forces

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Page 11/11

06/30/21 11:56:27

	ft	kips	kips
Roof	100.35	22.86	-22.53
Level 6	86.10	42.45	-41.83
Level 5	73.43	38.84	-38.28
Level 4	60.76	37.61	-37.07
Level 3	48.09	36.18	-35.66
Level 2	35.42	38.29	-37.74
Level 1	19.75	44.50	-43.86
		<hr/> 260.73	<hr/> -256.98

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_4_X-Y_CCW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	22.86	-22.53	43.71	-79.71
Level 6	1	86.10	42.45	-41.83	43.71	-79.71
Level 5	1	73.43	38.84	-38.28	43.71	-79.71
Level 4	1	60.76	37.61	-37.07	43.71	-79.71
Level 3	1	48.09	36.18	-35.66	43.71	-79.71
Level 2	1	35.42	38.29	-37.74	43.71	-79.71
Level 1	1	19.75	44.50	-43.86	43.71	-79.71

APPLIED STORY FORCES

Type: Wind_ASCE716_4_X-Y_CCW

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	22.86	-22.53
Level 6	86.10	42.45	-41.83
Level 5	73.43	38.84	-38.28
Level 4	60.76	37.61	-37.07
Level 3	48.09	36.18	-35.66
Level 2	35.42	38.29	-37.74
Level 1	19.75	44.50	-43.86
		<hr/> 260.73	<hr/> -256.98

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RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Building Code: IBC

Drift

08/25/21 10:17:20

Steel Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects

Member Force Output: At Face of Joint

P-Delta: Yes Scale Factor: 1.40

Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
W13	Wind DEFL	Wind_ASCE716_1_X
W14	Wind DEFL	Wind_ASCE716_1_Y
W15	Wind DEFL	Wind_ASCE716_2_X+E
W16	Wind DEFL	Wind_ASCE716_2_X-E
W17	Wind DEFL	Wind_ASCE716_2_Y+E
W18	Wind DEFL	Wind_ASCE716_2_Y-E
W19	Wind DEFL	Wind_ASCE716_3_X+Y
W20	Wind DEFL	Wind_ASCE716_3_X-Y
W21	Wind DEFL	Wind_ASCE716_4_X+Y_CW
W22	Wind DEFL	Wind_ASCE716_4_X+Y_CCW
W23	Wind DEFL	Wind_ASCE716_4_X-Y_CW
W24	Wind DEFL	Wind_ASCE716_4_X-Y_CCW
E5	EQ ELF Drift	EQ_ASCE716_X_+E_Drft
E6	EQ ELF Drift	EQ_ASCE716_X_-E_Drft
E7	EQ ELF Drift	EQ_ASCE716_Y_+E_Drft
E8	EQ ELF Drift	EQ_ASCE716_Y_-E_Drft

RESULTS:

Location (ft): (0.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	0.0002	-0.0545	0.0001	-0.0129	0.0000	0.0001
	Lp	0.0001	-0.0478	0.0000	-0.0110	0.0000	0.0001
	W13	1.0547	-0.0006	0.0584	0.0001	0.0003	0.0000
	W14	0.0042	0.4034	0.0003	0.0507	0.0000	0.0003
	W15	1.0185	0.2299	0.0572	0.0136	0.0003	0.0001
	W16	0.5636	-0.2308	0.0304	-0.0135	0.0002	0.0001
	W17	-0.2282	0.0683	-0.0135	0.0242	0.0001	0.0001
	W18	0.2344	0.5368	0.0138	0.0518	0.0001	0.0003
	W19	0.7942	0.3021	0.0440	0.0381	0.0003	0.0002
	W20	0.7879	-0.3030	0.0436	-0.0380	0.0003	0.0002
	W21	0.9397	0.5751	0.0533	0.0491	0.0003	0.0003

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Building Code: IBC

Drift

Page 2/4

08/25/21 10:17:20

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W22	0.2516	-0.1219	0.0127	0.0080	0.0001	0.0000
	W23	0.9350	0.1212	0.0530	-0.0080	0.0003	0.0000
	W24	0.2469	-0.5757	0.0124	-0.0490	0.0001	0.0003
	E5	1.4335	0.1337	0.1298	0.0127	0.0008	0.0001
	E6	1.1775	-0.1255	0.1057	-0.0117	0.0006	0.0001
	E7	-0.2892	0.9207	-0.0260	0.1631	0.0002	0.0010
	E8	0.3158	1.5333	0.0284	0.2181	0.0002	0.0013
Level 6	D	0.0001	-0.0416	0.0000	-0.0114	0.0000	0.0001
	Lp	0.0001	-0.0368	0.0000	-0.0098	0.0000	0.0001
	W13	0.9963	-0.0006	0.0841	0.0001	0.0006	0.0000
	W14	0.0039	0.3527	0.0003	0.0491	0.0000	0.0003
	W15	0.9613	0.2163	0.0817	0.0189	0.0005	0.0001
	W16	0.5332	-0.2173	0.0444	-0.0188	0.0003	0.0001
	W17	-0.2147	0.0441	-0.0187	0.0176	0.0001	0.0001
	W18	0.2206	0.4850	0.0192	0.0560	0.0001	0.0004
	W19	0.7502	0.2640	0.0633	0.0369	0.0004	0.0002
	W20	0.7443	-0.2650	0.0628	-0.0368	0.0004	0.0002
	W21	0.8864	0.5260	0.0757	0.0562	0.0005	0.0004
	W22	0.2389	-0.1299	0.0192	-0.0009	0.0001	0.0000
	W23	0.8820	0.1292	0.0753	0.0010	0.0005	0.0000
	W24	0.2345	-0.5267	0.0189	-0.0561	0.0001	0.0004
	E5	1.3037	0.1210	0.1555	0.0149	0.0010	0.0001
	E6	1.0719	-0.1138	0.1273	-0.0137	0.0008	0.0001
	E7	-0.2632	0.7576	-0.0308	0.1381	0.0002	0.0009
	E8	0.2874	1.3152	0.0337	0.2034	0.0002	0.0013
Level 5	D	0.0001	-0.0302	-0.0000	-0.0093	0.0000	0.0001
	Lp	0.0001	-0.0270	0.0000	-0.0083	0.0000	0.0001
	W13	0.9123	-0.0007	0.1112	0.0001	0.0007	0.0000
	W14	0.0036	0.3036	0.0005	0.0542	0.0000	0.0004
	W15	0.8795	0.1974	0.1079	0.0249	0.0007	0.0002
	W16	0.4889	-0.1984	0.0588	-0.0248	0.0004	0.0002
	W17	-0.1960	0.0264	-0.0246	0.0154	0.0002	0.0001
	W18	0.2014	0.4290	0.0253	0.0659	0.0002	0.0004
	W19	0.6869	0.2272	0.0837	0.0407	0.0006	0.0003
	W20	0.6815	-0.2282	0.0830	-0.0406	0.0005	0.0003
	W21	0.8107	0.4697	0.0999	0.0681	0.0007	0.0004
	W22	0.2197	-0.1290	0.0257	-0.0070	0.0002	0.0000
	W23	0.8066	0.1282	0.0994	0.0071	0.0007	0.0000
	W24	0.2156	-0.4706	0.0251	-0.0680	0.0002	0.0004
	E5	1.1481	0.1061	0.1785	0.0170	0.0012	0.0001
	E6	0.9446	-0.1001	0.1461	-0.0157	0.0010	0.0001
	E7	-0.2324	0.6195	-0.0359	0.1399	0.0002	0.0009
	E8	0.2537	1.1118	0.0392	0.2159	0.0003	0.0014

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Building Code: IBC

Drift

Page 3/4

08/25/21 10:17:20

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
Level 4	D	0.0001	-0.0208	0.0000	-0.0084	0.0000	0.0001
	Lp	0.0001	-0.0188	0.0000	-0.0075	0.0000	0.0000
	W13	0.8011	-0.0008	0.1443	0.0000	0.0009	0.0000
	W14	0.0031	0.2494	0.0006	0.0576	0.0000	0.0004
	W15	0.7716	0.1725	0.1398	0.0319	0.0009	0.0002
	W16	0.4300	-0.1737	0.0767	-0.0319	0.0005	0.0002
	W17	-0.1713	0.0110	-0.0316	0.0108	0.0002	0.0001
	W18	0.1761	0.3630	0.0325	0.0757	0.0002	0.0005
	W19	0.6032	0.1864	0.1087	0.0432	0.0007	0.0003
	W20	0.5985	-0.1876	0.1078	-0.0432	0.0007	0.0003
	W21	0.7107	0.4016	0.1292	0.0807	0.0008	0.0005
	W22	0.1940	-0.1220	0.0338	-0.0159	0.0002	0.0001
	W23	0.7072	0.1211	0.1286	0.0159	0.0008	0.0001
	W24	0.1905	-0.4025	0.0331	-0.0807	0.0002	0.0005
	E5	0.9696	0.0891	0.2080	0.0196	0.0014	0.0001
	E6	0.7984	-0.0843	0.1706	-0.0183	0.0011	0.0001
	E7	-0.1965	0.4796	-0.0420	0.1373	0.0003	0.0009
	E8	0.2145	0.8960	0.0459	0.2263	0.0003	0.0015
Level 3	D	0.0001	-0.0125	0.0000	-0.0056	0.0000	0.0000
	Lp	0.0001	-0.0113	0.0000	-0.0050	0.0000	0.0000
	W13	0.6568	-0.0008	0.1708	-0.0000	0.0011	0.0000
	W14	0.0026	0.1917	0.0007	0.0564	0.0000	0.0004
	W15	0.6318	0.1405	0.1652	0.0376	0.0011	0.0002
	W16	0.3533	-0.1417	0.0910	-0.0376	0.0006	0.0002
	W17	-0.1397	0.0003	-0.0372	0.0041	0.0002	0.0000
	W18	0.1435	0.2873	0.0383	0.0805	0.0003	0.0005
	W19	0.4945	0.1432	0.1286	0.0423	0.0008	0.0003
	W20	0.4907	-0.1444	0.1276	-0.0423	0.0008	0.0003
	W21	0.5815	0.3209	0.1526	0.0885	0.0010	0.0006
	W22	0.1602	-0.1061	0.0403	-0.0251	0.0003	0.0002
	W23	0.5786	0.1052	0.1518	0.0251	0.0010	0.0002
	W24	0.1573	-0.3218	0.0395	-0.0886	0.0003	0.0006
	E5	0.7616	0.0695	0.2220	0.0208	0.0015	0.0001
	E6	0.6278	-0.0661	0.1823	-0.0194	0.0012	0.0001
	E7	-0.1545	0.3423	-0.0453	0.1180	0.0003	0.0008
	E8	0.1686	0.6696	0.0495	0.2140	0.0003	0.0014
Level 2	D	0.0001	-0.0069	0.0000	-0.0063	0.0000	0.0000
	Lp	0.0001	-0.0063	0.0000	-0.0057	0.0000	0.0000
	W13	0.4860	-0.0008	0.2441	-0.0002	0.0013	0.0000
	W14	0.0019	0.1354	0.0010	0.0682	0.0000	0.0004
	W15	0.4666	0.1030	0.2354	0.0528	0.0013	0.0003
	W16	0.2624	-0.1042	0.1308	-0.0531	0.0007	0.0003
	W17	-0.1025	-0.0038	-0.0525	-0.0028	0.0003	0.0000

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Building Code: IBC

Drift

Page 4/4

08/25/21 10:17:20

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W18	0.1053	0.2068	0.0539	0.1050	0.0003	0.0006
	W19	0.3659	0.1009	0.1838	0.0510	0.0010	0.0003
	W20	0.3631	-0.1021	0.1824	-0.0512	0.0010	0.0003
	W21	0.4289	0.2324	0.2170	0.1184	0.0012	0.0006
	W22	0.1199	-0.0810	0.0588	-0.0419	0.0003	0.0002
	W23	0.4268	0.0801	0.2159	0.0417	0.0011	0.0002
	W24	0.1178	-0.2333	0.0577	-0.1186	0.0003	0.0006
	E5	0.5396	0.0487	0.2844	0.0263	0.0015	0.0001
	E6	0.4455	-0.0466	0.2341	-0.0247	0.0012	0.0001
	E7	-0.1091	0.2242	-0.0582	0.1265	0.0003	0.0007
	E8	0.1191	0.4556	0.0635	0.2497	0.0003	0.0013
Level 1	D	0.0000	-0.0006	0.0000	-0.0006	0.0000	0.0000
	Lp	0.0000	-0.0006	0.0000	-0.0006	0.0000	0.0000
	W13	0.2419	-0.0006	0.2419	-0.0006	0.0010	0.0000
	W14	0.0009	0.0672	0.0009	0.0672	0.0000	0.0003
	W15	0.2313	0.0501	0.2313	0.0501	0.0010	0.0002
	W16	0.1316	-0.0511	0.1316	-0.0511	0.0006	0.0002
	W17	-0.0500	-0.0011	-0.0500	-0.0011	0.0002	0.0000
	W18	0.0514	0.1019	0.0514	0.1019	0.0002	0.0004
	W19	0.1821	0.0499	0.1821	0.0499	0.0008	0.0002
	W20	0.1807	-0.0509	0.1807	-0.0509	0.0008	0.0002
	W21	0.2120	0.1140	0.2120	0.1140	0.0009	0.0005
	W22	0.0612	-0.0391	0.0612	-0.0391	0.0003	0.0002
	W23	0.2110	0.0384	0.2110	0.0384	0.0009	0.0002
	W24	0.0601	-0.1147	0.0601	-0.1147	0.0003	0.0005
	E5	0.2552	0.0224	0.2552	0.0224	0.0011	0.0001
	E6	0.2115	-0.0219	0.2115	-0.0219	0.0009	0.0001
	E7	-0.0510	0.0977	-0.0510	0.0977	0.0002	0.0004
	E8	0.0556	0.2059	0.0556	0.2059	0.0002	0.0009

.0015*Cd(5.5) < .01 - Ok

Plate

<u>P₁</u>	<u>P₂</u>	<u>P₃</u>	<u>P₄</u>	<u>P₅</u>
1 = -82.9 psf	1 = 130.2 psf	1 = -177.4 psf	1 = -67.2 psf	1 = -104 psf
2 = 25.2 psf	2 = 25.2 psf	2 = 25.2 psf	2 = 61.9 psf	2 = 61.9 psf

EAf = .781

Reduction Factor (Positive)

Effective Width Area	1	2	3	4	5
10	1.0	1.0	1.0	1.0	1.0
20	1.0	.98	.95	.96	.93
50	1.0	.96	.88	.92	.84
100	1.0	.94	.82	.88	.76
500	1.0	.9	.7	.8	.6

C & C Positive Pressure

Effective Width Area	1	2	3	4	5
10	19.6	19.6	19.6	48.4	48.4
20	19.6	19.3	18.6	46.5	45.0
50	19.6	18.8	17.3	44.4	40.6
100	19.6	18.5	16.1	42.5	36.8
500	19.6	17.7	13.8	38.7	29.0

Plate

<u>P₁</u>	<u>P₂</u>	<u>P₃</u>	<u>P₄</u>	<u>P₅</u>
1 = -82.9 psf	1 = 120.2 psf	1 = -177.4 psf	1 = -67.2 psf	1 = -104 psf
2 = 25.2 psf	2 = 25.2 psf	2 = 25.2 psf	2 = 61.9 psf	2 = 61.9 psf

$EAF = .781$

Reduction Factor (negative)

<u>Effective Wind Area</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>
10	1.0	1.0	1.0	1.0	1.0
20	.96	.96	.96	.95	.95
50	.92	.92	.92	.88	.88
100	.88	.88	.88	.82	.82
500	.8	.8	.8	.7	.7

C&C Negative Pressures

<u>Effective Wind Area</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>
10	-64.7	-101.6	-128.6	-52.4	-81.2
20	-62.1	-97.6	-122	-49.9	-77.1
50	-59.6	-93.5	-117	-46.2	-71.5
100	-57.0	-89.4	-111.8	-43.0	-66.5
500	-51.8	-81.2	-110.9	-36.8	-56.8

BRACED FRAME DESIGN

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1 BRACED FRAME ELEVATION

56.00 1/8" = 1'-0"

2 BRACED FRAME ELEVATION

56.00 1/8" = 1'-0"

RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Building Code: IBC

08/25/21 10:37:23

Steel Code: AISC341-16 - LRFD

Brace Parameters

Story: Level 1 Frame No: 1 Member No: 2
 Fy (ksi): 50.00 Size: HSS8X8X5/8
 Frame Type: Special Concentric Brace Frame -Chevron

F2.5 Bracing Members --- Additional Check Required

F2.3 Required Strength

Expected tension strength ($R_y \times F_y \times A_g$) of brace (kip) = 1148.00Expected comp. str. min ($R_y F_y A_g, (1.0/0.877) F_{cre} A_g$) of brace (kip) = 576.63

Expected post-buckling strength of brace (kip) = 172.99

F2.4a Lateral Force Distribution --- Additional Check Required

F2.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Flange b/t_f = 10.77 Limit = 13.23 OKWeb h/t_w = 10.77 Limit = 13.23 OK

F2.5b(1) Slenderness --- OK

Max kl/r = 89.49 kl/r limit = 200.00 OK

F2.6 Bracing Connections

F2.6c(1) Required Tensile Strength

Strength of the connection shall be the lesser of a or b as follows:

a. Expected ($R_y \times F_y \times A_g$) tensile strength of brace (kip) = 1148.00

b. Max force that the brace can transfer to the system - Not Performed

F2.6c(2) Required Compressive Strength

Required compression strength of brace P_c (kip) = 576.63

F2.6c(3a) Rotation Capacity

Brace buckles in direction of member Minor axis, $1.1R_y M_p$ (kip-ft) = 286.83

The design of the gusset plates shall include consideration of buckling.

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RAM Structural System

Member Code Check

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Building Code: IBC

08/25/21 10:35:38

Steel Code: AISC360-16 LRFD

BRACE INFORMATION:

Story Level = Level 1 Frame Number = 1 Brace Number = 2
 Fy (ksi) = 50.00
 Brace Size = HSS8X8X5/8

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	22.25	22.25
Lu for Bending (ft) _____	22.25	22.25
K _____	1.00	1.00

CONTROLLING BRACE FORCES - SHEAR

Load Combination: 1.400 D

Shear	Top	Vux (kip) _____	-0.00
		Vuy (kip) _____	-0.00
Shear	Bot.	Vux (kip) _____	-0.00
		Vuy (kip) _____	-0.00

SHEAR CHECK:

Vux (kip) = -0.00	0.90Vnx (kip) = 196.31	Vux/0.90Vnx = 0.000
Vuy (kip) = -0.00	0.90Vny (kip) = 196.31	Vuy/0.90Vny = 0.000

CONTROLLING BRACE FORCES - AXIAL

Load Combination: 1.278 D + 0.500 Lp + 1.000 E4

AXIAL CHECK:

Pu (kip) = 344.45	0.90Pnx (kip) = 410.90	Pu/0.90Pnx = 0.838
	0.90Pny (kip) = 410.90	Pu/0.90Pny = 0.838
	0.90Pn (kip) = 410.90	Pu/0.90Pn = 0.838

CONTROLLING BRACE FORCES - FLEXURE

Load Combination: 1.278 D + 0.500 Lp + 1.000 E4

Axial	Load (kip) _____	344.45
Moment	Top	Mux (kip-ft) _____
		Muy (kip-ft) _____
Moment	Bot.	Mux (kip-ft) _____
		Muy (kip-ft) _____

CALCULATED PARAMETERS:

Pu (kip) = 344.45	0.90Pn (kip) = 410.90
Mux (kip-ft) = 0.00	0.90Mnx (kip-ft) = 167.63
Muy (kip-ft) = 0.00	0.90Mny (kip-ft) = 167.63
KL/Rx = 89.49	KL/Ry = 89.49
Cbx = 1.00	

INTERACTION EQUATION:

Pu/0.90*Pn=0.838

Pr/Pc = 0.838

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Bay Size

$L = 10$ ft
 $H_{lower} = 19.25$ ft
 $\theta_{lower} = 27.4509978$ deg.
 $H_{upper,R} = 15.67$ ft
 $\theta_{upper,R} = 57.4555244$ deg.
 $H_{upper,L} = 15.67$ ft
 $\theta_{upper,L} = 57.4555244$ deg.

