1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
Shear Modulus:	G =	11200	ksi
Yield Strength:	F_{y} =	50	ksi
Ultimate Strength:	Fu =	70	ksi

Reference: AISC 14th
Section *Eq/Fig/Table*

Problem 4.18:

E

Select a pair of C10 channels for a tension member subjected to a dead lead of 120 kips and a live load of 275 kips. The channels are placed back to back and connected to a 3/4-in gusset plate by 7/8-in Ø bolts. Assume A588 Grade 50 steel for the channels and assume the gusset plate is sufficient. The member is 25 ft long. The bolts are arranged in two lines parallel to the length of the member. There are two bolts in each line 4 in on center.

Plate			
	F_{y} =	50	ksi
	Qty =	1	
Plate Width:	w_{tp} =	9	in
Plate Thickness:	t_{tp} =	3/4	in

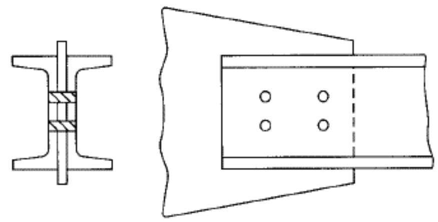


FIGURE P4-18

Required:

a) Select C10 channels for tension member

Method:

i) Determine Controlling Load Combination
ii) Check for limit states
iii) Check slenderness Ratio

Solution:

Member Length	L =	25	ft		
Dead Load	DL=	120	kip		120 kip
Live Load	LL=	275	kip		275 kip
Factors	ϕ_t=	0.9		Ω_t=	1.5
	ϕ_r=	0.75		Ω_t=	2
Number of Members		2			

Referenc AISC 14th
Section 'Fig/Table/No

LRFD | ASD

1) Demand: | **Demand:**

	LRFD			ASD		
Load	Pu =	584	kip	Pa =	395	kip
Yielding:	Ag =	12.98	in^2	Ag =	5.27	in^2
Area/Member	Arqd =	6.49	in^2	Arqd =	2.63	in^2

2) Capacity: | **Capacity:**

Member Selected: **C10X30** | **C10X30**

Web Thickness:	t_{wch}	0.673	in	t_{wch}	0.673	in	
	A_{ch}	8.81	in^2	A_{ch}	8.81	in^2	
Eccentricity	xbar	0.649	twch	xbar	0.649	twch	
	rmin	1.22	in	rmin	1.22	in	AISC Table 1-16

i) Gross Section Yielding: Gross Section Yielding:

Ag=	17.62	in^2	Ag=	17.62	in^2
Pn=	881.00	kip	Pn=	881.00	kip
ϕ_tPn=	792.90	kip	ϕ_tPn=	587.33	kip

ii) Tensile Rupture Strength: Tensile Rupture Strength:

Connection: **Connection:**

# Bolts/Channel/Area:		2	Units		2	Units
Bolt Size:	ϕ=	7/8	in	ϕ=	7/8	in
Area bolt holes:	Ah =	2.69	in^2	Ah =	2.69	in^2

Channel Section Strength **Channel Section Strength**

Nominal Area	An=	14.93	in^2	An=	14.93	in^2
Shear Lag	U=	0.84		U=	0.84	
Effective Area	Ae=	12.51	in^2	Ae=	12.51	in^2
Nominal Strength	Pn=	875.42	kip	Pn=	875.42	kip
Capacity	ϕ_tPn=	656.56	kip	Pn/Ω=	437.71	kip

iii) Design Check:		**Design Check:**	**Design Check**
Tension Strength	OK	OK	
Slenderness Ratio:	OK	OK	

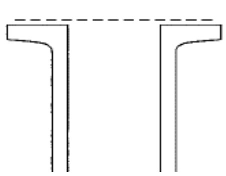
LRFD: 2 - C10X30 **ASD: 2 -** C10X30 **ANSWER**