Members

Beam = W21X201
 Slab Thk. = 5.5 in

Mat'l:

Gusset PL = A36
 Brace = A500-B SQ & RECT
 Rein PL = A572-50
 Beam = A992
 Column = A500-C SQ & RECT

Doubler PL Details:

$t_{PL} = 0.375$ in
 $h = 20$ in

Upper Brace

HSS8X8X5/8

Gusset Details:

$\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 1$ in.
 $t_g = 1.25$ in.
 $L_w = 28$ in.
 Pull-off (R) = 34.8 in.
 Pull-off (L) = 34.8 in.
 Clip = 1 in.
 Over Slot = 2 in.
 $D_{brace} = 7$
 $D_{Beam} = 7$

Brace Reinforcing Details:

$t_{PL} = 0.5$ in.
 $b = 7$ in.
 $L_w = 10$ in.
 $D_{brace} = 7$

Lower Brace

HSS8X8X5/8

Gusset Details:

$\phi_{bm} = 30$ deg
 $\phi_{col} = 30$ deg
 $a = 1$ in.
 $t_g = 1.25$ in.
 $L_w = 28$ in.
 Pull-off = 31.6 in.
 Clip = 1 in.
 Over Slot = 2 in.
 $D_{brace} = 7$
 $D_{Beam} = 7$

Brace Reinforcing Details:

$t_{PL} = 0.5$ in.
 $b = 7$ in.
 $L_w = 10$ in.
 $D_{brace} = 7$

$L_{min} = -22.447$ in

Analysis

Welds $F_{EXX} = 70$ ksi
 Weld Ductility Factor = 1.25
 Gusset K Factor = 0.65
 Drag DF = 0.5
 Doubler PL OK? No

Status

DESIGN OK!

Upper Gusset

Gusset Edge Forces

Left Brace:

$R_v F_y A_g = 1056.16$ k
 $1.14 F_{cre} A_g = 956.4121$ k
 $0.3 F_{cre} A_g = 286.9236$ k
 Right Brace:
 $R_v F_y A_g = 1056.16$ k
 $1.14 F_{cre} A_g = 956.4121$ k
 $0.3 F_{cre} A_g = 286.9236$ k
 $N_u = 84.08488$ k
 $V_u = 1082.671$ k
 $M_u = 12450.72$ in-k
 $N_{u,eq} = 742.1173$ k
 $N_u = 648.4463$ k
 $V_u = 722.5174$ k
 $M_u = 8308.95$ in-k
 $N_{u,eq} = 1087.582$ k

Gusset Geometry

$L_{Left} = 35.43$ in
 $L_{Right} = 35.43$ in
 $LA = 75.68$ in
 $LB = 30.53$ in
 $Lg, max = 24.29$ in

Lower Gusset

Gusset Edge Forces

$R_v F_y A_g = 1056.16$ k
 $1.14 F_{cre} A_g = 855.2663$ k
 $0.3 F_{cre} A_g = 256.5799$ k
 $N_u = 178.2741$ k
 $V_u = 881.1481$ k
 $M_u = 10133.2$ in-k
 $N_{u,eq} = 674.1435$ k
 $N_u = 709.5517$ k
 $V_u = 605.1597$ k
 $M_u = 6959.337$ in-k
 $N_{u,eq} = 1050.108$ k

Gusset Geometry

$L = 43.52$ in
 $LA = 81.74$ in
 $LB = 37.54$ in
 $Lg = 33.06$ in

Limit States

	Spec Eqn	R_n	ϕR_n	$R_n / \phi R_n$
<u>Upper Brace</u>				
Brace Weld	J2-4	1056.2	1091.4	0.97 OK
Net Sect. Rupt.	J4-2	1056.2	1083.1	0.98 OK
Shear Yield	J4-3	1056.2	2514.4	0.42 OK
Shear Rupture	J4-4	1056.2	2207.9	0.48 OK
Reinf. Weld	J2-4	192.5	194.9	0.99 OK
<u>Upper Brace-Gusset</u>				
Tension Yield	J4-1	1056.2	1633.4	0.65 OK
Buckling	E3	1052.1	1476.9	0.71 OK
Block Shear	J4-5	1056.2	1569.0	0.67 OK
<u>Upper Gusset-Beam</u>				
Shear Yield	J4-3	1082.7	2043.5	0.53 OK
Tension Yield	J4-1	742.1	3065.2	0.24 OK
		1087.6	3065.2	0.35 OK
Flexural Yield	F11-1	12450.7	57997.5	0.21 OK
Interaction		0.29	1.0	0.29 OK
		0.20	1.0	0.20 OK
Beam Weld	J2-4	1640.7	1788.6	0.92 OK
		1632.1	2035.7	0.80 OK
WLY	J10-2	742.1	3928.2	0.19 OK
		1087.6	3928.2	0.28 OK
WC	J10-4	742.1	4098.1	0.18 OK
		1087.6	4098.1	0.27 OK
<u>Lower Brace</u>				
Brace Weld	J2-4	1056.2	1091.4	0.97 OK
Net Sect. Rupt.	J4-2	1056.2	1083.1	0.98 OK
Shear Yield	J4-3	1056.2	2514.4	0.42 OK
Shear Rupture	J4-4	1056.2	2207.9	0.48 OK
Reinf. Weld	J2-4	192.5	194.9	0.99 OK
<u>Lower Brace-Gusset</u>				
Tension Yield	J4-1	1056.2	1633.4	0.65 OK
Buckling	E3	940.8	1355.2	0.69 OK
Block Shear	J4-5	1056.2	1569.0	0.67 OK
<u>Lower Gusset-Beam</u>				
Shear Yield	J4-3	881.1	2207.0	0.40 OK
Tension Yield	J4-1	674.1	3310.5	0.20 OK
		1050.1	3310.5	0.32 OK
Flexural Yield	F11-1	10133.2	67651.0	0.15 OK
Interaction		0.18	1.0	0.18 OK
		0.15	1.0	0.15 OK
Beam Weld	J2-4	1386.8	1970.4	0.70 OK
		1515.0	2235.5	0.68 OK
WLY	J10-2	674.1	4203.8	0.16 OK
		1050.1	4203.8	0.25 OK
WC	J10-4	674.1	4362.0	0.15 OK
		1050.1	4362.0	0.24 OK
<u>Beam</u>				
Shear Yield	G2-1	529.9	627.9	0.84 OK
		359.3	627.9	0.57 OK
Doubler Weld	J2-4	4.82	9.86	0.49 OK

Bay Size L = 10 ft H = 19.25 ft $\theta = 27.45099777$ deg. TOF = 1 ft NS Grout = 2 in	Mat'l: Gusset PL = A36 Brace = A500-C SQ & RECT Col = A992 Reinf. PL = A572-50 Base PL = A572-50 Lug = A572-50 $f'_c = 3000$ psi	Gusset Details: $\phi_{col} = 30$ deg $\phi_{bm} = 30$ deg a = 1 in. $t_g = 1.25$ in. $L_w = 28$ in. Pull-off = 30 in. Clip = 0.75 in. Over Slot = 2 in. $D_{brace} = 7$ $D_{column} = 7$ $D_{base\ PL} = 6$ Base PL Details: $D_{column-Base\ PL} = 10$	Anchor Rods $P_{ut-col} = 870$ k $P_{uc-col} = 0$ k Bolts = F1554 Gr 105 Dia. = 2 in $n_{back} = 2$ Bolts Back Offset = 6 in $n_{front} = 6$ Bolts Front Offset = 6 in Analysis $F_{EXX} = 70$ ksi Gusset K Factor = 0.6 Distribution Method = KISS Ductility Factor = 1.25
Members: Column = W14X132 Brace = HSS8X8X5/8 Base PL = 2.5 in (t) x 18 in (w) x 58.57 in (L) Lug = 3 in (t) x 18 in (w) x 11 in (d)	Reinf. Details: $t_{PL} = 0.5$ in. b = 7 in. $L_w = 10$ in. $D_{brace} = 7$		

Limit States

REVISE DESIGN!

Brace	Spec Eqn	R_u	ϕR_n	$R_u / \phi R_n$
Brace Weld	J2-4	1066.0	1091.4	0.98 OK
Net Sec. Rupt.	J4-2	1066.0	1080.9	0.99 OK
Shear Yield	J4-3	1066.0	2537.8	0.42 OK
Shear Rupture	J4-4	1066.0	2178.6	0.49 OK
Reinf. Weld	J2-4	192.5	194.9	0.99 OK
Gusset				
Tension Yield	J4-1	1066.0	1633.4	0.65 OK
Buckling	E3	997.8	1200.1	0.83 OK
Block Shear	J4-3	1066.0	1569.0	0.68 OK
Gusset at Col.				
Column Weld	J2-4	1317.0	1339.0	0.98 OK
Tension Yield	J4-1	464.0	2427.7	0.19 OK
Shear Yield	J4-2	946.0	1618.5	0.58 OK
Gusset at Base PL				
Base PL Weld	J2-4	632.1	726.9	0.87 OK
Tension Yield	J4-1	119.4	1666.6	0.07 OK
Shear Yield	J4-2	491.4	1111.1	0.44 OK
Base PL				
Col Weld	J2-4	1816.0	1841.8	0.99 OK
Concrete Bearing	22.8.3.1	491.4	537.0	0.92 OK
Lug Flexure	F11-1	3194.2	3645.0	0.88 OK
Anchor Rod Tension	J3-1	374.3	220.8	1.70 NG!
Base PL Flexure	F11-1	-1010.6	1265.6	-0.80 OK
	F11-1	3263.5	4118.3	0.79 OK

Connection Forces

$H_{uc} = 0.0$ k	$H_{ub} = 491.4$ k
$V_{uc} = 946.0$ k	$V_{ub} = 0.0$ k
$M_{uc} = 6952.9$ in-k	$M_{ub} = 1228.5$ in-k

Gusset Geometry

L = 59.5 in
LA = 41.9 in
LB = 60.7 in
Lg = 46.0 in

ADDT'L Anchors Rods
Were Included

MOMENT FRAME DESIGN

Example Calc

1 MOMENT FRAME ELEVATION

S6.02 1/8" = 1'-0"

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Building Code: IBC

Member Code Check

08/25/21 10:41:09

Steel Code: AISC360-16 LRFD

BEAM INFORMATION:

Story Level = Level 1 Frame Number = 3 Beam Number = 39
 Fy (ksi) = 50.00
 Beam Size = W30X108

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	32.00	8.00
Lu for Bending (ft) _____	32.00	8.00
K _____	1.00	1.00
Top Flange Continuously Braced _____	No	
Bottom Flange Continuously Braced _____	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.200 D + 0.500 Lp + 1.000 W1

Segment distance (ft) i - end _____ 24.00
 j - end _____ 32.00

SHEAR CHECK:

Vux (kip) = -58.66	1.00Vnx (kip) = 487.23	Vux/1.00Vnx = 0.120
Vuy (kip) = -0.00	0.90Vny (kip) = 430.92	Vuy/0.90Vny = 0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.400 D

Segment distance (ft) i - end _____ 0.00
 j - end _____ 8.00

AXIAL CHECK:

Pu (kip) = 0.00	0.90Pnx (kip) = 1426.50	Pu/0.90Pnx = 0.000
	0.90Pny (kip) = 1426.50	Pu/0.90Pny = 0.000
	0.90Pn (kip) = 1426.50	Pu/0.90Pn = 0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.200 D + 0.500 Lp + 1.000 W1

Segment distance (ft) i - end _____ 24.00
 j - end _____ 32.00

CALCULATED PARAMETERS:

Pu (kip) = 0.00	0.90Pn (kip) = 1426.50
Mux (kip-ft) = -552.29	0.90Mnx (kip-ft) = 1297.50
Muy (kip-ft) = 0.00	0.90Mny (kip-ft) = 164.63
Cbx = 1.43	

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.426

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RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Building Code: IBC

08/25/21 10:42:06

Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Level 1

Frame No: 3

Member No: 39

Fy (ksi): 50.00

Size: W30X108

Frame Type: Special Moment Resisting Frame

E3.4b Stability Bracing of Beams (D1.2b Highly Ductile) --- OK

Max Lu (in) = 96.00 Lu Limit = 107.50 OK

Lateral Bracing Requirements along Beam

Required strength of lateral brace along beam = 13.11 kip

Required stiffness of bracing (A-6-8) = 8737.37 kip / Lb

Lateral Bracing Requirements at Plastic Hinge

Required strength of lateral brace at hinge = 39.32 kip

Required stiffness of bracing (A-6-8) = 8737.37 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 ($Cd=2$)

E3.5a Basic Requirements (D1.1 Highly Ductile) --- OK

Flange b/tf = 6.91 Limit = 7.35 OK

Web h/tw = 49.50 Limit = 59.01 OK

E3.6b Beam-to-Column Connection Requirements

Beam-to-column connection must be capable of sustaining a 0.04 radian interstory drift ratio.

Required flexural strength of connection (kip-ft) = 1153.33 at story drift angle in (1)

All beam-to-column joints to demonstrate conformance with E3.6b(1) as indicated in E3.6b(2)

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RAM Structural System

Member Code Check

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Building Code: IBC

08/25/21 10:49:56

Steel Code: AISC360-16 LRFD

COLUMN INFORMATION:

Story Level = Level 1 Frame Number = 3 Column Number = 1
 Fy (ksi) = 50.00
 Column Size = W14X455

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	19.75	19.75
Lu for Bending (ft) _____	19.75	19.75
K _____	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.200 D + 0.500 Lp - 1.000 W1

Shear	Top	Vux (kip) _____	-45.75
		Vuy (kip) _____	0.23
Shear	Bot.	Vux (kip) _____	-45.75
		Vuy (kip) _____	0.23

SHEAR CHECK:

Vux (kip) = -45.75	1.00Vnx (kip) = 1151.40	Vux/1.00Vnx = 0.040
Vuy (kip) = 0.23	0.90Vny (kip) = 2912.11	Vuy/0.90Vny = 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.200 D + 1.600 Lp

AXIAL CHECK:

Pu (kip) = 517.88	0.90Pnx (kip) = 5585.68	Pu/0.90Pnx = 0.093
	0.90Pny (kip) = 4863.58	Pu/0.90Pny = 0.106
	0.90Pn (kip) = 4863.58	Pu/0.90Pn = 0.106

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.200 D + 0.500 Lp - 1.000 W1

Axial	Load (kip) _____	408.63
Moment	Top	Mux (kip-ft) _____
		Muy (kip-ft) _____
		-2.98
Moment	Bot.	Mux (kip-ft) _____
		Muy (kip-ft) _____
		-621.32
		1.58

CALCULATED PARAMETERS:

Pu (kip) = 408.63	0.90Pnx (kip) = 5585.68
	0.90Pny (kip) = 4863.58
Mux (kip-ft) = -621.32	0.90Mnx (kip-ft) = 3510.00
Muy (kip-ft) = 1.58	0.90Mny (kip-ft) = 1755.00
	Mcx (kip-ft) = 3469.89
KL/Rx = 32.35	KL/Ry = 54.22
Cbx = 2.15	

INTERACTION EQUATION:

Pu/0.90*Pn=0.073

Eq H1-3: 0.122 + 0.007 = 0.129

Eq H1-1b Per H1.3: 0.037 + 0.177 + 0.000 = 0.214

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RAM Structural System

Seismic Provisions Member Code Check

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 4 SDC D (06-30-21)

Building Code: IBC

08/25/21 10:42:06

Steel Code: AISC341-16 - LRFD

Column Parameters

Story: Level 1 Frame No: 3 Member No: 1

Fy (ksi): 50.00 Size: W14X455

Frame Type: Special Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pu (kip) = 581.03 --- Combination: 1.278 D + 0.500 Lp - 3.000 E2

Max Pu/ ϕ Pn = 0.12 OK

Tension: Max Pu (kip) = 83.11 --- Combination: 0.822 D + 3.000 E2

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:**Required tension and compression strength from D1.4a.**Shear in major axis (kip) = 88.07 --- Combination: 1.278 D + 0.500 Lp - 3.000 E2**Shear in minor axis (kip) = 15.35 --- Combination: 1.278 D + 0.500 Lp + 3.000 E4**Moment in major axis (kip-ft) = 1280.49 --- Combination: 1.278 D + 0.500 Lp - 3.000 E2**Moment in minor axis (kip-ft) = 304.06 --- Combination: 1.278 D + 0.500 Lp + 3.000 E4**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.*

E3.4c(1) Unbraced Connections --- OK

Joint Above Column is Restrained

Joint Below Column is Restrained

E3.5a Basic Requirements --- OK

For calculation of web limit: Pu (kip) = 581.03 ϕ Py (kip) = 6030.00

Basic Requirements (D1.1 Highly Ductile)

Flange b/tf = 2.62 Limit = 7.35 OK

Web h/tw = 5.63 Limit = 53.10 OK

E3.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

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STL BM W30x108

$$\begin{aligned} A &= 31.7 \text{ in}^2 & d &= 29.8" & t_w &= .545" & b_f &= 10.5" \\ \phi_f &= .76" & k_{des} &= 1.41" & k_{dev} &= 2" & k_1 &= 1\frac{1}{2}" \\ Z_x &= 246 \text{ in}^3 \end{aligned}$$

STL COL W14x500

$$\begin{aligned} A &= 147 \text{ in}^2 & d &= 19.6" & t_w &= 2.19" & b_f &= 17" \\ \phi_f &= 7.5" & k_{des} &= 4.1" & k_{dev} &= 4.81" & k_1 &= 2.31" \\ Z_x &= 1030 \text{ in}^3 \end{aligned}$$

Max Mom @ Hinge Zone

$$M_{pr} = 155 \text{ k-ft} = 18684 \text{ k-in}$$

Max Bolt Dia

$$d_b \leq \frac{b_f}{4} \left(1 - \frac{A_b F_b}{R_t F_u} \right) = \frac{10.5}{4} \left(1 - \frac{1.1(50)}{1.1(65)} \right) = 1.71" \rightarrow \text{use } 7/8"$$

Determine Req'd # of Bolts

$$n = \begin{cases} 1.0 F_u A_b = 1.0 (84 \text{ ksi}) (6.01 \text{ in}^2) \rightarrow 505 \text{ k/bolt} \rightarrow \text{Control} \\ 2.4 F_u b d_{btt} = 2.4 (1.1) (65 \text{ ksi}) (7/8") (7.6") = 1144 \text{ k/bolt} \\ 2.4 F_u p d_{btp} = 2.4 (1.1) (65 \text{ ksi}) (7/8") (1\frac{1}{2} ") = 225 \text{ k/bolt} \end{cases}$$

$$n = \frac{1.25 M_{pr}}{\phi_n (d_{btt})} = \frac{1.25 (18684 \text{ k-in})}{.9 (50.5 \text{ k/bolt}) (29.8" + 1\frac{1}{2} ") = 16.4 \text{ bolts} \rightarrow \text{use } (18) \text{ bolts}$$

Determine Plastic Hinge Zone

$$S_h = S + S (\frac{1}{2} - 1) = 4\frac{1}{2} + 0 (4 - 1) = 28.5'$$

Determine Force in Flange

$$F_{fr} = \frac{18684 \text{ k-in}}{29.8" + 1\frac{1}{2}"} = 597 \text{ kips}$$

$$n = \frac{597 \text{ k}}{.9 (50.5 \text{ k/bolt})} = 13 < 18 \rightarrow \text{OK}$$

Check R Yielding

$$\phi R_n = \phi F_y A_g = .9 (50 \text{ ksi}) 1 \frac{1}{2} (10.5) = 7094 > 5974$$

Check R Rupture

$$\phi R_n = \phi F_u A_n = .9 (65 \text{ ksi}) (1 \frac{1}{2}) (10.5 - 2 (1 \frac{5}{16})) = 7574 > 5974$$

Check BM Block Shear Rupture

$$R_n = \min (.6 F_u A_{nv} + F_u A_{nt}, .6 F_y A_{gv} + F_u A_{nt}) = 14204$$

$$A_{gv} = (2) (28) (.76) = 42.6 \text{ in}^2$$

$$A_{nv} = (2) (.76) (28 - 8 \frac{1}{2} (1 \frac{5}{16})) = 30.5 \text{ in}^2$$

$$A_{nt} = (2) (.76) (1 \frac{1}{2} - \frac{1}{2} (1 \frac{5}{16})) = 1.57 \text{ in}^2$$

$$\phi R_n = .9 (14204) = 12804 > 5974$$

Check R Bending

$$L = 4'$$

$$k = 0.65$$

$$k_y = \frac{.65 (4')}{1.5' / \sqrt{12}} = 6 < 25 \rightarrow \phi R_n = \phi F_y A_g$$

$$\phi R_n = \phi F_y A_g = .9 (50 \text{ ksi}) 1 \frac{1}{2} (11 \frac{1}{2}) = 776 > 5974$$

Check Shear conn.

$$V_n = \frac{2 M_p}{L_h} + V_{gn} = \frac{2 (18684 \text{ k-in})}{31.5'} + 7542 = 1564$$

Check column Panel zone Shear

$$R_n = \frac{(2) M_p}{d_c + (2) t_p} = \frac{(2) 7814 \text{ k-in}}{19.6' + (2) 1 \frac{1}{2}} = 732 \text{ k-in}$$

$$\phi R = \phi (.6) F_y d_c t_w \left(1 + \frac{3 b t_w^2}{d_c d_c t_w} \right) = 1.0 (.6) 50 \text{ ksi} (19.6') 2.19 \left(1 + \frac{3 (0.7') (2.5')^2}{29.5' (19.6') 2.19} \right) = 1920 \text{ k-in} > R_n$$

MF Base conn

$$M_u = 657k-ft$$

top tension on stud

$$T = M_u / D = 657k-ft (12) / 196" = 412k$$

headed stud strength

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} < R_g R_p A_{sc} F_u = 21.5k/sqd$$

$$\# \text{ Bolts req'd} = T_u / Q_n = \frac{412k}{65(21.5k/sqd)} = 30 \text{ studs EA. Flange}$$

W27x94

Plastic Hinge Lower Flange Bracing

$$P_n = .06 F_y F_y \bar{E} / h_o = .06 (2.12) (1.1) 50 ksi \left(\frac{278 in^3}{27} \right) = 72 k$$

$$L = 8'-0"$$

Use L 5x5x1/2

Intermediate Bracing

$$L_b = .095 F_y E / (F_y F_y) = .095 (2.39) (29,000 ksi) / (1.1 (50 ksi)) = 120' = 10'-0"$$

$$P_n = .02 \left(\frac{M_r C_b}{h_o} \right) = .02 \left(\frac{1.1 (50 ksi) 467 in^3 (1.0)}{27.2} \right) = 16 k$$

$$k_{br} = 1/0.75 \left(\frac{10 M_r C_b}{L_b h_o} \right) = 1/0.75 \left(\frac{10 (1.1) 50 ksi (467 in^3)}{(96') 27.2} \right) = 110.8 \text{ ksi}$$

$$\text{Check W14x50} \rightarrow k = \frac{A F_y}{L} = 110 \text{ ksi} > 110.8 \text{ ksi}$$

Check @ End cond.

1st & 2nd Floor Int

Lower col W14x50

$$Z_x = 1050 \text{ in}^3, A_g = 14.7 \text{ in}^2$$

Upper col W14x42

$$Z_x = 869 \text{ in}^3, A_g = 12.5 \text{ in}^2$$

$P_r = 544 \text{ k} \rightarrow \text{From DAM}$

$$M_c = \phi Z_{xc} (F_y - P_r/A_g) = (1050 \text{ in}^3 + 869 \text{ in}^3) (50 \text{ ksi} - 544 \text{ k} / 14.7 \text{ in}^2) = 88850 \text{ k-in}$$

Str DM W20x108

$$Z_x = 346 \text{ in}^3$$

$V_r = 55 \text{ k} \rightarrow \text{RAM}$

$$M_D = \phi (Z_x A_g F_y) + \phi V_r d = 2 (346 \text{ in}^3) (50 \text{ ksi}) (1.1) (1.15) + 55 \text{ k} (25' + 15') = 46000 \text{ k-in} < M_c \rightarrow \text{okay}$$

W 14x50 → V14x42.6

$M_u = 605 \text{ k-ft}$, $V_u = 116 \text{ k}$

Controlling Section Prop

$d = 18.7'$

$b_t = 16.7'$

$t_f = 2.04''$

$t_w = 1.88''$

$$P_{nt} = \frac{b_t b_t}{A_g} P_u = \frac{16.7'' (2.04'')}{175 \text{ in}^2} 431 \text{ k} = 175 \text{ k}$$

$$T_u = M_u/d - P_{nt} = \frac{605 \text{ k-ft} (12)}{18.7'} - 175 = 212 \text{ k}$$

$$\# \text{ of Bolts} = 212 \text{ k} / 24.3 \text{ k/bolt} = 8.78 \rightarrow \text{use (12) } 7/8'' \text{ A325N Bolts}$$

Try 24' R EA. Side

Tension Yield: $\phi F_y A_g = 0.9 (36 \text{ ksi}) 24' (16.5'') = 400 \text{ k}$

Shear Rupture: $\phi F_u A_e = .75 (58 \text{ ksi}) (16.5' - 4') (1.5/16'') 24' = 466 \text{ k}$

Block Shear Rupture:

$$R_n = \min (0.6 F_u A_{nv} + F_u A_{nt}, .6 F_y A_{gv} + F_u A_{nt}) = 420 \text{ k}$$

$$\phi R_n = .75 (420 \text{ k}) = 320 \text{ k} > P_u$$

$$F_u = 58 \text{ ksi}$$

$$F_y = 36 \text{ ksi}$$

$$A_{nv} = (2) 24'' (8' - 2 \cdot \frac{1}{2} (1.5/16'')) = 8.5 \text{ in}^2$$

$$A_{nt} = (2) 24'' (1.94'') = 2.91 \text{ in}^2$$

$$A_{gv} = (2) 24'' (8' - 2'') = 12.0 \text{ in}^2$$

Check Shear

$$V_u = 116 \text{ k}$$

$$\# \text{ of Bolts} = 116 \text{ k} / 24.3 \text{ k/bolt} = 4.8 \rightarrow \text{use (6) Bolts } 7/8'' \text{ A325N Bolts}$$

Check R 24'

Shear Yields: $\phi R_n = \phi .6 F_y A_{gv} = 1.0 (.6) 36 \text{ ksi} (6' - 2') (1.5/16'') 24' = 144 \text{ k}$

Shear Rupture: $\phi R_n = \phi .6 F_u A_{nv} = .75 (.6) 58 \text{ ksi} (6' - 2') (1.5/16'') 24' = 130.7 \text{ k}$

W 14x211 → W14x145

$$M_u = 330 \text{ k-ft}, \quad V_u = 32.7 \text{ k}$$

Continuing Section Prop

$$d = 15.7'$$

$$b_f = 15.5'$$

$$t_f = 1.09'$$

$$t_w = .68'$$

$$P_{nt} = \frac{b_f t_f}{A_g} P_u = \frac{15.5 (1.09)}{62 \text{ in}^2} 132 \text{ k} = 52.5 \text{ k}$$

$$T_u = M_u/d - P_{nt} = \frac{330 \text{ k-ft} (12)}{15.7'} - 52.5 \text{ k} = 200 \text{ k}$$

$$\# \text{ of bolts} = 200 \text{ k} / 24.7 \text{ k/bolt} = 8.2 \rightarrow \text{use (10) } 7/8" \text{ A325N bolts}$$

Tension Yield: $\phi F_y A_g$ EA. side

$$\phi F_y A_g = 0.9 (36 \text{ ksi}) (16.5') = 267 \text{ k}$$

$$\text{Shear Rupture: } \phi F_u A_e = .75 (58 \text{ ksi}) (16.5' - 4') (15/16') = 277 \text{ k}$$

Block Shear Rupture:

$$P_n = \min (0.6 F_u A_{nv} + F_u A_{nt}, .6 F_y A_{gv} + F_u A_{nt}) = 220.5 \text{ k}$$

$$\phi P_n = .75 (220.5 \text{ k}) = 165 \text{ k} < P_u \rightarrow \text{use } 3/4" \text{ R} \rightarrow \phi P_n = 246 \text{ k}$$

$$F_u = 58 \text{ ksi}$$

$$F_y = 36 \text{ ksi}$$

$$A_{nv} = (2) 1/2" (5' - 1 \cdot 1/2 (15/16')) = 7.6 \text{ in}^2$$

$$A_{nt} = (2) 1/2" (1.94') = 1.94 \text{ in}^2$$

$$A_{gv} = (2) 1/2" (5') = 5.0 \text{ in}^2$$

Check Shear

$$V_u = 51.4 \text{ k}$$

$$\# \text{ of bolts} = 51.4 \text{ k} / 24.7 \text{ k/bolt} = 2.1 \rightarrow \text{use (4) bolts } 7/8" \text{ A325N bolts}$$

Check R 3/4"

$$\text{Shear Yields: } \phi P_n = \phi .6 F_y A_{gv} = 1.0 (.6) (36 \text{ ksi}) (6") (2) = 144 \text{ k}$$

$$\text{Shear Rupture: } \phi P_n = \phi .75 F_u A_{nv} = .75 (.6) (58 \text{ ksi}) (6" - 1") (15/16') = 133.7 \text{ k}$$

2018 IBC Risk Category IV Seismic Design Category C

This section presents representative structural calculations for Building 2 (7-story healthcare facility in Long Island, New York) Design Case F. Project objectives, building selection criteria, seismic design criteria, structural design drawings, references, and project participants are documented in NIST GCR 21-917-48v2A.

STRUCTURAL CALCULATIONS

FOR

APPLIED TECHNOLOGY COUNCIL PROJECT #146
HEALTHCARE FACILITY DESIGN F
LONG ISLAND, NEW YORK

PREPARED BY
PCS STRUCTURAL SOLUTIONS

SEPTEMBER 30, 2021
19-051

TABLE OF CONTENTS

DESIGN CRITERIA.....	1
GRAVITY DESIGN.....	10
BEAM DESIGN.....	11
COLUMN DESIGN.....	16
LATERAL DESIGN.....	20
BRACED FRAME DESIGN.....	43
MOMENT FRAME DESIGN.....	52

DESIGN CRITERIA

Project: ATC BLDG 2 RC4 Design F

Job Number: 19-051

Sheet: _____ of _____

Name: JAD

Originating Office: Tacoma

Date: 08/25/21

DESIGN CRITERIA CHECKLIST

CODE: IBC 2018, ASCE 7-16

LOCATION: Long Island, NY

VERTICAL DESIGN CRITERIA

	DEAD	LIVE	PARTITION	CONCENTRATED
ROOF:	100 PSF	40 PSF		
OPERATING ROOMS	95 PSF	60 PSF	15 PSF	1000 #
LIBRARY:	95 PSF	150 PSF	15 PSF	1000 #
PRIVATE ROOMS	95 PSF	100 PSF	15 PSF	
CORRIDORS ABOVE THE 1st FLOOR	95 PSF	80 PSF	15 PSF	

WIND DESIGN CRITERIA

BASIC WIND SPEED (V) = 130 MPH	▼	(Per ASCE 7-16 Sec. 26.5.1, Fig. 26.5-1A; 1B; 1C & 1D, or as required by Bld'g Dept.)
RISK CATEGORY: III & IV	▼	(Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)
EXPOSURE CATEGORY: B	▼	(Per ASCE 7-16 Section 26.7.3)
TOPOGRAPHIC FACTOR (K_{zt}): <u>1.00</u>	▼	(Per ASCE 7-16 Section 26.8.2)
MEAN ROOF HEIGHT: 30 FT	▼	(See ASCE 7-16 Section 26.2 - Definitions)
ELEVATION: <u>0 FT</u>	▼	(See ASCE 7-16 Section 26.9)
ENCLOSURE CLASSIFICATION: Enclosed	▼	(See ASCE 7-16 Section 26.2 & Table 26.13-1)
ROOF TYPE: Monoslope	▼	(See ASCE 7-16 Figure 27.3-1)
ROOF SLOPE (α :12): <u>0.00:12</u>		(Enter vertical rise in 12 horizontal units)
θ (degrees): <u>0.00</u>		

SEISMIC DESIGN CRITERIA

RISK CATEGORY: IV	▼	(Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)
SITE CLASS: D	▼	(Per IBC Section 1613.2.2, Assumed as "D" or per Geotech.)
IMPORTANCE FACTOR (I_E): <u>1.5</u>		(Per ASCE 7-16 Table 1.5-2)
STRUCTURAL SYSTEM (R): <u>3</u>		(Per ASCE 7-16 Table 12.2-1)
OVERSTRENGTH FACTOR (Ω_o): <u>3.0</u>		(Per ASCE 7-16 Table 12.2-1)
INFORMATION BELOW FROM "EARTHQUAKE SPECTRAL RESPONSE ACCELERATION MAPS" PER USGS		
LATITUDE: <u>40.647</u>	$S_s =$ <u>0.245</u>	$F_a =$ <u>1.600</u>
LONGITUDE: <u>-73.564</u>	$S_1 =$ <u>0.055</u>	$F_v =$ <u>2.400</u>

DEFLECTION CRITERIA

FLOOR (LIVE):	L/ 480	▼	ROOF (LIVE):	L/ 360	▼
FLOOR (TOTAL):	L/ 360	▼	ROOF (TOTAL):	L/ 240	▼
WALLS:	L/ 360	▼	SPECIAL:	L/	▼

SOIL DESIGN CRITERIA

REPORT: <u>NO</u>	SEE SOILS REPORT FOR ACTIVE, PASSIVE PRESSURES AND FRICTION COEFFICIENT	
BEARING: <u>6000 PSF</u>	MINIMUM FOOTING DIMENSIONS:	
ACTIVE: <u>35 PCF</u>		
PASSIVE: <u>250 PCF</u>		
COEFFICIENT OF FRICTION: <u>0.35</u>	CONTINUOUS:	<u>1'-4"</u>
PILE TYPE: <u>NONE</u>	SPREAD:	<u>1'-6"</u>
VERTICAL CAPACITY: <u>N/A</u>	FROST DEPTH:	<u>1'-6"</u>
UPLIFT CAPACITY: <u>N/A</u>	LATERAL CAPACITY:	<u>N/A</u>
	SIZE:	<u>N/A</u>

Project: ATC BLDG 2 RC4 Design F

Job Number: 19-051

Sheet: _____ of _____

Name: JAD

Originating Office: Tacoma

Date: 08/25/21

MATERIALS

CONCRETE

Footings/Piles:	3000 PSI	Columns:	4000 PSI
Slabs/Walls:	4000 PSI	Beams:	4000 PSI
-	-	-	-

REINFORCING

Steel Grade = 60 $f_y = 60$ KSI

STRUCTURAL STEEL

W-Flange Beams	ASTM A992	$f_y = 50$ KSI
Shapes & Plates	ASTM A36	$f_y = 36$ KSI
Pipes	ASTM A53, Grade B	$f_y = 35$ KSI
HSS Rect.	ASTM A500, Grade C	$f_y = 50$ KSI
HSS Round	ASTM A500, Grade C	$f_y = 46$ KSI

This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

DESIGN CRITERIA - WIND

FIGURE 27.3-8: Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases per ASCE 7-16

FIGURE 27.3-8 Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases

DESIGN CRITERIA - WIND

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, C_p , for Enclosed and Partially Enclosed Buildings - Walls and Roofs per ASCE 7-16

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, C_p , for Enclosed and Partially Enclosed Buildings—Walls and Roofs

DESIGN CRITERIA - WIND

FIGURE 30.3-1: Components and Cladding [$h \leq 60$ ft]: External Pressure Coefficients, ($G C_p$), for Enclosed and Partially Enclosed Buildings - Walls

Diagram

Notation

a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

Exception: For buildings with $\theta = 0^\circ$ to 7° and a least horizontal dimension greater than 300 ft (90 m), dimension a shall be limited to a maximum of $0.8h$.

h = Mean roof height, in ft (m), except that eave height shall be used for $\theta \leq 10^\circ$.

θ = Angle of plane of roof from horizontal, in degrees.

External Pressure Coefficient, ($G C_p$) - Walls

Notes

1. Vertical scale denotes ($G C_p$) to be used with q_h .
2. Horizontal scale denotes effective wind area, in $ft^2 (m^2)$.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of ($G C_p$) for walls shall be reduced by 10% when $\theta \leq 10^\circ$.

FIGURE 30.3-1 Components and Cladding [$h \leq 60$ ft ($h \leq 18.3$ m)]: External Pressure Coefficients, ($G C_p$), for Enclosed and Partially Enclosed Buildings—Walls

DESIGN CRITERIA - SEISMIC

ASCE 7-16 SECTION 12.8 - EQUIVALENT LATERAL FORCE PROCEDURE

RISK CATEGORY:	IV	LATITUDE:	40.647
SITE CLASS:	D	LONGITUDE:	-73.564
IMPORTANCE FACTOR (I_E):	1.5	$S_S =$	0.245
STRUCTURAL SYSTEM (R):	3	$S_1 =$	0.055
OVERSTRENGTH FACTOR (Ω_o):	3	$F_a =$	1.600
		$F_v =$	2.400

ASCE 7-16 SECTION 11.4 SEISMIC GROUND MOTION VALUES

Section 11.4.4 - Coefficients and Risk-Targeted Maximum Considered Earthquake (MCER) Spectral Response Acceleration Parameters

$$S_{MS} = F_a * S_S = 0.392 \quad S_{M1} = F_v * S_1 = 0.132$$

Section 11.4.5 - Design Spectral Response Acceleration Parameters

$$S_{DS} = 2/3 * S_{MS} = 0.261 \quad S_{D1} = 2/3 * S_{M1} = 0.088$$

ASCE 7-16 SECTION 11.6 - SEISMIC DESIGN CATEGORY - SECTION 12.8.2 - PERIOD DETERMINATION

ASCE 7-16 TABLE 11.6-1			
SEISMIC DESIGN CATEGORY BASED ON S_{DS}			
	RISK CATEGORY:		
	I & II	III	IV
$< 0.167g$	A	A	A
$< 0.33g$	B	B	C
$< 0.50g$	C	C	D
$\geq 0.50g$	D	D	D
	C		

ASCE 7-16 TABLE 11.6-2			
SEISMIC DESIGN CATEGORY BASED ON S_{D1}			
	RISK CATEGORY:		
	I & II	III	IV
$< 0.067g$	A	A	A
$< 0.133g$	B	B	C
$< 0.20g$	C	C	D
$\geq 0.20g$	D	D	D
	C		

Each building and structure shall be assigned to the most severe Seismic Design Category in accordance with Table 11.6-1 or Table 11.6-2, irrespective of the fundamental period of vibration of the structure.