6.22. MATERIAL PROPERTIES:

Modulus of Elasticity	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	$F_{y\,=}$	36	ksi
	$Fu_{\,=}$	70	ksi

Problem 6.22:

Select the lightest pair of C9 channels to support the loads $P_D = 50$ k and $P_L = 90$ k. The member is to be 20 ft long with both ends pinned and is to be arranged as shown in the accompanying illustration. Use A36 steel and design single lacing and end tie plates, assuming that $\frac{3}{4}$-in diameter bolts are to be used for connections. Assume that the bolts are located $1\frac{1}{4}$ in from the back of channels. Solve by LRFD and ASD procedures.



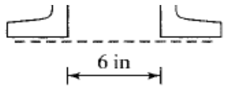


FIGURE P6-22

Required:

a) Select C9 channels for compression member
b) Design Single Lacing For Channels
c) Design End Tie Plates

Method:

i) Determine Controlling Load Combination
ii) Check for limit states
iii) Check slenderness Ratio

Solution:

Member Length	L =	20	ft				
Dead Load	DL=	50	kip		50	kip	
Live Load	LL=	90	kip		90	kip	
Factors	ϕ_t=	0.9		Ω_t=	1.5		
	ϕ_r=	0.75		Ω_t=	2		
Number of Members		2	Units				
Distance between members		6	in				Referenc AISC 14th

Section *Eq/Fig/Table*

PART A: DESIGN OF CHANNELS E

LRFD				ASD			Reference Section	Eq/Fig/Table
1) Demand:				**Demand:**				
Load	Pu =	204	kip	Pa =	140	kip		
Compression:								
Assume	K =	1		K =	1			
	KL/r =	80		KL/r =	80			
	ϕF_{cr} =	25	ksi	ϕF_{cr} =	25	ksi	AISC	Table 4-22
Area required:	Ag =	8.16	in^2	Ag =	5.60	in^2		
Area/Member	Arqd =	4.08	in^2	Arqd =	2.80	in^2		
2) Capacity:				**Capacity:**				
Member Selected:		C9X15			C9X15		AISC	Table 1-1
Web Thickness:	t_{wch}	0.285	in	t_{wch}	0.285	in	AISC	Table 1-1
	A_{ch}	4.41	in^2	A_{ch}	4.41	in^2	AISC	Table 1-1
Eccentricity	xbar	0.586	in	xbar	0.586	in	AISC	Table 1-1
Mom. Inertia, x	I_{xch}	51	in^4	I_{xch}	51	in^4	AISC	Table 1-1
Mom. Inertia, y	I_{ych}	1.91	in^4	I_{ych}	1.91	in^4	AISC	Table 1-1
Radius of Gyration, x	r_{xw} =	3.4	in	r_{xw} =	3.4	in	AISC	Table 1-1
Radius of Gyration, y	ryw =	0.659	in	ryw =	0.659	in	AISC	Table 1-1

Built Up Section **Built Up Section**

Distance to centroid	dx =	0	in	dx =	0	in		
Distance to centroid	dy =	3.59	in	dy =	3.59	in		
Mom. Inertia, x	I_{xch}	102.00	in^4	I_{xch}	102.00	in^4		
Mom. Inertia, y	I_{ych}	117.24	in^4	I_{ych}	117.24	in^4		
Smallest	Imin	102.00	in^4	Imin	102.00	in^4		
	rmax	3.40	in	rmax	3.40	in		
	KL/r =	70.6		KL/r =	70.6		AISC	Table 4-22
	$\phi F_{cr} =$	24.88	ksi	$F_{cr/\Omega} =$	16.58	ksi	*Interpolation*	
	ϕPn=	219.44	kip	Pn/Ω=	146.24	kip		

iii) Design Check:		**Design Check:**	**Design Check**
Compressive Strength	OK	OK	

LRFD: 2 - C9X15 **ASD: 2 - C9X15** **ANSWER**

PART B: DESIGN OF SINGLE LACING

Lacing Check:

Boltline distance from channel		1 1/4	in	dc =	1 1/4	in	Min	Table J3.4
Distance between bolt lines		8 1/2	in	db =	8 1/2	in		
Angle between Laces		60	deg	angle =	60	deg		
Length of Lace		9.8	in	length =	9.8	in		
Smallest Radius of Gyration		0.659	in	rmin =	0.659	in		
Stiffness per Lace		14.9		klace =	14.9			
Stiffness of Member:		70.6		kmember =	70.6			
Check:	Klace<Kmember ?	OK			OK			

i Demand Force on lacing bar:

Shear Force	Vu =	4.39	kip	Va =	2.92	kip	
Force per side	Vu/2 =	2.19	kip	Vu/2 =	1.46	kip	
Compressive Force:	Pc =	2.53		Pc =	1.69		Demand Force

ii Capacity

Design of bar:

Width	b =	1	in	b =	1	in		
Thickness	**t =**	1/4	in	**t =**	1/4	in	Trial	
Area	A =	1/4	in^2	A =	1/4	in^2		
Moment of Inertia	**I =**	0.00	in^4	**I =**	0.00	in^4		
Radius of Gyration	**r =**	0.07	in	**r =**	0.07	in		
Assume	K =	1		K =	1			
	KL/r =	135.8		KL/r =	39.2		AISC	Table 4-22
Capacity Strength	$\phi F_{cr} =$	12.21	ksi	$F_{cr/\Omega} =$	16.58	ksi	Capacity	*Interpolation*
Area required:	Ag =	0.207	in^2	Ag =	0.10	in^2		
Width Required:	brqd =	0.829	in	brqd =	0.407	in		
Min Width	bmin =	2.5	in	bmin =	2.5	in		
Width:	b =	2.5	in	b =	2.5	in		
Length of Bars:	L =	12.3	in	L =	9.8	in		
Length:	L =	12.5	in	L =	12.5	in		

LRFD: **ASD**

1/4	**2 1/2**	12 1/2	in	1/4	**2 1/2**	12 1/2	in	**ANSWER**

PART C: DESIGN OF END TIE PLATES

Minimum length	8 1/2	in	8 1/2	in
Minimum thickness	0.17	in	0.17	in
Minimum Width	11	in	8.5	in

LRFD:				**ASD**				
3/16	8 1/2	12	in2	3/16	8 1/2	12	in2	**ANSWER**

7.5. MATERIAL PROPERTIES:

Modulus of Elasticity	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	F_{y} =	50	ksi
	Fu =	70	ksi

Problem 7.5:

7-5. Determine the available column strength for column AB in the frame shown for which $F_y = 50$ ksi. Otherwise, the conditions are exactly as those described for Prob. 7-3.

a. Assume elastic behavior. (*Ans.* 1095 k, LRFD; 729 k, ASD)

b. Assume inelastic behavior and $P_D = 240$ k and $P_L = 450$ k. (*Ans.* 1098 k, LRFD; 735 k, ASD)

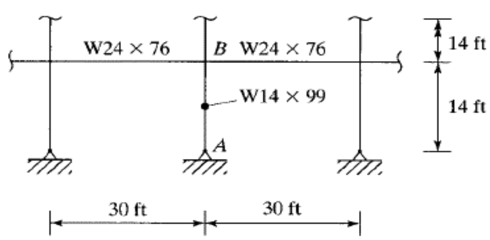


FIGURE P7-5

Required:

a) Available column strength for elastic behavior

b) Available column strength for inelastic behavior

Method:

i) Determine Member Stiffness, Joint Rotational Stiffness and Effective Length Factor

ii) Determine Strength

iii) Determine Stiffness Reduction factor and Strength

Solution:

BRACED-FRAME INFORMATION:

#	Member	Section	Length	I	E	Support end A	Support end B	Stiffness
		(Shape)	*(ft)*	*(in^4)*	*(Msi)*		*(kip.ft)*	*(kip.ft)*
0	No Member	**0**	0	0	0	0	0	0
1	Column AB	**W14X99**	14	1110	29	Pin	Normal	2E+06
2	Column BC	**W14X99**	14	1110	29	Pin	Normal	2E+06

3	Beam BR	W24X76	30	2100	29	Pin	Normal	2E+06
4	Beam BL	W24X76	30	2100	29	Pin	Normal	2E+06

DETERMINATION OF EFFECTIVE LENGTH FACTOR K (BRACED)

#	End	Support	C1	C2	B1	B2	Rotational Stiffness	K			
(#)	*(type)*	*Units*		*(Shape,*	*(ft)*	*(in ^{4})*	*(kip.ft)*				
0	A	Pin					10.00	1.81	App. 7	Fig.	C-A-7.1
	B	Normal	1	0	3	4	0.566				

LRFD			ASD				
1) Demand:			**Demand:**				
	K =	1.81		K =	1.81		Determined above
	rx/ry =	1.66		rx/ry =	1.66		
	KL/r =	15.27 ft		KL/r =	15.27 ft		
Capacity	ϕ_tPn=	1095.00 ft		Pn/Ω=	729.00 ft		**ANSWER** Table 4-1

PART B: INELASTIC DESIGN E

Dead Load	DL=	240	kip		240	kip
Live Load	LL=	450	kip		450	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	

LRFD				ASD			
1) Demand:				**Demand:**			
Load	Pu =	1008	kip		Pa =	690	kip

2. Stiffness Reduction Factor

Member Shape:	W	W14X99		W	W14X99		
	Ag=	29.1	in^2	Ag=	29.1	in^2	
	Fy =	50	ksi	Fy =	50	ksi	
	Py =	1455	kip	Py =	1455	kip	
	α =	1		α =	1.6		
Ratio:	αPu/Py=	0.693		αPu/Py=	0.759		
Factor	τb=	0.851		τb=	0.732		
Rotational Stiffness	G =	0.48		G =	0.41		
	K =	1.79		K =	1.77		Determined above
	rx/ry =	1.66		rx/ry =	1.66		
	KL/r =	15.10	ft	KL/r =	14.93	ft	
Capacity	ϕ_tPn=	1098.00	ft	Pn/Ω=	735.00	ft	**ANSWER** Table 4-1

7.6. MATERIAL PROPERTIES:

Modulus of Elasticity	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	$F_{y\,=}$	50	ksi
	$Fu_{\,=}$	70	ksi

Problem 7.6:

7-6. Repeat Prob. 7-5 if $P_D = 225$ k and $P_L = 375$ k and a W12 × 87 section is used.

Required:

a) Available column strength for elastic behavior
b) Available column strength for inelastic behavior

Method:

i) Determine Member Stiffness, Joint Rotational Stiffness and Effective Length Factor
ii) Determine Strength
iii) Determine Stiffness Reduction factor and Strength

Solution:

BRACED-FRAME INFORMATION:

#	Member	Section	Length	I	E	Support end A	Support end B	Stiffness
		(Shape)	(ft)	(in^4)	(Msi)		(kip.ft)	(kip.ft)
0	No Member	**0**	0	0	0	0	0	0
1	Column AB	**W12X87**	14	740	29	Pin	Normal	2E+06
2	Column BC	**W12X87**	14	740	29	Pin	Normal	2E+06
3	Beam BR	**W24X76**	30	2100	29	Pin	Normal	2E+06
4	Beam BL	**W24X76**	30	2100	29	Pin	Normal	2E+06

DETERMINATION OF EFFECTIVE LENGTH FACTOR K (BRACED)