PERIOD DETERMINATION:

$$C_t = 0.028$$

$$h_n = 100 \text{ FT}$$

$$x = 0.8$$

$$T_a = C_t * h_n^x = 1.115$$

ASCE 7-16 SECTION 12.8.1.1 - SEISMIC RESPONSE COEFFICIENT

$$\text{GENERAL EQUATION: } C_S = S_{DS}/(R/I) = 0.131 \quad \text{EQ. 12.8-2}$$

$$\text{MAXIMUM: } C_S = S_{D1}/(T^*(R/I)) = 0.039 \quad \text{<--CONTROLS EQ. 12.8-3}$$

$$\text{MINIMUM: } C_S = 0.044 * S_{DS} * I > 0.01 = 0.017 \quad \text{EQ. 12.8-5}$$

$$\text{For structures located where } S_1 > 0.6g$$

$$C_S = 0.5 * S_1/(R/I) = 0.000 \quad \text{EQ. 12.8-6}$$

ASCE 7-10 SECTION 12.8.1 - SEISMIC BASE SHEAR

$$V = C_S * W = 0.039 * W$$

W = the total dead load and applicable portion of other loads as indicated in Section 12.7.2

Search Information

Coordinates: 40.647053, -73.5641258
Elevation: 42 ft
Timestamp: 2021-06-29T20:48:41.993Z
Hazard Type: Seismic
Reference Document: ASCE7-16
Risk Category: IV
Site Class: D-default

MCE_R Horizontal Response Spectrum

Design Horizontal Response Spectrum

Basic Parameters

Name	Value	Description
S _S	0.245	MCE _R ground motion (period=0.2s)
S ₁	0.055	MCE _R ground motion (period=1.0s)
S _{MS}	0.392	Site-modified spectral acceleration value
S _M	0.132	Site-modified spectral acceleration value
S _{DS}	0.261	Numeric seismic design value at 0.2s SA
S _{D1}	0.088	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	C	Seismic design category
F _a	1.6	Site amplification factor at 0.2s
F _v	2.4	Site amplification factor at 1.0s
CR _S	0.943	Coefficient of risk (0.2s)
CR	0.939	Coefficient of risk (1.0s)
PGA	0.146	MCE _G peak ground acceleration
F _{PGA}	1.508	Site amplification factor at PGA
PGA _M	0.22	Site modified peak ground acceleration
T _L	6	Long-period transition period (s)
SsRT	0.245	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.26	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.055	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.059	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)

S1D	0.6	Factored deterministic hazard spectral acceleration (2% probability of exceedance in 50 years)
PGAd	0.5	Factored deterministic acceleration value (PGA)

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

Disclaimer

Hazard loads are provided by the U.S. Geological Survey [Seismic Design Web Services](#).

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Publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

Search Information

Coordinates: 40.647053, -73.5641258
Elevation: 42 ft
Timestamp: 2021-06-29T16:29:22.950Z
Hazard Type: Wind

ASCE 7-16

MRI 10-Year	75 mph
MRI 25-Year	84 mph
MRI 50-Year	93 mph
MRI 100-Year	99 mph
Risk Category I	110 mph
Risk Category II	122 mph
Risk Category III	▲ 131 mph

If the structure under consideration is a healthcare facility and you are also within 1 mile of the coastal mean high water line, you are in a wind-borne debris region. If other occupancy, use the Risk Category II basic wind speed contours to determine if you are in a wind-borne debris region.

Risk Category IV ▲ 137 mph

You are in a wind-borne debris region if you are also within 1 mile of the coastal mean high water line.

ASCE 7-10

MRI 10-Year	77 mph
MRI 25-Year	87 mph
MRI 50-Year	94 mph
MRI 100-Year	102 mph
Risk Category I	114 mph
Risk Category II	125 mph
Risk Category III-IV	▲ 135 mph

If the structure under consideration is a healthcare facility and you are also within 1 mile of the coastal mean high water line, you are in a wind-borne debris region. If other occupancy, use the Risk Category II basic wind speed contours to determine if you are in a wind-borne debris region.

ASCE 7-05

ASCE 7-05 Wind Speed ▲ 115 mph

You are in a wind-borne debris region if you are also within 1 mile of the coastal mean high water line.

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

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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GRAVITY DESIGN

BEAM DESIGN

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (10-20-20)

Building Code: IBC

Floor Map

10/20/20 10:06:28

Steel Code: AISC 360-16 LRFD

Floor Type: Floor

Beam Numbers

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Beam Design

02/02/21 12:25:21

Steel Code: AISC 360-16 LRFD

Floor Type: Floor**Beam Number = 98****SPAN INFORMATION (ft): I-End (47.25,-123.50) J-End (47.25,-91.50)**Beam Size (User Selected) = W21X50 $F_y = 50.0$ ksi

Total Beam Length (ft) = 32.00

COMPOSITE PROPERTIES (Not Shored):

		Left		Right
Deck Label		Deck		Deck
Concrete thickness (in)		4.50		4.50
Unit weight concrete (pcf)		150.00		150.00
f_c (ksi)		5.00		5.00
Decking Orientation		perpendicular		perpendicular
Decking type		VERCO W3 Formlok		VERCO W3 Formlok
beff (in)	=	91.50	Y bar(in)	= 23.01
Mnf (kip-ft)	=	1038.49	Mn (kip-ft)	= 846.60
C (kips)	=	372.16	PNA (in)	= 20.19
Ieff (in4)	=	3120.64	Itr (in4)	= 3986.68
Stud length (in)	=	4.50	Stud diam (in)	= 0.75
Rg Stud Red. Factor	Rg[1] = 1.00		Rg[2] = 0.85	
Rp Stud Red. Factor	Rp[1] = 0.60		Rp[2] = 0.60	
Stud Capacity (kips)	Qn[1] = 17.2		Qn[2] = 14.6	
# of studs:	Max = 64	Partial = 48	Actual = 48	
Number of Stud Rows	= 2	Percent of Full Composite Action	= 50.63	

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.724	0.610	0.610	---	NonR	0.114	0.152
	32.000	0.724	0.610	0.610			0.114	0.152

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 32.45 kips 1.00Vn = 237.12 kips**MOMENTS (Ultimate):**

Span	Cond	LoadCombo	Mu	@	Lb	Cb	Phi	Phi*Mn
			kip-ft	ft	ft			kip-ft
Center	PreCmp+	1.2DL+1.6LL	124.9	16.0	0.0	1.00	0.90	412.50
	Init DL	1.4DL	109.3	16.0	---	---		
	Max +	1.2DL+1.6LL	259.6	16.0	---	---	0.90	761.94
Controlling		1.2DL+1.6LL	259.6	16.0	---	---	0.90	761.94

REACTIONS (kips):

	Left	Right
Initial reaction	12.20	12.20
DL reaction	11.59	11.59
Max +LL reaction	11.59	11.59
Max +total reaction (factored)	32.45	32.45

DEFLECTIONS:

			Ratio
Initial load (in)	at 16.00 ft =	-0.504	L/D = 761
Live load (in)	at 16.00 ft =	-0.189	L/D = 2033 > 360 ^{14 of 58} 0.18
Post Comp load (in)	at 16.00 ft =	-0.219	L/D = 1756 > 240 0.14

RAM Structural System

Gravity Beam Design

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Page 2/2

02/02/21 12:25:21

Steel Code: AISC 360-16 LRFD

Net Total load (in)	at 16.00 ft	=	-0.723	L/D =	531	>	240	0.45
---------------------	-------------	---	--------	-------	-----	---	-----	------

Project ID:
Project # :
Bay ID : 1-Floor: B-C , 5-4
Model : ATC BLDG 2 ASCE RC 3 (10-20-20)

VIBRATION ANALYSIS:

Equipment: User Defined
Evaluation Criterion: Sensitive Equipment
Reference: Murray, T.M., Allen, D.E. and Ungar, E.E.,
"Floor Vibrations Due to Human Activity",
AISC Design Guide #11, 1997

Vibrational Velocity: V= 6000. μ -in/sec (User Defined)
Beams are not continuous at girders
Girders are not continuous at columns

EVALUATION:

Walking Speed (Steps/min)	V (μ -in/sec)	Evaluation V(limit) (μ -in/sec)	Criterion
100	26932	> 6000	Not Satisfied
75	5985	<= 6000	Satisfied
50	1609	<= 6000	Satisfied

PARAMETER SUMMARY:

Parameter	Beam	Left Girder	Right Girder	
Member	W21X50	W24X68	W24X68	
Uniform Load, plf	751.5	1644.9	2646.8	
Effective I, in ⁴	4150.8	6410.2	7206.5	
Frequency, Hz	9.21	8.52	7.12	5.63

FRAMING: (User Modified Bay)

Girder Span = 30.50 ft
Beam Spans:
Left = 0.00 ft
Center = 32.00 ft
Right = 20.33 ft

Girders/Walls:
Left -W24X68
Right -W24X68

Beam -W21X50
4 spaces at 91.50 in

Concrete: dc = 7.50 in
f'c = 5.00 ksi
wt. = 150.0 pcf
Deck Height = 3.00 in

Loading: Dead = 4.00 psf
Live = 11.00 psf
Collateral = 0.00 psf

Warning: The bay configuration and/or properties analyzed here have been modified by the user and differ from the actual configuration and/or properties of the bay in the RAM Structural System.

COLUMN DESIGN

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RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Column Design

02/12/21 11:17:28

Steel Code: AISC360-16 LRFD

Story level Level 2, Column Line B-4, Column # 7

Fy (ksi) = 50.00

Column Size

= W14X176

Orientation (deg.) = 0.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	15.67	15.67
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	10.10	10.35
Bottom _____	10.10	10.35

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	545.17	488.28	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 1435.44	0.90Pnx (kip) = 2189.60	Pu/0.90Pnx = 0.656
	0.90Pny (kip) = 1986.72	Pu/0.90Pny = 0.723
	0.90Pn (kip) = 1986.72	Pu/0.90Pn = 0.723

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 3:

	Dead	Live	Roof
Axial (kip) _____	545.17	488.28	0.00
Moments Top Mx (kip-ft) _____	-1.64	-1.64	0.00
My (kip-ft) _____	0.21	0.21	0.00
Bot Mx (kip-ft) _____	-2.05	-16.72	0.00
My (kip-ft) _____	0.27	5.71	0.00

Reverse curvature about X-Axis

Reverse curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 1435.44	0.90*Pn (kip) = 1986.72
Mux (kip-ft) = -29.21	0.90*Mnx (kip-ft) = 1200.00
Muy (kip-ft) = 9.46	0.90*Mny (kip-ft) = 611.25
Rm = 1.00	
Cbx = 1.86	
Cmx = 0.54	Cmy = 0.57
Pex (kip) = 17322.52	Pey (kip) = 6783.30
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.723

Eq H1-1a: $0.723 + 8/9(0.024 + 0.015) = 0.758$ This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

RAM Structural System

RAM Steel 17.00.01.09

DataBase: ATC BLDG 2 ASCE RC 3 (1-26-21)

Building Code: IBC

Gravity Column Design

Page 2/2

02/12/21 11:17:28

Steel Code: AISC360-16 LRFD

Story level Level 1, Column Line B-4, Column # 7

Fy (ksi) = 50.00

Column Size

= W14X176

Orientation (deg.) = 0.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft) _____	19.75	19.75
K _____	1	1
Braced Against Joint Translation _____	Yes	Yes
Column Eccentricity (in) Top _____	10.10	10.35
Bottom _____	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip) _____	635.24	578.35	0.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL)

Pu (kip) = 1687.64	0.90Pnx (kip) = 2110.42	Pu/0.90Pnx = 0.800
	0.90Pny (kip) = 1808.38	Pu/0.90Pny = 0.933
	0.90Pn (kip) = 1808.38	Pu/0.90Pn = 0.933

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 6:

	Dead	Live	Roof
Axial (kip) _____	635.24	567.03	0.00
Moments Top Mx (kip-ft) _____	-1.63	-1.63	0.00
My (kip-ft) _____	0.21	4.53	0.00
Bot Mx (kip-ft) _____	0.00	0.00	0.00
My (kip-ft) _____	0.00	0.00	0.00

Single curvature about X-Axis

Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL)

Pu (kip) = 1669.53	0.90*Pn (kip) = 1808.38
Mux (kip-ft) = -4.56	0.90*Mnx (kip-ft) = 1200.00
Muy (kip-ft) = 7.50	0.90*Mny (kip-ft) = 611.25
Rm = 1.00	
Cbx = 1.67	
Cmx = 0.60	Cmy = 0.60
Pex (kip) = 10904.73	Pey (kip) = 4270.17
B1x = 1.00	B1y = 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.923

Eq H1-1a: $0.923 + 8/9(0.004 + 0.012) = 0.938$ This publication is available free of charge from: <https://doi.org/10.6028/NIST.GCR.21-917-48v2B>

LATERAL

Seismic

Site Class info

Site Class D $\rightarrow S_{DS} = 0.261$

$V_{ult} = 137 \text{ mph}$

Seismic

$R = 3$

$C_s = \frac{S_{DS}}{R} (I_e) = \frac{0.261}{3} (1.50) = 0.131$

$W = 100 \text{ psf} (15630 \text{ ft}^2) + (95 \text{ psf} + 100 \text{ psf}) (15630 \text{ ft}^2) 6 = 11410 \text{ k}$

$V_{Fa} = C_s W = 0.131 (11410 \text{ k}) = 1495 \text{ k}$

Check min & max C_s

$T = C_t h^x = .028 (100)^{.8} = 1.15 \text{ @ MF}$
 $= .02 (100)^{.75} = 0.67 \text{ @ DF} \quad \left. \vphantom{\begin{matrix} T = C_t h^x \\ = .02 (100)^{.75} \end{matrix}} \right\} < T_L = 6 \text{ sec}$

$\text{Max } C_s = \frac{S_{D1}}{T (C/I_e)} = \frac{0.088}{1.15 (3/1.5)} = .038 \text{ @ MF} \rightarrow V = .038 (W) = 440 \text{ k}$

$= \frac{0.088}{.67 (3/1.5)} = 0.070 \text{ @ DF} \rightarrow V = .070 (W) = 797 \text{ k} \rightarrow \text{Seismic controlled @ DF}$

$\text{Min } C_s = .044 S_{D1} I_e = .044 (.261) 1.50 = .017$

Wind Loads

- Exposure B
- Ultimate Wind Speed = 137 mph
- $I_w = 1.0$

$$q_z = .00256 k_z k_{zt} k_d k_c V^2 = .00256 k_z (.85)(1.0) 129^2 = 36.2 \text{ psf } k_z$$

Where:

k_z = varies

$$k_{zt} = 1.0$$

$$k_d = 0.85$$

$$k_c = 1.0$$

q_z as a function of height

$$\begin{aligned} q_{15} &\rightarrow k_z = .57 \rightarrow q_{15} = 23.2 \text{ psf} \\ q_{20} &\rightarrow k_z = .62 \rightarrow q_{20} = 25.4 \text{ psf} \\ q_{25} &\rightarrow k_z = .66 \rightarrow q_{25} = 27.0 \text{ psf} \\ q_{30} &\rightarrow k_z = .70 \rightarrow q_{30} = 28.5 \text{ psf} \\ q_{40} &\rightarrow k_z = .76 \rightarrow q_{40} = 31.0 \text{ psf} \\ q_{50} &\rightarrow k_z = .81 \rightarrow q_{50} = 33.0 \text{ psf} \\ q_{60} &\rightarrow k_z = .85 \rightarrow q_{60} = 34.7 \text{ psf} \\ q_{70} &\rightarrow k_z = .89 \rightarrow q_{70} = 36.2 \text{ psf} \\ q_{80} &\rightarrow k_z = .93 \rightarrow q_{80} = 37.7 \text{ psf} \\ q_{90} &\rightarrow k_z = .96 \rightarrow q_{90} = 38.0 \text{ psf} \\ q_{100} &\rightarrow k_z = .99 \rightarrow q_{100} \approx q_h = 40.4 \text{ psf} \end{aligned}$$

Wind Pressure

$$P_e = q_z GCP - q_h GCP = G(q_z CP - q_h CP)$$

Where:

G = Gust Factor $\rightarrow$ see next page $\rightarrow$ ASCE 7-16 26.11

CP = 0.8 For windward, -0.5 For Leeward

Gust Factor For MF
Natural Frequency

Left = 124' < 4 times h → use approx Eqn 26.11-2

$$\eta = \frac{22.2}{h^{.8}} = \frac{22.2}{100^{.8}} = .55 < 1 \rightarrow \text{Flexible}$$

Determine Gust Factor

$$I_z = C \left(\frac{z}{z_0} \right)^{.6} = .3 \left(\frac{33}{600} \right)^{.6} = .272$$

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{0.4h}{L_z} \right)^{.63}}} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{124 + 100}{390.6} \right)^{.63}}} = 0.83$$

Where:

$$L_z = 1.6 \left(\frac{z}{33} \right)^{.8} = 320 \left(\frac{60}{33} \right)^{.8} = 390.6'$$

$$g_R = g_V = 3.4$$

$$g_R = \sqrt{2 \ln(3600h)} + \frac{0.577}{\sqrt{2 \ln(3600h)}} = 4.04$$

$$R = \sqrt{\frac{1}{2} R_h R_b R_o (.53 + 4.7 R_o)} = \sqrt{\frac{1}{2} (.083) (.207) (.259) (.53 + 4.7 (.087))} = 0.275$$

Where:

$$V_z = \bar{V} \left(\frac{z}{33} \right)^{.08} \left(\frac{88}{60} \right)^{.1} V$$

$$= 0.45 \left(\frac{60}{33} \right)^{.4} \left(\frac{88}{60} \right)^{.125} = 95.8$$

$$N_1 = \frac{\eta L_z}{V_z} = \frac{.55 (390.6)}{95.8} = 2.24$$

$$R_h = \frac{7.47 N_1}{(1 + 0.3 N_1)^{.53}} = \frac{7.47 (2.24)}{(1 + 0.3 (2.24))^{.53}} = .083$$

$$\eta_h = 4.6 n h / V_z = 4.6 (.55)^{100} / 95.8 = 2.64 \rightarrow R_h = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .207$$

$$\eta_b = 4.6 n h / V_z = 4.6 (.55)^{124} / 95.8 = 3.27 \rightarrow R_b = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .259$$

$$\eta_L = \frac{15.4 n L}{V_z} = \frac{15.4 (.55) 124}{95.8} = 10.96 \rightarrow R_L = \frac{1}{2} \eta^2 (1 - e^{-\eta}) = .087$$

$$G_F = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_R^2 Q^2 + g_o^2 L^2}}{1 + 1.7 g_o L} \right) = 0.925 \left(\frac{1 + 1.7 (.272) \sqrt{3.4^2 (.80)^2 + 4.04^2 (.275)^2}}{1 + 1.7 (3.4) (.272)} \right) = 0.84$$

Gust Factor For BF

Natural Frequency

Left = 124' < 4 times h → use approx Eqn 26.11-2

$$n = \frac{75}{h} = \frac{75}{100} = 0.75 \rightarrow \text{Flexible}$$

Determine Gust Factor

$$I_z = C \left(\frac{z}{z_0} \right)^{1/6} = .3 \left(\frac{33}{600} \right)^{1/6} = .272$$

$$Q = \sqrt{\frac{1}{1 + 0.65 \left(\frac{0.4h}{L_z} \right)^{6.5}}} = \sqrt{\frac{1}{1 + 0.65 \left(\frac{124 + 100}{390.6} \right)^{6.5}}} = 0.83$$

Where:

$$L_z = 1.6 \left(\frac{z}{33} \right)^{.5} = 320 \left(\frac{60}{33} \right)^{.5} = 390.6'$$

$$g_R = g_V = 2.4$$

$$g_R = \sqrt{2 \ln(3600h)} + \frac{0.577}{\sqrt{2 \ln(3600h)}} = 4.12$$

$$R = \sqrt{\frac{1}{R_h R_b R_o} (.53 + 4.7 R_z)} = \sqrt{\frac{1}{.05 (.069) .239 (.199) (.53 + 4.7 (.065))}} = 0.191$$

Where:

$$V_z = \bar{V} \left(\frac{z}{33} \right)^{.5} \left(\frac{88}{60} \right)^{.5} V$$

$$= 0.45 \left(\frac{60}{33} \right)^{.5} \left(\frac{88}{60} \right)^{.5} 125 = 95.8$$

$$N_1 = \frac{n L_z}{V_z} = \frac{.75 (390.6)}{95.8} = 3.06$$

$$R_h = \frac{7.47 N_1}{(1 + 0.3 N_1)^{.53}} = \frac{7.47 (3.06)}{(1 + 0.3 (3.06))^{.53}} = .069$$

$$N_h = 4.6 n h / V_z = 4.6 (.75) 100 / 95.8 = 3.6 \rightarrow R_h = \frac{1}{N_h} \left(1 - e^{-N_h} \right) = .0239$$

$$N_b = 4.6 n h / V_z = 4.6 (.75) 124 / 95.8 = 4.466 \rightarrow R_b = \frac{1}{N_b} \left(1 - e^{-N_b} \right) = .0199$$

$$N_L = \frac{15.4 n L}{V_z} = \frac{15.4 (.75) 124}{95.8} = 14.95 \rightarrow R_L = \frac{1}{N_L} \left(1 - e^{-N_L} \right) = .065$$

$$G_F = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_R^2 Q^2 + g_o^2 R^2}}{1 + 1.7 g_o I_z} \right) = 0.925 \left(\frac{1 + 1.7 (.272) \sqrt{3.4^2 (.80)^2 + 4.12^2 (.191)^2}}{1 + 1.7 (.34) (.272)} \right) = 0.85$$

Effective Wind Pressures

<u>feet</u>	<u>Pressure</u>
0-15	32.9 psf
15-20	32.9 psf
20-25	34.4 psf
25-30	35.5 psf
30-40	36.5 psf
40-50	38.2 psf
50-60	39.6 psf
60-70	40.8 psf
70-80	41.8 psf
80-90	43.0 psf
90-100	42.9 psf

Base Shear

$$\begin{aligned}
 V_L &= B(P_s(15') + 5'(P_{20} + P_{25} + P_{30}) + 10'(P_{40} + P_{50} + P_{60} + P_{70} + P_{80} + P_{90} + P_{100})) \\
 &= 124'-0" (32.9 \text{ psf}(15') + 5'(32.9 \text{ psf} + 34.4 \text{ psf} + 35.5 \text{ psf}) + 10'(36.5 \text{ psf} + 38.2 \text{ psf} + 39.6 \text{ psf} + 40.8 \text{ psf} + 41.8 \text{ psf} + 43.0 \text{ psf}) \\
 &\quad + 10'(42.9 \text{ psf})) \\
 &= 477k < V_{Fa} \rightarrow \text{Wind Controlled @ MF}
 \end{aligned}$$

RAM Structural System

Bentley DataBase: ATC BLDG 2 Design F (06-30-21)

Loads and Applied Forces

08/25/21 13:12:04

LOAD CASE: EQ ELF

Seismic ASCE 7-16 Equivalent Lateral Force

Importance Factor: 1.50 TL: 6.00 s

Site Class D: Stiff Soil, Default

Ss: 0.245 g S1: 0.055 g

Use Specified: SDs: 0.261 g SD1: 0.088 g

Risk Category: IV Seismic Design Category: C

Provisions for: Force

Ground Level: Base

Dir	Eccent	R	Ta Equation			Building Period-T			
X	+ And -	3.0	Std,Ct=0.028,x=0.80			User Defined			
Y	+ And -	3.0	Std,Ct=0.020,x=0.75			User Defined			
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
X	1.118	1.700	1.150	1.150	0.131	0.038	0.017	0.038	1.325
Dir	Ta	Cu	T	T-used	Cs	Cs(max)	Cs(min)	Cs-used	k
					Eq12.8-2	Eq12.8-3	Eq12.8-5		
Y	0.634	1.700	0.630	0.630	0.131	0.070	0.017	0.070	1.065

Total Building Weight (kips) = 11286.99

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	111.48	0.00	62.23	-55.23
Level 6	1	86.10	95.56	0.00	62.23	-55.23
Level 5	1	73.43	77.39	0.00	62.23	-55.23
Level 4	1	60.76	60.21	0.00	62.23	-55.23
Level 3	1	48.09	44.17	0.00	62.23	-55.23
Level 2	1	35.42	29.45	0.00	62.23	-55.23
Level 1	1	19.75	13.58	0.00	62.23	-55.23

APPLIED STORY FORCES

Type: EQ_ASCE716_X+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	111.48	0.00
Level 6	86.10	95.56	0.00
Level 5	73.43	77.39	0.00
Level 4	60.76	60.21	0.00
Level 3	48.09	44.17	0.00

RAM Structural System

Bentley DataBase: ATC BLDG 2 Design F (06-30-21)

Loads and Applied Forces

Page 2/11

08/25/21 13:12:04

Level 2	35.42	29.45	0.00
Level 1	19.75	13.58	0.00
		<hr/> 431.85	<hr/> 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	100.35	111.48	0.00	62.23	-67.82
Level 6	1	86.10	95.56	0.00	62.23	-67.82
Level 5	1	73.43	77.39	0.00	62.23	-67.82
Level 4	1	60.76	60.21	0.00	62.23	-67.82
Level 3	1	48.09	44.17	0.00	62.23	-67.82
Level 2	1	35.42	29.45	0.00	62.23	-67.82
Level 1	1	19.75	13.58	0.00	62.23	-67.82

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	111.48	0.00
Level 6	86.10	95.56	0.00
Level 5	73.43	77.39	0.00
Level 4	60.76	60.21	0.00
Level 3	48.09	44.17	0.00
Level 2	35.42	29.45	0.00
Level 1	19.75	13.58	0.00
		<hr/> 431.85	<hr/> 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	100.35	0.00	184.52	68.57	-61.53
Level 6	1	86.10	0.00	164.59	68.57	-61.53
Level 5	1	73.43	0.00	138.92	68.57	-61.53
Level 4	1	60.76	0.00	113.54	68.57	-61.53
Level 3	1	48.09	0.00	88.51	68.57	-61.53
Level 2	1	35.42	0.00	63.91	68.57	-61.53
Level 1	1	19.75	0.00	34.31	68.57	-61.53

RAM Structural System

DataBase: ATC BLDG 2 Design F (06-30-21)

Loads and Applied Forces

Page 3/11

08/25/21 13:12:04

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	184.52
Level 6	86.10	0.00	164.59
Level 5	73.43	0.00	138.92
Level 4	60.76	0.00	113.54
Level 3	48.09	0.00	88.51
Level 2	35.42	0.00	63.91
Level 1	19.75	0.00	34.31
		0.00	788.30

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	184.52	55.89	-61.53
Level 6	1	86.10	0.00	164.59	55.89	-61.53
Level 5	1	73.43	0.00	138.92	55.89	-61.53
Level 4	1	60.76	0.00	113.54	55.89	-61.53
Level 3	1	48.09	0.00	88.51	55.89	-61.53
Level 2	1	35.42	0.00	63.91	55.89	-61.53
Level 1	1	19.75	0.00	34.31	55.89	-61.53

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	184.52
Level 6	86.10	0.00	164.59
Level 5	73.43	0.00	138.92
Level 4	60.76	0.00	113.54
Level 3	48.09	0.00	88.51
Level 2	35.42	0.00	63.91
Level 1	19.75	0.00	34.31
		0.00	788.30

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RAM Structural System

Bentley DataBase: ATC BLDG 2 Design F (06-30-21)

Loads and Applied Forces

Page 4/11

08/25/21 13:12:04

LOAD CASE: Wind

ASCE 7-16

Exposure: B

Basic Wind Speed (mph): 137.0

Apply Directionality Factor, $K_d = 0.85$ Use Topography Factor, $K_{zt} = 1.00$ Ground Elevation Factor, $K_e = 1.00$

Use Natural Frequency for X-Direction (Hz) = 0.55

Use Natural Frequency for Y-Direction (Hz) = 0.75

Gust Factor for Flexible Structures, G: Use Calculated G for X-Dir.

Gust Factor for Flexible Structures, G: Use Calculated G for Y-Dir.

Damping Ratio for Flexible Structures = 0.05

Mean Roof Height (ft): Top Story Height = 100.35

Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 0.550

Structure is Flexible

Y-Direction: Natural Frequency = 0.750

Structure is Flexible

CpWindward = 0.80 qLeeward (qh) = 40.40 psf

GCpn (Parapet): Windward = 1.50

Leeward = -1.00

Height ft	Kz	Kzt	qz psf	Gust Factor G		CpLeeward		Pressure (psf)	
				X	Y	X	Y	X	Y
100.35	0.989	1.000	40.401	0.872	0.851	-0.498	-0.500	45.740	44.719
86.10	0.947	1.000	38.671	0.872	0.851	-0.498	-0.500	44.534	43.541
73.43	0.905	1.000	36.952	0.872	0.851	-0.498	-0.500	43.334	42.370
60.76	0.857	1.000	35.005	0.872	0.851	-0.498	-0.500	41.976	41.044
48.09	0.802	1.000	32.743	0.872	0.851	-0.498	-0.500	40.398	39.503
35.42	0.735	1.000	30.004	0.872	0.851	-0.498	-0.500	38.487	37.637
19.75	0.622	1.000	25.392	0.872	0.851	-0.498	-0.500	35.270	34.495
0.00	0.575	1.000	23.472	0.872	0.851	-0.498	-0.500	33.931	33.188

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_X

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	40.65	0.00	62.25	-61.75
Level 6	1	86.10	75.47	0.00	62.25	-61.75
Level 5	1	73.43	69.05	0.00	62.25	-61.75
Level 4	1	60.76	66.86	0.00	62.25	-61.75
Level 3	1	48.09	64.32	0.00	62.25	-61.75
Level 2	1	35.42	68.08	0.00	62.25	-61.75
Level 1	1	19.75	79.10	0.00	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_1_X

Level	Ht	Fx	Fy
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30 of 58

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Page 5/11

08/25/21 13:12:04

Loads and Applied Forces

	ft	kips	kips
Roof	100.35	40.65	0.00
Level 6	86.10	75.47	0.00
Level 5	73.43	69.05	0.00
Level 4	60.76	66.86	0.00
Level 3	48.09	64.32	0.00
Level 2	35.42	68.08	0.00
Level 1	19.75	79.10	0.00
		<hr/> 463.53	<hr/> 0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_1_Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	40.06	62.25	-61.75
Level 6	1	86.10	0.00	74.37	62.25	-61.75
Level 5	1	73.43	0.00	68.05	62.25	-61.75
Level 4	1	60.76	0.00	65.90	62.25	-61.75
Level 3	1	48.09	0.00	63.39	62.25	-61.75
Level 2	1	35.42	0.00	67.10	62.25	-61.75
Level 1	1	19.75	0.00	77.98	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_1_Y

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	40.06
Level 6	86.10	0.00	74.37
Level 5	73.43	0.00	68.05
Level 4	60.76	0.00	65.90
Level 3	48.09	0.00	63.39
Level 2	35.42	0.00	67.10
Level 1	19.75	0.00	77.98
		<hr/> 0.00	<hr/> 456.85

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_2_X+E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	30.49	0.00	62.25	43.79
Level 6	1	86.10	56.60	0.00	62.25	-43.79

31 of 58

RAM Structural System

Loads and Applied Forces

RAM Frame 17.00.01.09

Page 6/11

DataBase: ATC BLDG 2 Design F (06-30-21)

08/25/21 13:12:04

Level 5	1	73.43	51.78	0.00	62.25	-43.79
Level 4	1	60.76	50.15	0.00	62.25	-43.79
Level 3	1	48.09	48.24	0.00	62.25	-43.79
Level 2	1	35.42	51.06	0.00	62.25	-43.79
Level 1	1	19.75	59.33	0.00	62.25	-43.79

APPLIED STORY FORCES

Type: Wind_ASCE716_2_X+E

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	30.49	0.00
Level 6	86.10	56.60	0.00
Level 5	73.43	51.78	0.00
Level 4	60.76	50.15	0.00
Level 3	48.09	48.24	0.00
Level 2	35.42	51.06	0.00
Level 1	19.75	59.33	0.00
		347.64	0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_2_X-E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	30.49	0.00	62.25	-79.71
Level 6	1	86.10	56.60	0.00	62.25	-79.71
Level 5	1	73.43	51.78	0.00	62.25	-79.71
Level 4	1	60.76	50.15	0.00	62.25	-79.71
Level 3	1	48.09	48.24	0.00	62.25	-79.71
Level 2	1	35.42	51.06	0.00	62.25	-79.71
Level 1	1	19.75	59.33	0.00	62.25	-79.71

APPLIED STORY FORCES

Type: Wind_ASCE716_2_X-E

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	30.49	0.00
Level 6	86.10	56.60	0.00
Level 5	73.43	51.78	0.00
Level 4	60.76	50.15	0.00
Level 3	48.09	48.24	0.00
Level 2	35.42	51.06	0.00
Level 1	19.75	59.33	0.00

RAM Structural System

Bentley

Loads and Applied Forces

RAM Frame 17.00.01.09

Page 7/11

DataBase: ATC BLDG 2 Design F (06-30-21)

08/25/21 13:12:04

347.64 0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_2_Y+E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	30.04	80.79	-61.75
Level 6	1	86.10	0.00	55.78	80.79	-61.75
Level 5	1	73.43	0.00	51.03	80.79	-61.75
Level 4	1	60.76	0.00	49.42	80.79	-61.75
Level 3	1	48.09	0.00	47.54	80.79	-61.75
Level 2	1	35.42	0.00	50.33	80.79	-61.75
Level 1	1	19.75	0.00	58.48	80.79	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_2_Y+E

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	0.00	30.04
Level 6	86.10	0.00	55.78
Level 5	73.43	0.00	51.03
Level 4	60.76	0.00	49.42
Level 3	48.09	0.00	47.54
Level 2	35.42	0.00	50.33
Level 1	19.75	0.00	58.48
		0.00	342.64

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_2_Y-E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	0.00	30.04	43.71	-61.75
Level 6	1	86.10	0.00	55.78	43.71	-61.75
Level 5	1	73.43	0.00	51.03	43.71	-61.75
Level 4	1	60.76	0.00	49.42	43.71	-61.75
Level 3	1	48.09	0.00	47.54	43.71	-61.75
Level 2	1	35.42	0.00	50.33	43.71	-61.75
Level 1	1	19.75	0.00	58.48	43.71	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_2_Y-E

Level	Ht	Fx	Fy
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33 of 58

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Page 8/11

08/25/21 13:12:04

Loads and Applied Forces

	ft	kips	kips
Roof	100.35	0.00	30.04
Level 6	86.10	0.00	55.78
Level 5	73.43	0.00	51.03
Level 4	60.76	0.00	49.42
Level 3	48.09	0.00	47.54
Level 2	35.42	0.00	50.33
Level 1	19.75	0.00	58.48
		0.00	342.64

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_3_X+Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	30.49	30.04	62.25	-61.75
Level 6	1	86.10	56.60	55.78	62.25	-61.75
Level 5	1	73.43	51.78	51.03	62.25	-61.75
Level 4	1	60.76	50.15	49.42	62.25	-61.75
Level 3	1	48.09	48.24	47.54	62.25	-61.75
Level 2	1	35.42	51.06	50.33	62.25	-61.75
Level 1	1	19.75	59.33	58.48	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_3_X+Y

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	30.49	30.04
Level 6	86.10	56.60	55.78
Level 5	73.43	51.78	51.03
Level 4	60.76	50.15	49.42
Level 3	48.09	48.24	47.54
Level 2	35.42	51.06	50.33
Level 1	19.75	59.33	58.48
		347.64	342.64

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_3_X-Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	30.49	-30.04	62.25	-61.75
Level 6	1	86.10	56.60	-55.78	62.25	-61.75

RAM Structural System

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Loads and Applied Forces

RAM Frame 17.00.01.09

Page 9/11

DataBase: ATC BLDG 2 Design F (06-30-21)

08/25/21 13:12:04

Level 5	1	73.43	51.78	-51.03	62.25	-61.75
Level 4	1	60.76	50.15	-49.42	62.25	-61.75
Level 3	1	48.09	48.24	-47.54	62.25	-61.75
Level 2	1	35.42	51.06	-50.33	62.25	-61.75
Level 1	1	19.75	59.33	-58.48	62.25	-61.75

APPLIED STORY FORCES

Type: Wind_ASCE716_3_X-Y

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	30.49	-30.04
Level 6	86.10	56.60	-55.78
Level 5	73.43	51.78	-51.03
Level 4	60.76	50.15	-49.42
Level 3	48.09	48.24	-47.54
Level 2	35.42	51.06	-50.33
Level 1	19.75	59.33	-58.48
		347.64	-342.64

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_4_X+Y_CW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	22.86	22.53	43.71	-43.79
Level 6	1	86.10	42.45	41.83	43.71	-43.79
Level 5	1	73.43	38.84	38.28	43.71	-43.79
Level 4	1	60.76	37.61	37.07	43.71	-43.79
Level 3	1	48.09	36.18	35.66	43.71	-43.79
Level 2	1	35.42	38.29	37.74	43.71	-43.79
Level 1	1	19.75	44.50	43.86	43.71	-43.79

APPLIED STORY FORCES

Type: Wind_ASCE716_4_X+Y_CW

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	22.86	22.53
Level 6	86.10	42.45	41.83
Level 5	73.43	38.84	38.28
Level 4	60.76	37.61	37.07
Level 3	48.09	36.18	35.66
Level 2	35.42	38.29	37.74
Level 1	19.75	44.50	43.86

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Loads and Applied Forces

Page 10/11

08/25/21 13:12:04

260.73 256.98

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_4_X+Y_CCW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	22.86	22.53	80.79	-79.71
Level 6	1	86.10	42.45	41.83	80.79	-79.71
Level 5	1	73.43	38.84	38.28	80.79	-79.71
Level 4	1	60.76	37.61	37.07	80.79	-79.71
Level 3	1	48.09	36.18	35.66	80.79	-79.71
Level 2	1	35.42	38.29	37.74	80.79	-79.71
Level 1	1	19.75	44.50	43.86	80.79	-79.71

APPLIED STORY FORCES

Type: Wind_ASCE716_4_X+Y_CCW

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	22.86	22.53
Level 6	86.10	42.45	41.83
Level 5	73.43	38.84	38.28
Level 4	60.76	37.61	37.07
Level 3	48.09	36.18	35.66
Level 2	35.42	38.29	37.74
Level 1	19.75	44.50	43.86

260.73 256.98

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_4_X-Y_CW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	22.86	-22.53	80.79	-43.79
Level 6	1	86.10	42.45	-41.83	80.79	-43.79
Level 5	1	73.43	38.84	-38.28	80.79	-43.79
Level 4	1	60.76	37.61	-37.07	80.79	-43.79
Level 3	1	48.09	36.18	-35.66	80.79	-43.79
Level 2	1	35.42	38.29	-37.74	80.79	-43.79
Level 1	1	19.75	44.50	-43.86	80.79	-43.79

APPLIED STORY FORCES

Type: Wind_ASCE716_4_X-Y_CW

Level	Ht	Fx	Fy
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36 of 58

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Page 11/11

08/25/21 13:12:04

Loads and Applied Forces

	ft	kips	kips
Roof	100.35	22.86	-22.53
Level 6	86.10	42.45	-41.83
Level 5	73.43	38.84	-38.28
Level 4	60.76	37.61	-37.07
Level 3	48.09	36.18	-35.66
Level 2	35.42	38.29	-37.74
Level 1	19.75	44.50	-43.86
		<hr/> 260.73	<hr/> -256.98

APPLIED DIAPHRAGM FORCES

Type: Wind_ASCE716_4_X-Y_CCW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	100.35	22.86	-22.53	43.71	-79.71
Level 6	1	86.10	42.45	-41.83	43.71	-79.71
Level 5	1	73.43	38.84	-38.28	43.71	-79.71
Level 4	1	60.76	37.61	-37.07	43.71	-79.71
Level 3	1	48.09	36.18	-35.66	43.71	-79.71
Level 2	1	35.42	38.29	-37.74	43.71	-79.71
Level 1	1	19.75	44.50	-43.86	43.71	-79.71

APPLIED STORY FORCES

Type: Wind_ASCE716_4_X-Y_CCW

Level	Ht ft	Fx kips	Fy kips
Roof	100.35	22.86	-22.53
Level 6	86.10	42.45	-41.83
Level 5	73.43	38.84	-38.28
Level 4	60.76	37.61	-37.07
Level 3	48.09	36.18	-35.66
Level 2	35.42	38.29	-37.74
Level 1	19.75	44.50	-43.86
		<hr/> 260.73	<hr/> -256.98

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Plate

P₁

1 = -82.9 PSI

2 = 25.2 PSI

P₂

1 = 130.2 PSI

2 = 25.2 PSI

P₃

1 = -177.4 PSI

2 = 25.2 PSI

P₄

1 = -67.2 PSI

2 = 61.9 PSI

P₅

1 = -104 PSI

2 = 61.9 PSI

EMF = .781

Reduction Factor (Positive)

Effective Width Area	1	2	3	4	5
10	1.0	1.0	1.0	1.0	1.0
20	1.0	.98	.95	.96	.93
50	1.0	.96	.88	.92	.84
100	1.0	.94	.82	.88	.76
500	1.0	.9	.7	.8	.6

C & C Positive Pressure

Effective Width Area	1	2	3	4	5
10	19.6	19.6	19.6	48.4	48.4
20	19.6	19.3	18.6	46.5	45.0
50	19.6	18.8	17.3	44.4	40.6
100	19.6	18.5	16.1	42.5	36.8
500	19.6	17.7	13.8	38.7	29.0

Plate

<u>P₁</u>	<u>P₂</u>	<u>P₃</u>	<u>P₄</u>	<u>P₅</u>
1 = -82.9 psf	1 = 120.2 psf	1 = -177.4 psf	1 = -67.2 psf	1 = -104 psf
2 = 25.2 psf	2 = 25.2 psf	2 = 25.2 psf	2 = 61.9 psf	2 = 61.9 psf

$EAF = .781$

Reduction Factor (negative)

<u>Effective Wind Area</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>
10	1.0	1.0	1.0	1.0	1.0
20	.96	.96	.96	.95	.95
50	.92	.92	.92	.88	.88
100	.88	.88	.88	.82	.82
500	.8	.8	.8	.7	.7

C & C Negative Pressures

<u>Effective Wind Area</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>
10	-64.7	-101.6	-128.6	-52.4	-81.2
20	-62.1	-97.6	-122	-49.9	-77.1
50	-59.6	-93.5	-117	-46.2	-71.5
100	-57.0	-89.4	-111.8	-43.0	-66.5
500	-51.8	-81.2	-110.9	-36.8	-56.8

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Building Code: IBC

Drift

08/25/21 13:14:10

Steel Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects

Member Force Output: At Face of Joint

P-Delta: Yes Scale Factor: 1.40

Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
W13	Wind DEFL	Wind_ASCE716_1_X
W14	Wind DEFL	Wind_ASCE716_1_Y
W15	Wind DEFL	Wind_ASCE716_2_X+E
W16	Wind DEFL	Wind_ASCE716_2_X-E
W17	Wind DEFL	Wind_ASCE716_2_Y+E
W18	Wind DEFL	Wind_ASCE716_2_Y-E
W19	Wind DEFL	Wind_ASCE716_3_X+Y
W20	Wind DEFL	Wind_ASCE716_3_X-Y
W21	Wind DEFL	Wind_ASCE716_4_X+Y_CW
W22	Wind DEFL	Wind_ASCE716_4_X+Y_CCW
W23	Wind DEFL	Wind_ASCE716_4_X-Y_CW
W24	Wind DEFL	Wind_ASCE716_4_X-Y_CCW
E5	EQ ELF Drift	EQ_ASCE716_X_+E_Drft
E6	EQ ELF Drift	EQ_ASCE716_X_-E_Drft
E7	EQ ELF Drift	EQ_ASCE716_Y_+E_Drft
E8	EQ ELF Drift	EQ_ASCE716_Y_-E_Drft

RESULTS:

Location (ft): (0.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	0.0007	-0.0709	0.0002	-0.0159	0.0000	0.0001
	Lp	0.0006	-0.0626	0.0001	-0.0139	0.0000	0.0001
	W13	1.5160	-0.0037	0.0845	0.0002	0.0005	0.0000
	W14	0.0060	0.4844	0.0004	0.0645	0.0000	0.0004
	W15	1.4599	0.3249	0.0823	0.0193	0.0005	0.0001
	W16	0.8142	-0.3304	0.0444	-0.0189	0.0003	0.0001
	W17	-0.3238	0.0301	-0.0190	0.0289	0.0001	0.0002
	W18	0.3329	0.6965	0.0195	0.0678	0.0001	0.0004
	W19	1.1415	0.3606	0.0636	0.0485	0.0004	0.0003
	W20	1.1325	-0.3661	0.0631	-0.0482	0.0004	0.0003
	W21	1.3446	0.7660	0.0763	0.0653	0.0004	0.0004

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Building Code: IBC

Drift

Page 2/4

08/25/21 13:14:10

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W22	0.3678	-0.2252	0.0191	0.0075	0.0001	0.0000
	W23	1.3378	0.2211	0.0759	-0.0072	0.0004	0.0000
	W24	0.3610	-0.7702	0.0187	-0.0651	0.0001	0.0004
	E5	3.0065	0.2787	0.2661	0.0262	0.0016	0.0002
	E6	2.4664	-0.2690	0.2168	-0.0236	0.0013	0.0001
	E7	-0.4585	1.2076	-0.0401	0.2365	0.0002	0.0014
	E8	0.5010	2.1807	0.0438	0.3212	0.0003	0.0019
Level 6	D	0.0006	-0.0550	0.0000	-0.0151	0.0000	0.0001
	Lp	0.0006	-0.0487	0.0000	-0.0130	0.0000	0.0001
	W13	1.4316	-0.0039	0.1284	0.0001	0.0008	0.0000
	W14	0.0057	0.4200	0.0005	0.0618	0.0000	0.0004
	W15	1.3776	0.3056	0.1239	0.0280	0.0008	0.0002
	W16	0.7698	-0.3114	0.0687	-0.0279	0.0005	0.0002
	W17	-0.3049	0.0012	-0.0277	0.0179	0.0002	0.0001
	W18	0.3134	0.6287	0.0285	0.0748	0.0002	0.0005
	W19	1.0779	0.3120	0.0967	0.0464	0.0006	0.0003
	W20	1.0694	-0.3179	0.0959	-0.0463	0.0006	0.0003
	W21	1.2682	0.7007	0.1143	0.0771	0.0008	0.0005
	W22	0.3487	-0.2327	0.0307	-0.0075	0.0002	0.0000
	W23	1.2618	0.2283	0.1137	0.0075	0.0007	0.0000
	W24	0.3423	-0.7051	0.0301	-0.0770	0.0002	0.0005
	E5	2.7403	0.2525	0.3410	0.0323	0.0022	0.0002
	E6	2.2496	-0.2454	0.2795	-0.0299	0.0018	0.0002
	E7	-0.4184	0.9711	-0.0505	0.1938	0.0003	0.0013
	E8	0.4572	1.8595	0.0552	0.3008	0.0004	0.0020
Level 5	D	0.0006	-0.0399	0.0004	-0.0128	0.0000	0.0001
	Lp	0.0006	-0.0357	0.0004	-0.0113	0.0000	0.0001
	W13	1.3032	-0.0040	0.1715	0.0001	0.0011	0.0000
	W14	0.0052	0.3582	0.0007	0.0687	0.0000	0.0005
	W15	1.2537	0.2776	0.1651	0.0370	0.0011	0.0002
	W16	0.7011	-0.2835	0.0922	-0.0368	0.0006	0.0002
	W17	-0.2772	-0.0167	-0.0366	0.0140	0.0002	0.0001
	W18	0.2849	0.5540	0.0376	0.0890	0.0002	0.0006
	W19	0.9813	0.2656	0.1292	0.0516	0.0008	0.0003
	W20	0.9735	-0.2716	0.1281	-0.0514	0.0008	0.0003
	W21	1.1540	0.6237	0.1520	0.0945	0.0010	0.0006
	W22	0.3180	-0.2252	0.0417	-0.0171	0.0003	0.0001
	W23	1.1482	0.2207	0.1512	0.0172	0.0010	0.0001
	W24	0.3122	-0.6281	0.0410	-0.0943	0.0003	0.0006
	E5	2.3994	0.2202	0.4053	0.0382	0.0027	0.0003
	E6	1.9701	-0.2154	0.3329	-0.0351	0.0022	0.0002
	E7	-0.3679	0.7773	-0.0603	0.1931	0.0004	0.0013
	E8	0.4020	1.5587	0.0658	0.3207	0.0004	0.0021

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Building Code: IBC

Drift

Page 3/4

08/25/21 13:14:10

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
Level 4	D	0.0002	-0.0271	0.0000	-0.0110	0.0000	0.0001
	Lp	0.0002	-0.0244	0.0000	-0.0098	0.0000	0.0001
	W13	1.1317	-0.0041	0.2019	-0.0001	0.0013	0.0000
	W14	0.0045	0.2895	0.0008	0.0657	0.0000	0.0004
	W15	1.0886	0.2406	0.1939	0.0429	0.0013	0.0003
	W16	0.6089	-0.2468	0.1090	-0.0431	0.0007	0.0003
	W17	-0.2406	-0.0307	-0.0426	0.0055	0.0003	0.0000
	W18	0.2473	0.4650	0.0438	0.0930	0.0003	0.0006
	W19	0.8521	0.2141	0.1520	0.0492	0.0010	0.0003
	W20	0.8454	-0.2202	0.1508	-0.0493	0.0010	0.0003
	W21	1.0019	0.5292	0.1783	0.1019	0.0012	0.0007
	W22	0.2762	-0.2081	0.0498	-0.0282	0.0003	0.0002
	W23	0.9969	0.2035	0.1774	0.0281	0.0012	0.0002
	W24	0.2712	-0.5338	0.0489	-0.1021	0.0003	0.0007
	E5	1.9941	0.1821	0.4361	0.0403	0.0029	0.0003
	E6	1.6372	-0.1803	0.3589	-0.0379	0.0024	0.0002
	E7	-0.3077	0.5842	-0.0650	0.1700	0.0004	0.0011
	E8	0.3362	1.2381	0.0711	0.3077	0.0005	0.0020
Level 3	D	0.0001	-0.0161	0.0000	-0.0076	0.0000	0.0001
	Lp	0.0001	-0.0145	0.0000	-0.0069	0.0000	0.0000
	W13	0.9298	-0.0040	0.1889	-0.0002	0.0012	0.0000
	W14	0.0037	0.2238	0.0008	0.0639	0.0000	0.0004
	W15	0.8947	0.1977	0.1819	0.0407	0.0012	0.0003
	W16	0.4999	-0.2037	0.1013	-0.0410	0.0007	0.0003
	W17	-0.1980	-0.0362	-0.0404	0.0064	0.0003	0.0000
	W18	0.2035	0.3720	0.0416	0.0895	0.0003	0.0006
	W19	0.7001	0.1649	0.1422	0.0478	0.0009	0.0003
	W20	0.6946	-0.1709	0.1411	-0.0481	0.0009	0.0003
	W21	0.8237	0.4273	0.1676	0.0976	0.0011	0.0006
	W22	0.2265	-0.1799	0.0457	-0.0259	0.0003	0.0002
	W23	0.8195	0.1755	0.1668	0.0257	0.0011	0.0002
	W24	0.2223	-0.4317	0.0448	-0.0978	0.0003	0.0006
	E5	1.5580	0.1417	0.3702	0.0346	0.0024	0.0002
	E6	1.2783	-0.1425	0.3038	-0.0327	0.0020	0.0002
	E7	-0.2426	0.4143	-0.0566	0.1540	0.0004	0.0010
	E8	0.2651	0.9303	0.0619	0.2740	0.0004	0.0018
Level 2	D	0.0001	-0.0084	0.0000	-0.0072	0.0000	0.0000
	Lp	0.0001	-0.0076	0.0000	-0.0065	0.0000	0.0000
	W13	0.7409	-0.0038	0.2231	-0.0004	0.0012	0.0000
	W14	0.0029	0.1599	0.0009	0.0783	0.0000	0.0004
	W15	0.7128	0.1570	0.2162	0.0493	0.0011	0.0003
	W16	0.3986	-0.1627	0.1185	-0.0499	0.0006	0.0003
	W17	-0.1576	-0.0426	-0.0490	0.0083	0.0003	0.0000

42 of 58

RAM Structural System

RAM Structural System 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Building Code: IBC

Drift

Page 4/4

08/25/21 13:14:10

Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W18	0.1620	0.2825	0.0504	0.1091	0.0003	0.0006
	W19	0.5579	0.1171	0.1680	0.0584	0.0009	0.0003
	W20	0.5535	-0.1228	0.1667	-0.0590	0.0009	0.0003
	W21	0.6561	0.3296	0.2000	0.1188	0.0011	0.0006
	W22	0.1808	-0.1540	0.0521	-0.0312	0.0003	0.0002
	W23	0.6528	0.1497	0.1990	0.0307	0.0011	0.0002
	W24	0.1775	-0.3339	0.0511	-0.1192	0.0003	0.0006
	E5	1.1878	0.1071	0.3816	0.0364	0.0020	0.0002
	E6	0.9745	-0.1098	0.3112	-0.0349	0.0017	0.0002
	E7	-0.1860	0.2603	-0.0610	0.1852	0.0003	0.0010
	E8	0.2033	0.6563	0.0666	0.3146	0.0004	0.0017
Level 1	D	0.0001	-0.0012	0.0001	-0.0012	0.0000	0.0000
	Lp	0.0001	-0.0011	0.0001	-0.0011	0.0000	0.0000
	W13	0.5178	-0.0034	0.5178	-0.0034	0.0022	0.0000
	W14	0.0020	0.0817	0.0020	0.0817	0.0000	0.0003
	W15	0.4965	0.1077	0.4965	0.1077	0.0021	0.0005
	W16	0.2801	-0.1128	0.2801	-0.1128	0.0012	0.0005
	W17	-0.1085	-0.0509	-0.1085	-0.0509	0.0005	0.0002
	W18	0.1116	0.1734	0.1116	0.1734	0.0005	0.0007
	W19	0.3898	0.0587	0.3898	0.0587	0.0016	0.0002
	W20	0.3868	-0.0638	0.3868	-0.0638	0.0016	0.0003
	W21	0.4561	0.2109	0.4561	0.2109	0.0019	0.0009
	W22	0.1287	-0.1228	0.1287	-0.1228	0.0005	0.0005
	W23	0.4538	0.1190	0.4538	0.1190	0.0019	0.0005
	W24	0.1264	-0.2147	0.1264	-0.2147	0.0005	0.0009
	E5	0.8062	0.0707	0.8062	0.0707	0.0034	0.0003
	E6	0.6633	-0.0749	0.6633	-0.0749	0.0028	0.0003
	E7	-0.1250	0.0751	-0.1250	0.0751	0.0005	0.0003
	E8	0.1366	0.3417	0.1366	0.3417	0.0006	0.0014

.0034*Cd(3) = .0102 --- Ok

BRACED FRAME DESIGN

1 BRACED FRAME ELEVATION
S6.00 1/8" = 1'-0"

2 BRACED FRAME ELEVATION
S6.00 1/8" = 1'-0"

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Building Code: IBC

Member Code Check

08/25/21 13:18:40

Steel Code: AISC360-16 LRFD

BRACE INFORMATION:

Story Level = Level 1 Frame Number = 1 Brace Number = 2
 Fy (ksi) = 50.00
 Brace Size = HSS10X10X1/2

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	22.25	22.25
Lu for Bending (ft) _____	22.25	22.25
K _____	1.00	1.00

CONTROLLING BRACE FORCES - SHEAR

Load Combination: 1.400 D

Shear	Top	Vux (kip) _____	-0.00
		Vuy (kip) _____	-0.00
Shear	Bot.	Vux (kip) _____	-0.00
		Vuy (kip) _____	-0.00

SHEAR CHECK:

Vux (kip) = -0.00	0.90Vnx (kip) = 216.07	Vux/0.90Vnx = 0.000
Vuy (kip) = -0.00	0.90Vny (kip) = 216.07	Vuy/0.90Vny = 0.000

CONTROLLING BRACE FORCES - AXIAL

Load Combination: 1.252 D + 0.500 Lp + 1.000 E8

AXIAL CHECK:

Pu (kip) = 339.19	0.90Pnx (kip) = 545.29	Pu/0.90Pnx = 0.622
	0.90Pny (kip) = 545.29	Pu/0.90Pny = 0.622
	0.90Pn (kip) = 545.29	Pu/0.90Pn = 0.622

CONTROLLING BRACE FORCES - FLEXURE

Load Combination: 1.252 D + 0.500 Lp + 1.000 E8

Axial	Load (kip) _____	339.19
Moment	Top	Mux (kip-ft) _____
		Muy (kip-ft) _____
Moment	Bot.	Mux (kip-ft) _____
		Muy (kip-ft) _____

CALCULATED PARAMETERS:

Pu (kip) = 339.19	0.90Pn (kip) = 545.29
Mux (kip-ft) = 0.00	0.90Mnx (kip-ft) = 227.63
Muy (kip-ft) = 0.00	0.90Mny (kip-ft) = 227.63
KL/Rx = 69.21	KL/Ry = 69.21
Cbx = 1.00	

INTERACTION EQUATION:

Pu/0.90*Pn=0.622

Pr/Pc = 0.622

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RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Building Code: IBC

Member Code Check

08/25/21 13:18:40

Steel Code: AISC360-16 LRFD

COLUMN INFORMATION:

Story Level = Level 1 Frame Number = 1 Column Number = 22
 Fy (ksi) = 50.00
 Column Size = W14X109

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	19.75	19.75
Lu for Bending (ft) _____	19.75	19.75
K _____	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.252 D + 0.500 Lp + 1.000 E7

Shear	Top	Vux (kip) _____	1.97
		Vuy (kip) _____	0.59
Shear	Bot.	Vux (kip) _____	1.97
		Vuy (kip) _____	0.59

SHEAR CHECK:

Vux (kip) = 1.97	1.00Vnx (kip) = 225.22	Vux/1.00Vnx = 0.009
Vuy (kip) = 0.59	0.90Vny (kip) = 678.02	Vuy/0.90Vny = 0.001

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.252 D + 0.500 Lp + 1.000 E8

AXIAL CHECK:

Pu (kip) = 550.49	0.90Pnx (kip) = 1295.19	Pu/0.90Pnx = 0.425
	0.90Pny (kip) = 1073.19	Pu/0.90Pny = 0.513
	0.90Pn (kip) = 1073.19	Pu/0.90Pn = 0.513

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.252 D + 0.500 Lp + 1.000 E8

Axial	Load (kip) _____	550.49
Moment	Top	Mux (kip-ft) _____
		Muy (kip-ft) _____
Moment	Bot.	Mux (kip-ft) _____
		Muy (kip-ft) _____

CALCULATED PARAMETERS:

Pu (kip) = 550.49	0.90Pn (kip) = 1073.19
Mux (kip-ft) = 33.73	0.90Mnx (kip-ft) = 720.00
Muy (kip-ft) = -8.61	0.90Mny (kip-ft) = 347.63
KL/Rx = 38.07	KL/Ry = 63.41
Cbx = 1.80	

INTERACTION EQUATION:

Pu/0.90*Pn=0.513

Eq H1-1a: 0.513 + 8/9(0.047 + 0.025) = 0.577

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RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Building Code: IBC

Member Code Check

08/25/21 13:18:40

Steel Code: AISC360-16 LRFD

BEAM INFORMATION:

Story Level = Level 1 Frame Number = 1 Beam Number = 42
 Fy (ksi) = 50.00
 Beam Size = W16X26

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	10.25	10.25
Lu for Bending (ft) _____	10.25	10.25
K _____	1.00	1.00
Top Flange Continuously Braced _____	No	
Bottom Flange Continuously Braced _____	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.200 D + 1.600 Lp

Segment distance (ft) i - end _____ 10.00
 j - end _____ 10.25

SHEAR CHECK:

Vux (kip) = -11.99	0.90Vnx (kip) = 105.97	Vux/0.90Vnx = 0.113
Vuy (kip) = -0.00	0.90Vny (kip) = 102.46	Vuy/0.90Vny = 0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.200 D + 1.600 Lp

Segment distance (ft) i - end _____ 10.25
 j - end _____ 20.50

AXIAL CHECK:

Pu (kip) = 20.71	0.90Pnx (kip) = 291.14	Pu/0.90Pnx = 0.071
	0.90Pny (kip) = 141.75	Pu/0.90Pny = 0.146
	0.90Pn (kip) = 141.75	Pu/0.90Pn = 0.146

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.200 D + 1.600 Lp

Segment distance (ft) i - end _____ 10.25
 j - end _____ 20.50

CALCULATED PARAMETERS:

Pu (kip) = 20.71	0.90Pn (kip) = 141.75
Mux (kip-ft) = -23.32	0.90Mnx (kip-ft) = 165.75
Muy (kip-ft) = 0.00	0.90Mny (kip-ft) = 20.55
KL/Rx = 19.65	KL/Ry = 110.07
Cbx = 2.82	

INTERACTION EQUATION:

Pu/0.90*Pn=0.146

Eq H1-1b: 0.073 + 0.141 + 0.000 = 0.214

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HSS 10x10x 1/2

$$R_n = 472k$$

Brace to Gussier conn:

Brace weld: $\phi R_n = 1.392(5)4(18") = 501k > 465k \rightarrow 16" min$

Net section rupture: $\phi P_n = \phi A_n U F_u = .75(16.0 in^2) .79(62ksi) = 590k > T_u \rightarrow OK$

Where:

$$A_n = 17.2 in^2 - (2)(1.25"(.465")) = 16.0 in^2$$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{2.75"}{18"} = .79$$

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{10^2 + (2)(10)}{4(10+10)} = 2.75" \quad , \quad L = 22"$$

Shear Yield: $\phi R_n = \phi (.6) F_y (b) L_w (4) = 1.0 (.6) 50ksi (.465") 18" (4) = 1000k > T_u$

Shear Rupture: $\phi R_n = \phi .6 F_u (b) L_w (4) = (.75) .6 (62ksi) .465" (18") 4 = 934k > T_u$

Gussier PL

Buckling: $K L_r = \frac{.7(10")}{2.14 \sqrt{0.75}} = 9.7 < 25$

$$\phi P_n = \phi F_y A_g = .9(26 ksi) 1.5" (12") = 486k > P_u \rightarrow OK$$

HSS 10x10x1/4

Gusset @ Beam

$$N_{ub} = 465 \text{ k} \cos(61) = 225 \text{ k}$$

$$M_{ub} = 225 \text{ k} (8") = 1800 \text{ k-in}$$

$$N_{ub} = \frac{4(1800 \text{ k-in})}{19"} = 380 \text{ k}$$

Tension yield: $\phi R_n = \phi F_y t(L) = .9(36 \text{ ksi})(1/4")(19") = 770 \text{ k} > N_{ub}$

Shear yield: $\phi R_n = \phi .6 F_y t(L) = 1.0(.6)(36 \text{ ksi})(1/4")(19") = 462 \text{ k} > V_{ub}$

Bolted conn. @ Gusset

Use 1/2" A490 Bolts

$$\phi R_n = \phi F_u A_b = .75(68 \text{ ksi})(1/2")^2 \pi = 40 \text{ k}$$

$$n = R_u / \phi R_n / \text{bolt} = 472 \text{ k} / 40 \text{ k/bolt} = 11.8 \rightarrow \text{use } (12)$$

Check 1/2" kerf R 1/2" (12) Bolts

Bearing & tearout

$$R_n = (2.4 \alpha F_u (n_b) + 1.2 L_e t F_u (n_r)) = 2.4(1/2") 1/2" (58 \text{ ksi}) 10 + 1.2(1.5") 1/2" (58 \text{ ksi}) 2 = 240 \text{ k}$$

$$\phi R_n = .75(240 \text{ k}) = 180 \text{ k} > R_u \rightarrow \text{OK}$$

Shear yielding

$$\phi R_n = .6 F_y A_{gv} = .6(36 \text{ ksi})(1/2" (12)) = 1170 \text{ k} > V_u$$

Shear rupture

$$\phi R_n = \phi .6 F_u A_{nv} = .75(.6) 58 \text{ ksi} (1/2") (12 - 4(1/2")) = 608 \text{ k} > R_u \text{ OK}$$

$$\alpha = 9.8^\circ, \beta = 14.5^\circ$$

$$e_b = 3', e_c = 7' \quad \left. \vphantom{\begin{matrix} \alpha \\ \beta \end{matrix}} \right\} r = \sqrt{(\alpha + e_b)^2 + (\beta + e_c)^2} = 28.1'$$

$$V_{uc} = \frac{\beta}{r} P_u = \frac{14.5}{28.1} (472k) = 243k$$

$$N_{uc} = \frac{e_c}{r} P_u = \frac{7}{28.1} (472k) = 117k$$

$$R_u = \sqrt{243k^2 + 117k^2} = 270k$$

$$\# Bolts = \frac{270k}{40k/0.004} = 6.8 \text{ bolts} \rightarrow \text{use 7 bolts}$$

Bearing $\rightarrow \phi R_n = \phi t F_u = 101k (1') = 101k$
Tension $\rightarrow \phi R_n = 76.7k/in = 76.7k (1') = 77k$

$\left. \begin{matrix} \phi R_n = 61k \text{ in shear} > V_{uc} \\ \phi R_n = 540k \text{ in tension} > N_{uc} \end{matrix} \right\}$

Tension yield: $\phi R_n = \phi F_y t (L) = .9(36ksi)(1')(2') = 630k > N_{uc}$

Shear yield: $\phi R_n = \phi .6 F_y t L = 1.0(.6)(36ksi)(1')(2') = 454k > V_{uc}$

Tension rupture: $\phi R_n = \phi F_u A_e = .75(58ksi)(2' - (7)(1\frac{1}{8}'))(1') = 590k > N_{uc}$

Shear rupture: $\phi R_n = \phi 0.6 F_u A_n = .75(.6)(58ksi)(2' - 7(1\frac{1}{8}'))(1') = 353k < V_{uc} \rightarrow \text{use } 7/8" \text{ PL}$

Req'd weld @ single P.

$$R_u = 270k, \theta = \tan^{-1} \left(\frac{117k}{243k} \right) = 25.8^\circ$$

$$D = \frac{R_u}{1.392(4)(1+.5 \sin(\theta))^{1.5}} = \frac{270k}{1.392(4)(1+.5 \sin(25.8^\circ))^{1.5}} = 403 \rightarrow \text{use } 1/2"$$

Foundation Calc

$$T_u = 840k$$

$$V_u = 187k$$

Check Shear Wg

$$\text{Conc. BRC} = \phi (.85) f_c A_g = .9 (.85) (34k) 6 \frac{1}{2} (7) = 105k > V_u$$

$$P_{\text{Bending}} = \phi F_y Z_x = \frac{.9 (50ksi) 2 \frac{1}{2} (7)}{4} = 492k-in < 94k (1 \frac{1}{2} + 6 \frac{1}{2} \frac{1}{2}) = 447k-in$$

Column Weld

$$T_u = 840k$$

13 7/8" Weld

$$1.39 (6) 1.5 ((10)^2 + 156^2 (2) + (15.6 - 1 \frac{1}{2} (2))^2) = 960k > P_u \rightarrow \text{OK}$$

Check (6) 1-1/2" A490 Anchors

$$T_u = 0.9 (105ksi) (1 \frac{1}{2})^2 \frac{1}{4} (6) = 1002k > T_u$$

Base R Bending

$$T_u / > \text{Base at 800} = 280k$$

$$M_u = \frac{P_u}{4} = \frac{280k (12)}{4} = 840k-in$$

$$\phi M_n = .9 F_y Z_x = .9 (50ksi) (12 (2 \frac{1}{2})^2 \frac{1}{4}) = 847k-in > M_u \rightarrow \text{use } 2 \frac{1}{2} \text{ thick R}$$

MOMENT FRAME DESIGN

RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Building Code: IBC

Member Code Check

08/25/21 13:18:40

Steel Code: AISC360-16 LRFD

BEAM INFORMATION:

Story Level = Level 1 Frame Number = 3 Beam Number = 39
 Fy (ksi) = 50.00
 Beam Size = W36X150

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	32.00	8.00
Lu for Bending (ft) _____	32.00	8.00
K _____	1.00	1.00
Top Flange Continuously Braced _____	No	
Bottom Flange Continuously Braced _____	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.252 D + 0.500 Lp + 1.000 E6

Segment distance (ft) i - end _____ 24.00
 j - end _____ 32.00

SHEAR CHECK:

Vux (kip) = -72.38	1.00Vnx (kip) = 673.13	Vux/1.00Vnx = 0.108
Vuy (kip) = 0.00	0.90Vny (kip) = 609.12	Vuy/0.90Vny = 0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.400 D

Segment distance (ft) i - end _____ 0.00
 j - end _____ 8.00

AXIAL CHECK:

Pu (kip) = 0.00	0.90Pnx (kip) = 1993.50	Pu/0.90Pnx = 0.000
	0.90Pny (kip) = 1993.50	Pu/0.90Pny = 0.000
	0.90Pn (kip) = 1993.50	Pu/0.90Pn = 0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.252 D + 0.500 Lp - 1.000 E6

Segment distance (ft) i - end _____ 0.00
 j - end _____ 8.00

CALCULATED PARAMETERS:

Pu (kip) = 0.00	0.90Pn (kip) = 1993.50
Mux (kip-ft) = -750.28	0.90Mnx (kip-ft) = 2178.75
Muy (kip-ft) = 0.00	0.90Mny (kip-ft) = 265.88
Cbx = 1.36	

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.344

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RAM Structural System

RAM Frame 17.00.01.09

DataBase: ATC BLDG 2 Design F (06-30-21)

Building Code: IBC

Member Code Check

08/25/21 13:18:40

Steel Code: AISC360-16 LRFD

COLUMN INFORMATION:

Story Level = Level 1 Frame Number = 3 Column Number = 1
 Fy (ksi) = 50.00
 Column Size = W14X730

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft) _____	19.75	19.75
Lu for Bending (ft) _____	19.75	19.75
K _____	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.252 D + 0.500 Lp - 1.000 E6

Shear	Top	Vux (kip) _____	-32.29
		Vuy (kip) _____	0.80
Shear	Bot.	Vux (kip) _____	-32.29
		Vuy (kip) _____	0.80

SHEAR CHECK:

Vux (kip) = -32.29	1.00Vnx (kip) = 2063.04	Vux/1.00Vnx = 0.016
Vuy (kip) = 0.80	0.90Vny (kip) = 4746.01	Vuy/0.90Vny = 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.200 D + 1.600 Lp

AXIAL CHECK:

Pu (kip) = 518.80	0.90Pnx (kip) = 9095.66	Pu/0.90Pnx = 0.057
	0.90Pny (kip) = 8024.28	Pu/0.90Pny = 0.065
	0.90Pn (kip) = 8024.28	Pu/0.90Pn = 0.065

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.252 D + 0.500 Lp - 1.000 E6

Axial	Load (kip) _____	434.76
Moment	Top	Mux (kip-ft) _____
		Muy (kip-ft) _____
		-15.86
Moment	Bot.	Mux (kip-ft) _____
		Muy (kip-ft) _____
		0.00
		0.00

CALCULATED PARAMETERS:

Pu (kip) = 434.76	0.90Pnx (kip) = 9095.66
	0.90Pny (kip) = 8024.28
Mux (kip-ft) = 589.46	0.90Mnx (kip-ft) = 6225.00
Muy (kip-ft) = -15.86	0.90Mny (kip-ft) = 3060.00
	Mcx (kip-ft) = 6189.59
KL/Rx = 29.06	KL/Ry = 50.58
Cbx = 1.67	

INTERACTION EQUATION:

Pu/0.90*Pn=0.048

Eq H1-3: 0.080 + 0.003 = 0.083

Eq H1-1b Per H1.3: 0.024 + 0.095 + 0.000 = 0.119

STL BM W33x130

$$\begin{aligned} A &= 38.2 \text{ in}^2 & d &= 31.1" & t_w &= 0.58" & b_f &= 11.5" \\ \phi_f &= 0.855" & k_{col} &= 1.56' & k_{beam} &= 2.75' & h &= 1.5/8" \\ Z_x &= 467 \text{ in}^3 \end{aligned}$$

STL COL W30x235

$$\begin{aligned} A &= 69.2 \text{ in}^2 & d &= 31.5" & t_w &= .85" & b_f &= 15.1" \\ \phi_f &= 1.5" & k_{col} &= 2.29 & k_{beam} &= 2.75' & h &= 1.5/8" \\ Z_x &= 847 \text{ in}^3 \end{aligned}$$

Max Mom

$$M_u = 720 \text{ k-ft} \quad \text{w-tr} = 8760 \text{ k-in}$$

Determine Req'd # of 7/8" Bolts

$$r_n = \begin{cases} 1.0 F_u A_b = 1.0 (84 \text{ ksi}) (60) \text{ in}^2 = 50.5 \text{ y/bolt} \rightarrow \text{control} \\ 2.4 F_u b \phi_{bft} = 2.4 (1.1) (65 \text{ ksi}) (7/8") (8.55') = 128 \text{ y/bolt} \\ 2.4 F_u p d \phi_{bft} = 2.4 (4) (65 \text{ ksi}) (7/8") (7/4") = 225 \text{ y/bolt} \end{cases}$$

$$n = \frac{1.75 M_u}{\phi_r (d+t)} = \frac{1.75 (8760 \text{ k-in})}{.9 (50.5 \text{ y/bolt}) (31.5" + 7.5")} = 7 \text{ bolts} \rightarrow \text{use } (8) \text{ bolts}$$

Determine Force in Flange

$$F_{fr} = \frac{8760 \text{ k-in}}{31.5" + 7.5"} = 260 \text{ kips}$$

$$n = \frac{540 \text{ k}}{.9 (50.5 \text{ y/bolt})} = 12 < 16 \rightarrow \text{okay}$$

Check R Yielding

$$\phi R_n = \phi F_y A_g = .9 (50 \text{ ksi}) .75'' (11.5'') = 389 \text{ k} > 260 \text{ k}$$

Check R Rupture

$$\phi R_n = \phi F_u A_n = .9 (65 \text{ ksi}) (.75'') (11.5'' - 2 (1.5'')) = 422 \text{ k} > 260 \text{ k}$$

Check BM Block Shear Rupture

$$R_n = \min (.6 F_u A_{nv} + F_u A_{nt}, .6 F_y A_{gv} + F_u A_{nt}) = 740 \text{ k}$$

$$A_{gv} = (2) (10'') .75'' = 15 \text{ in}^2$$

$$A_{nv} = (2) .75'' (10'' - 2 (1.5'')) = 11 \text{ in}^2$$

$$A_{nt} = (2) .75'' (1'' - 1.5'') = 0 \text{ in}^2$$

$$\phi R_n = .9 (740 \text{ k}) = 666 \text{ k} > 260 \text{ k}$$

Check R Bending

$$L = 4'$$

$$k = 0.65$$

$$k L_r = \frac{.65 (4'')}{.75''/\sqrt{12}} = 12 < 25 \rightarrow \phi R_n = \phi F_y A_g$$

$$\phi R_n = \phi F_y A_g = .9 (50 \text{ ksi}) .75'' (11.5'') = 389 \text{ k} > 260 \text{ k}$$

Check column Panel Zone Shear

$$R_u = \frac{(2) M_u}{d_c + (2) \rho} = \frac{(2) 8760 \text{ k-in}}{21.5'' + (2) .75''} = 540 \text{ k-in}$$

$$\phi R = \phi (.6) F_y d_c t_w \left(1 + \frac{3 b_f t_w^2}{d_s d_c t_w} \right) = 1.0 (.6) 50 \text{ ksi} (21.5'') .85'' \left(1 + \frac{3 (11.5'') (1.5'')^2}{21.5'' (21.5'') .85''} \right) = 871 \text{ k-in} > R_u$$

Ground Motion Selection for Performance-Based Seismic Evaluation

This section presents background documentation for ground motion selection conducted to support performance-based seismic evaluation of Building 2 (7-story healthcare facility in Long Island, New York) designed for 1987 NBC “Risk Category IV” “Seismic Design Category B” equivalent (Design Case C). Seismic design criteria, performance-based seismic evaluation findings, and structural design drawings are documented in NIST GCR 21-917-48v2A.

TABLE OF CONTENTS

General	Page 1
Lateral Force Resisting System	Page 1
Gravity Force Resisting System	Page 3
Codes and References	Page 3
Performance Objectives for Structural Performance Assessment	Page 4
Site-Specific Probabilistic Seismic Hazard Analysis	Page 4
Ground Motion Record Selection	Page 6
Spectrum Matching	Page 7
APPENDIX A - PEER NGA West Search Parameters and Ground Motions	Page 10
APPENDIX B - Spectral Matching Results	Page 46

LIST OF FIGURES

Figure 1 - 3D View (SW Corner) of LFRS	Page 1
Figure 2 - MCER Target Spectrum	Page 3
Figure 3 - Attenuation Equations for Sources	Page 4
Figure 4 - Magnitude-Distance Deaggregation	Page 5
Figure 5 - Spectrum Matching - Kocaeli Turkey	Page 7
Figure 6 - All Earthquake Resultant Spectrums	Page 8
Figure 7 - Resultant Mean/Maximum Direction Spectrum	Page 8
Figure 8 - Spectrum Comparison	Page 9

LIST OF TABLES

Table 1 - Ground Motion BIN's	Page 6
Table 2 - Ground Motion BIN Matching	Page 6
Table 3 - Selected Ground Motion Summary	Page 7

General

The Building 2 structure is an archetype steel building for a Healthcare Facility.

This building is a 7-story structure located on Long Island, NY (40.7379 Latitude, -73.6398 Longitude). The building is approximately square in plan with typical plan dimensions of 124ft x 129ft, and a roof height of approximately 99ft above the adjacent grade.

The Site-Specific Probabilistic Seismic Hazard Analysis, Ground Motion Selection, and Spectrum Matching process was overseen and peer reviewed by CB Crouse, PhD, PE, who served as the subject matter expert.

Lateral Force Resisting System

The complete Lateral Force Resisting System (LFRS) consists of reinforced slab on metal deck diaphragms, 2-story X steel concentric braced frames in the north-south (H2) direction, steel moment frames in the east-west (H1) direction, and all attachments between these elements and the foundation.

The plan layout of the LFRS is 'H' shaped with moment resisting frames taking east-west lateral loads and located on the north and south sides of the structure. The braced frames resist the north-south lateral loads and are located at approximately the centerline of the building. See *Figure 1* for orientation of the LFRS.

Figure 1 - 3D View (SW Corner) of LFRS

Under the 1987 National Building Code (NBC), the structure would be classified as ‘Braced Frame’ and ‘Space Frame’ systems. The horizontal force factor ‘K’ is 1.0 for braced frames (as defined in the NBC, which are equivalent to steel ordinary concentrically braced frames per the Minimum Design Loads and Associated Criteria for Buildings and Other Structures, American Society of Civil Engineers [ASCE] 7-16) and 0.67 for ‘special’ space frames (as defined in the NBC, which are equivalent to ordinary moment frames per ASCE 7-16). The NBC code level base shear for this building has been calculated to be 962 kips in the north-south direction and 645 kips in the east-west direction.

Foundation support is provided by reinforced concrete spread footings interconnected by reinforced concrete grade beams.

Approximate building periods as calculated by the analysis model indicate the period range for scaling and matching per ASCE 7-16 16.2 are as follows:

Upper Bound:

- $T_{H1} = 1.62 \text{ seconds} \times 2 \Rightarrow \mathbf{3.24 \text{ seconds}}$
- $T_{H2} = 0.57 \text{ seconds} \times 2 \Rightarrow \mathbf{1.14 \text{ seconds}}$

Lower Bound:

- $T_{H1} = \mathbf{0.61 \text{ seconds}} > 20\% \text{ of } 1.62 = 0.32 \text{ seconds (at 90\% mass participation)}$
- $T_{H2} = \mathbf{0.45 \text{ seconds}} > 20\% \text{ of } 0.57 = 0.11 \text{ seconds (at 90\% mass participation)}$

The software used for analysis was Perform3d version 8.0.0 by Computers and Structures, Inc.

See *Figure 2* for the Maximum Considered Earthquake [MCE] record spectrum calculated for the site set at a value of 760 m/s (assumed) for the shear wave velocity averaged over top 30 meters.

Figure 2 - MCER Target Spectrum

Gravity Force Resisting System

The gravity force resisting system is comprised of normal weight concrete slab on metal deck with composite steel beams and steel columns. Columns are supported on concrete spread footings.

Codes and References

The design of Building 2 - Healthcare Facility structure is intended to satisfy the requirements of the NBC for an “Essential Facility” or “Risk Category IV” equivalent structure as defined by the 2018 International Building Code (IBC). The design of the LFRS was performed per the Distribution of Lateral Forces requirements as outlined in Section 1113.5 of the NBC. Specific material design was in accordance with American Concrete Institute Building Code Requirements for Structural Concrete (ACI) 318-83 and Eighth Edition of the Manual of Steel Construction, American Institute of Steel Construction, 1978 (AISC).

Performance Objectives for Structural Performance Assessment

Overall Building Performance

The assessment of the LFRS will be performed using the Non-Linear Dynamic Procedure (NDP) per Section 7.4.4 of the Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers [ASCE] 41-17. Modeling and Acceptance Criteria were determined using ASCE 41-17 and the Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings, Applied Technology Council (ATC) 72-1. Acceptance Criteria were selected consistent with achieving a Life Safety (LS) level of performance for the Maximum Considered Earthquake/Basic Safety Earthquake (MCE/BSE-2N) level of shaking and an Immediate Occupancy (IO) level of performance for the Design Basis Earthquake (DBE/BSE-1N) level of shaking using Non-Linear Time History Analysis (NLTHA).

Site-Specific Probabilistic Seismic Hazard Analysis

A site-specific probabilistic seismic hazard analysis based on the 2014 United States Geological Survey (USGS) Seismic Source Characterization (SSC) model was performed for this project utilizing the commercial software EZ-Frisk version 8.07 by Fugro. The 2018 USGS SSC Model contains seismic source geometries and recurrence models developed by USGS for the 2014 Update of the United States National Seismic Hazard Maps.

The group of ground motion prediction equations as indicated in *Figure 3* below were used to evaluate the MCE hazard at the site. Intensity type used was the geometric mean horizontal component of spectral response at 5% damping. See *Figure 3* below from EZ-Frisk outlining the seismic sources, attenuation equations, and the software calculated weight factors.

Analysis Options														Seismic sources	Attenuation equations	Equations for sources	Site parameters	Calculation parameters	
		Attenuation equations		Frankel (1996)	Toro (1999)			Tavakoli-Pezeshk (2005)	Atkinson-Boore (2006p)	Pezeshk et al (2011)	Atkinson (2008p)	Somerville (2001)							
				USGS 2008	USGS 2008	Silva et al (2002)	Campbell (2003)	ENA USGS	USGS 2014	USGS 2014	USGS 2014	USGS 2014	USGS 2008						
				Truncated Mw	2008 Mw	2008 Mw	2008 Mw	2008 Mw	Table Soil	Table Soil	Table Soil	Table Soil	2008 Mw						
Seismic source	Region	Fault mechanism																	
CEUS Gridded Source	USGS 2014 CEUS Gridded Source	Strike Slip Fault		0.06	0.13	0.06	0.13	0.13	0.25	0.16	0.08								
ERM-N Composite	USGS 2014 RLME Eastern Rift Margin-N	Strike Slip Fault		0.06	0.11	0.06	0.11	0.11	0.22	0.15	0.08	0.1							
Wabash Valley Composite	USGS 2014 RLME Wabash Valley	Strike Slip Fault		0.06	0.11	0.06	0.11	0.11	0.22	0.15	0.08	0.1							
New Madrid - Composite	USGS 2014 New Madrid	Reverse Fault		0.06	0.11	0.06	0.11	0.11	0.22	0.15	0.08	0.1							
ERM-S Composite	USGS 2014 RLME Eastern Rift Margin-S	Strike Slip Fault		0.06	0.11	0.06	0.11	0.11	0.22	0.15	0.08	0.1							
Commerce Composite	USGS 2014 RLME Commerce	Strike Slip Fault		0.06	0.11	0.06	0.11	0.11	0.22	0.15	0.08	0.1							
Marianna Composite	USGS 2014 RLME Marianna	Strike Slip Fault		0.06	0.11	0.06	0.11	0.11	0.22	0.15	0.08	0.1							
Charleston Composite	USGS 2014 RLME Charleston	Strike Slip Fault		0.16			0.17	0.17	0.3	0.2									
Meers Composite	USGS 2014 Oklahoma	Strike Slip Fault		0.16			0.17	0.17	0.3	0.2									
Cheraw Composite	USGS 2014 Colorado	Normal Fault		0.16			0.17	0.17	0.3	0.2									
Charlevoix Composite	USGS 2014 RLME Charlevoix	Strike Slip Fault		0.16			0.17	0.17	0.3	0.2									

Figure 3 – Attenuation Equations for Sources

A probabilistic seismic hazard analysis with a deaggregate seismic hazard analysis for a probability level of 1 second 2475-year spectral acceleration was computed. See *Figure 4* for the resulting plot.

**Magnitude-Distance Deaggregation
Spectral Response @ 5% Damping - Average Horizontal Component**

Period: 1
Amplitude: 0.0456186
Hazard: 0.000404042
Mean Magnitude: 6.24
Mean Distance: 141.40

Figure 4

The deaggregation data was broken up into three separate bins to be representative of the results and determine the number of record pairs of earthquake ground motions required from each bin. See Table 1:

Table 1 - Ground Motion BIN's

BIN	Magnitude Range (all distances)	Magnitude Average	Distance Rbar Average (km)	# of Ground Motion Pairs
1	5.05-5.95	5.50	38	4
2	6.05-6.95	6.48	139	5
3	7.05-7.95	7.28	298	2

Ground Motion Record Selection

Ground motions were selected from the Pacific Earthquake Engineering Research Center [PEER] Ground Motion Database NGA-West with the implementation of the MCE Target spectrum and parameters determined from the deaggregation study. See Appendix A for PEER NGA-West search parameters and ground motions. The PEER Ground Motion Database NGA-East was reviewed, but did not have appropriate ground motions for the site parameters determined in the hazard analysis. The geographical location of the ground motions is not as critical as aligning with the parameters determined by the deaggregation study which is geographically site specific.

Table 2 – Ground Motion Bin Matching

BIN #	Name, Location	Magnitude	Rrup (km)
1	Southern Calif, CA	6.00	73
	Livermore-01, CA	5.80	36
	Westmorland, CA	5.90	19
	Chalfant Valley-01, CA	5.77	24
	Bin Average	5.87	38
	Bin Deaggregation Target	5.50	38
2	Borrego Mtn, CA	6.63	129
	Bam Iran, Iran	6.20	114
	Chi-Chi Taiwan - 04, Taiwan	6.60	105
	Tottori Japan, Japan	6.61	170
	Darfield New Zeland, New Zeland	7.00	175
	Bin Average	6.61	138
	Bin Deaggregation Target	6.48	138
3	Kocaeli Turkey, Turkey	7.51	165
	Hector Mine, CA	7.13	205
	Bin Average	7.32	185
	Bin Deaggregation Target	7.28	298

Eleven pairs of ground motions were selected per ASCE 7-16 section 16.2.2. The unscaled records are summarized in Table 3:

Table 3 - Selected Ground Motion Summary

Name, Location	Date	Magnitude	Fault Type	Station	Rrup (km)	RSN	Vs30 (m/sec)
Southern Calif, CA	1952	6.00	Strike Slip	San Luis Obispo	73	17	493.50
Livermore-01, CA	1980	5.80	Strike Slip	Fremont - Mission San Jose	36	213	367.57
Westmorland, CA	1981	5.90	Strike Slip	Superstition Mtn Camera	19	318	362.38
Chalfant Valley-01, CA	1986	5.77	Strike Slip	Lake Crowley - Shehorn Res	24	546	456.83
Borrego Mtn, CA	1968	6.63	Strike Slip	San Onofre - So Cal Edison	129	40	442.88
Bam Iran, Iran	1999	6.20	Strike Slip	TCU033	114	2846	423.40
Chi-Chi Taiwan - 04, Taiwan	2003	6.60	Strike Slip	Golbaf	105	4045	441.84
Tottori Japan, Japan	2000	6.61	Strike Slip	SMN014	170	6373	385.91
Darfield New Zeland, New Zeland	2010	7.00	Strike Slip	OAMS	175	6945	437.72
Kocaeli Turkey, Turkey	1999	7.51	Strike Slip	Tekirdag	165	1172	521.76
Hector Mine, CA	1999	7.13	Strike Slip	Castaic - Old Ridge Route	205	1775	450.28

Spectrum Matching

To scale the records appropriately per ASCE 7-16 section 16.2.3.2 for the two hazard levels: MCE/BSE-2N and DBE/BSE-1N, the record pairs first were matched to the target spectrum using EZ-Frisk. The matching strategy utilized was the Enhanced AI Atik Approach. See example matched spectrum results for the Kocaeli Turkey record in *Figure 5* (Step 4 indicates the final step in the matching procedure). See Appendix B for matched spectrum results for all ground motions.

Figure 5 - Spectrum Matching - Kocaeli Turkey

The Maximum Direction Spectrum was determined using BiSpec Professional 2.20 software by Earthquake Solutions which is a nonlinear spectral analysis software program. See *Figure 6* and *Figure 7* for the All-Earthquakes Resultant Spectrum plot and the Resultant Mean/Maximum Direction Spectra results.

Figure 6 - All Earthquake Resultant Spectrums

Figure 7 - Resultant Mean/Maximum Direction Spectrum

The Mean Maximum Direction Spectrum was then compared to the MCE Target Design Spectrum and the Scaled (x1.1) MCE Target Design Spectrum as defined in ASCE 7-16 16.2. See *Figure 8* for log scale plot for spectrum comparison. In review it was determined the scaled spectrum would be sufficient and meet the intents of the code provisions.

Figure 8 - Spectrum Comparison

The eleven pairs of ground motion accelerations were then scaled accordingly to produce the MCE (x1.1) and Design Based Earthquake [DBE] ($x1.1^{2/3}$) events.

APPENDIX A

PEER NGA West Search Parameters and Ground Motions

Load Sample Input Values

Clear Input Values

Search

These characteristics are defined in the NGA-West2 Flatfile. You need to re-run Search when any of these parameters are updated.

Record Characteristics:

RSN(s) : RSN1,...RSNn

Event Name :

Station Name :

Search Parameters:

Fault Type :

Magnitude :
min,max

R_JB(km) :
min,max

R_rup(km) :
min,max

Vs30(m/s) :
min,max

D5-95(sec) :
min,max

Pulse :

Additional Characteristics:

Max No. Records :
(<=100)

Initial ScaleFactor :
min,max

Suite

Spectral Ordinate :

Damping Ratio :

Suite Average :

Scaling

Scaling Method :

MSE = Computed Weighted Mean Squared Error of record, and suite average, wrt target spectrum.

Weight Function

Used in both search and scaling when computing MSE. Values can be updated for rescaling. Intermediate points are interpolated with $W = \text{fxn}(\log(T))$

Period Points : (T1,T2, ... Tn)

Weights : (W1,W2, ... Wn)

Record Sequence Number: 17
 Earthquake Name: Southern Calif
 Station Name: San Luis Obispo
 Result ID: 1
 Scale Factor: 1.9169739

Display time series of selected record:

Horizontal-1 Component

Display time series of selected record: Unscaled ▼

Horizontal-2 Component

Vertical

Record Sequence Number: 213
 Earthquake Name: Livermore-01
 Station Name: Fremont - Mission San Jose
 Result ID: 5
 Scale Factor: 1.70701003

-Record is highlighted in yellow in the plot
 Click on each label in the legend to zoom
 Window Zooming is possible only

Display time series of selected record: Unscaled ▼

Horizontal-1 Component

Horizontal-2 Component

Display time series of selected record:

Horizontal-2 Component

Vertical Component

Record Sequence Number: 318
 Earthquake Name: Westmorland
 Station Name: Superstition Mtn Camera
 Result ID: 12
 Scale Factor: 1.25541393

-Record is highlighted in yellow
 Click on each label in the legend
 Window Zooming is possible

Display time series of selected record:

Horizontal-1 Component

Horizontal-2 Component

Display time series of selected record:

Horizontal-2 Component

Vertical Component

Record Sequence Number: 546
 Earthquake Name: Chalfant Valley-01
 Station Name: Lake Crowley - Shehorn Res.
 Result ID: 18
 Scale Factor: 2.9111227

Display time series of selected record: Unscaled ▼

Horizontal-1 Component

Horizontal-2 Component

Display time series of selected record:

Horizontal-2 Component

Vertical Component

Load Sample Input Values
Clear Input Values

Search

These characteristics are defined in the NGA-West2 Flatfile. You need to re-run Search when any of these parameters are updated.

Record Characteristics:

RSN(s) : *RSN1,..RSNn*

Event Name :

Station Name :

Search Parameters:

Fault Type :

Magnitude :

min,max

R_JB(km) :

min,max

R_rup(km) :

min,max

Vs30(m/s) :

min,max

D5-95(sec) :

min,max

Pulse :

Additional Characteristics:

Max No. Records :

(≤ 100)

Initial ScaleFactor :

min,max

Suite

Spectral Ordinate :

Damping Ratio :

Suite Average :

Scaling

Scaling Method :

MSE = Computed Weighted Mean Squared Error of record, and suite average, wrt target spectrum.

Weight Function

Used in both search and scaling when computing MSE. Values can be updated for rescaling. Intermediate points are interpolated with $W = \text{fxn}(\log(T))$

Period Points : ($T_1, T_2, \dots T_n$)

Weights : ($W_1, W_2, \dots W_n$)

Record Sequence Number: 40
 Earthquake Name: Borrego Mtn
 Station Name: San Onofre - So Cal Edison
 Result ID: 1
 Scale Factor: 1.70617394

-Record is highlighted in yellow
 Click on each label in the legend
 Window Zooming is possible or

Display time series of selected record:

Horizontal-1 Component

Horizontal-2 Component

Display time series of selected record: Unscaled ▼

Horizontal-2 Component

Vertical Component

Record Sequence Number: 2846
 Earthquake Name: Chi-Chi, Taiwan-04
 Station Name: TCU033
 Result ID: 3
 Scale Factor: 2.96133649

Display time series of selected record:

Horizontal-1 Component

Display time series of selected record: Unscaled ▼

Horizontal-2 Component

Vertical Component

Results -- Selected-Record -- Response Spectra -- Bam, Iran :: Golbaf

Record Sequence Number: 4045
 Earthquake Name: Bam, Iran
 Station Name: Golbaf
 Result ID: 4
 Scale Factor: 2.54743321

Display time series of selected record: ▼

Horizontal-1 Component

Display time series of selected record: ▼

Horizontal-2 Component

Results -- Selected-Record -- Response Spectra -- Tottori, Japan :: SMN014

Record Sequence Number: 6373
 Earthquake Name: Tottori, Japan
 Station Name: SMN014
 Result ID: 14
 Scale Factor: 2.90991512

Display time series of selected record:

Horizontal-1 Component

Display time series of selected record: Unscaled

Horizontal-2 Component

Record Sequence Number: 6945
 Earthquake Name: Darfield, New Zealand
 Station Name: OAMS
 Result ID: 20
 Scale Factor: 2.57209731

Display time series of selected record:

Horizontal-1 Component

Display time series of selected record: Unscaled

Horizontal-2 Component

Vertical Component

Load Sample Input Values

Clear Input Values

Search

These characteristics are defined in the NGA-West2 Flatfile. You need to re-run Search when any of these parameters are updated.

Record Characteristics:

RSN(s) : *RSN1,...RSNn*

Event Name :

Station Name :

Search Parameters:

Fault Type :

Magnitude :
min,max

R_JB(km) :
min,max

R_rup(km) :
min,max

Vs30(m/s) :
min,max

D5-95(sec) :
min,max

Pulse :

Additional Characteristics:

Max No. Records :
(<=100)

Initial ScaleFactor :
min,max

Suite

Spectral Ordinate :

Damping Ratio :

Suite Average :

Scaling

Scaling Method :

MSE = Computed Weighted Mean Squared Error of record, and suite average, wrt target spectrum.

Weight Function

Used in both search and scaling when computing MSE. Values can be updated for rescaling. Intermediate points are interpolated with $W = \text{fxn}(\log(T))$

Period Points : ($T_1, T_2, \dots T_n$)

Weights : ($W_1, W_2, \dots W_n$)

Record Sequence Number: 1172
 Earthquake Name: Kocaeli, Turkey
 Station Name: Tekirdag
 Result ID: 1
 Scale Factor: 1.93269994

Display time series of selected record: ▼

Horizontal-1 Component

Display time series of selected record: Unscaled ▼

Horizontal-2 Component

Verti

acc (g)

velo (cm/s)

disp (cm)

Record Sequence Number: 1775
 Earthquake Name: Hector Mine
 Station Name: Castaic - Old Ridge Route
 Result ID: 5
 Scale Factor: 1.74820301

-Record is highlighted in yellow
 Click on each label in the legend
 Window Zooming is possible

- Target Spectrum
- Scaled Spectra:
 - GeoMean
 - Geometric mean
 - Scaled Spectra:
 - Geometric mean + stdev
 - Scaled Spectra:
 - Geometric mean - stdev
 - H1
 - H2
 - Ψ
 - SRSS
 - RotD50
 - RotD100
 - Geomean

Display time series of selected record:

Horizontal-1 Component

Horizontal-2 Component

Display time series of selected record: Unscaled

Horizontal-2 Component

Vertical Component

APPENDIX B

Spectral Matching Results

Record: Kocaeli_TK N-S

Ground Motion

Spectrum

Convergence

Record: Kocaeli_TK E-W

Ground Motion

Spectrum

Convergence

Record: Hector_ORR N-S

Spectrum

Convergence

Record: Hector_ORR E-W

Ground Motion

Spectrum

Convergence

Record: Borrego Mtn N-S

Ground Motion

Spectrum

Convergence

Record: Borrego Mtn E-W

Ground Motion

Spectrum

Convergence

Record: Chi-Chi_Taiwan N-S

Ground Motion

Spectrum

Convergence

Record: Chi-Chi_Taiwan E-W

Ground Motion

Spectrum

Convergence

Record: Bam_Iran N-S

Ground Motion

Spectrum

Convergence

Spectrum

Convergence

Record: Tottori_Japan N-S

Ground Motion

Spectrum

Convergence

Convergence

Record: Darfield_New Zealand N-S

Ground Motion

Spectrum

Convergence

Record: Darfield_New Zealand E-W

Ground Motion

Spectrum

Convergence

Record: Southern Calif N-S

Ground Motion

Spectrum

Convergence

Record: Southern Calif E-W

Ground Motion

Spectrum

Convergence

Ground Motion

Spectrum

Convergence

Convergence

Ground Motion

Spectrum

Convergence

Record: Westmorland E-W

Ground Motion

Spectrum

Convergence

Record: Chalfant Valley-01 N-S

Ground Motion

Spectrum

Convergence

Record: Chalfant Valley-01 E-W

Ground Motion

Spectrum

Convergence