#	End	Support	C1	C2	B1	B2	Rotational Stiffness	K			
(#)	(type)	Units		(Shape,	(ft)	(in^4)	(kip.ft)				
0	A	Pin					10.00	1.76	App. 7	Fig.	C-A-7.1
	B	Normal	1	0	3	4	0.378				

LRFD ASD

1) Demand: **Demand:**

	LRFD			ASD		
K =	1.76		K =	1.76		Determined above
rx/ry =	1.75		rx/ry =	1.75		
KL/r =	14.08	ft	KL/r =	14.08	ft	
Capacity $\phi_t Pn$=	$\boxed{922.00}$	ft	Pn/Ω=	$\boxed{614.00}$	ft	**ANSWER** Table 4-1

PART B: INELASTIC DESIGN

E

Dead Load	DL=	225	kip		225	kip
Live Load	LL=	375	kip		375	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	

LRFD				ASD		
1) Demand:				**Demand:**		
Load	Pu =	870	kip	Pa =	600	kip

2. Stiffness Reduction Factor

		LRFD			ASD		
Member Shape:	W	**W12X87**		W	**W12X87**		
	Ag=	25.6	in^2	Ag=	25.6	in^2	
	Fy =	50	ksi	Fy =	50	ksi	
	Py =	1280	kip	Py =	1280	kip	
	α =	1		α =	1.6		
Ratio:	αPu/Py=	0.680		αPu/Py=	0.750		
Factor	τb=	0.871		τb=	0.750		
Rotational Stiffness	G =	0.33		G =	0.42		
	K =	1.75		K =	1.74		Determined above
	rx/ry =	1.75		rx/ry =	1.75		
	KL/r =	14.00	ft	KL/r =	13.92	ft	
Capacity	ϕ_tPn=	925.00	ft	Pn/Ω=	617.00	ft	**ANSWER** Table 4-1

7.10. MATERIAL PROPERTIES:

Modulus of Elasticity	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	$F_{y=}$	50	ksi
	$Fu_=$	70	ksi

Problem 7.10:

Use the Effective Length Method, assume elastic behavior, and use both the LRFD and ASD methods. The columns are assumed to have no bending moments.

7-10. Repeat Prob. 7-9, assuming that the outside columns are fixed at the bottom.

Design W14 columns for the bent shown in the accompanying figure, with 50 ksi steel. The columns are braced top and bottom against sidesway out of the plane of the frame so that $K_y = 1.0$ in that direction. Sidesway is possible in the plane of the frame, the x-x axis. Design the interior column as a leaning column, $K_x = K_y = 1.0$ and the exterior columns as a moment frame columns, K_x determined from the alignment chart. (*Ans.* (Interior) W14 × 176, LRFD; W14 × 193, ASD – (Exterior) W14 × 211, LRFD and ASD)

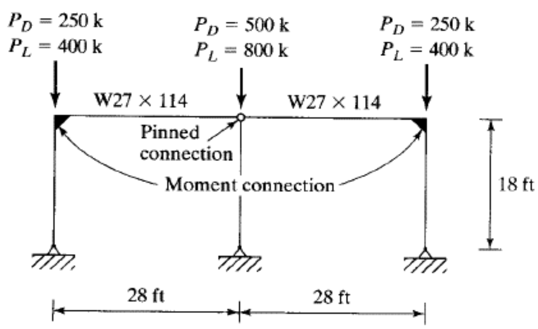


FIGURE P7-9

Required:

a) Design W14 columns for the frame in Figure P7-9 above (interior)
b) and Exterior

Method:

i) Determine Member Stiffness, Joint Rotational Stiffness and Effective Length Factor
ii) Determine Strength
iii) Determine Stiffness Reduction factor and Strength

Solution:

MOMENT-FRAME INFORMATION:

#	Member	Section	Length	I	E	Support end A	Support end B	Stiffness	
		(Shape)	*(ft)*	*(in^4)*	*(Msi)*		*(kip.ft)*	*(kip.ft)*	
0	No Member	**0**	0	0	0	0	0	0	
1	Exterior Column	**W14X176**	18	2140	29	Fixed	Normal	3E+06	LRFD/ASD
2	Interior Column	**W14X176**	18	2140	29	Pin	Normal	3E+06	LRFD
3	Interior Column ASD	**W14X193**	18	2400	29	Pin	Normal	4E+06	ASD
4	Beam BR	**W27X114**	28	4080	29	Pin	Normal	2E+06	*Adjusted for pin end*
5	Beam BL	**W27X114**	28	4080	29	Pin	Normal	2E+06	*Adjusted* page 16.1-514

DETERMINATION OF EFFECTIVE LENGTH FACTOR K (MOMENT)

#	End	Support	C1	C2	B1	B2	Rotational Stiffness	K	
(#)	*(type)*	*Units*		*(Shape,*	*(ft)*	*(in^4)*	*(kip.ft)*		
1	A	Fixed					1.00	1.4	App. 7 Fig. C-A-7.2
	B	Normal	1	0	0	5	1.63	1	
2	A	Pin					10.00	1	Given, x
	B	Normal	2	0	4	5	0.816	1	Given, y
3	A	Pin					10.00	1	Given, x
	B	Normal	3	0	4	5	0.915	1	Given, y

PART A: INTERIOR COLUMN

E

Dead Load	DL=	500	kip		500	kip
Live Load	LL=	800	kip		800	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	

LRFD				ASD				
1) Demand:				**Demand:**				
Load	Pu =	1880	kip	Pa =	1300	kip		**Demand**
	KLy =	18.00	ft	KLy =	18.00	ft		*From factors above*
2) Capacity	ϕ_tPn=	1890.00	ft	Pn/Ω=	1380.00	ft		**Capacity** Table 4-1
3) Design Check:				**Design Check:**				**Design Check**
Strength		OK			OK			
Use	**LRFD:**	**W14X176**		**ASD:**	**W14X193**			**ANSWER**

PART B: EXTERIOR COLUMN

E

Dead Load	DL=	250	kip	DL=	250	kip
Live Load	LL=	400	kip	LL=	400	kip
Leaning Column Loac	LCL =	940	kip	LCL =	650	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	

LRFD				ASD			
1) Demand:				**Demand:**			
Leaning Load	Pu =	1880	kip	Pa =	1300	kip	
Individual Load	Pu =	940	kip	Pa =	650	kip	
Individual Load							
	KLy =	0.00	ft	KLy =	0.00	ft	*From factors above*
2) Capacity	ϕ_tPn=	1890.00	ft	Pn/Ω=	1270.00	ft	**Capacity**
3) Design Check:				**Design Check:**			**Design Check**
Strength		OK			OK		
Leaning Load							
	Kx =	1.40		Kx =	1.40		
	rx/ry =	1.6		rx/ry =	1.6		
Equivalent K	KL/r =	0.00	ft	KL/r =	0.00	ft	
2) Capacity	ϕ_tPn=	1980.00	kip	Pn/Ω=	1317.00	kip	**Capacity** Table 4-1
3) Design Check:				**Design Check:**			**Design Check**
Strength		OK			OK		

Use	LRFD: ft=	ASD: ft=	ANSWER

Problem 7.11:

Required:

a) Design W14 columns for the frame in Figure P7-9 above (interior)
b) and Exterior

Method:

i) Determine Member Stiffness, Joint Rotational Stiffness and Effective Length Factor
ii) Determine Strength
iii) Determine Stiffness Reduction factor and Strength

Solution:

MOMENT-FRAME INFORMATION:

#	Member	Section	Length	I	E	Support end A	Support end B	Stiffness
		(Shape)	*(ft)*	*(in^4)*	*(Msi)*		*(kip.ft)*	*(kip.ft)*
0	No Member	**0**	0	0	0	0	0	0
1	Bottom Column	**W14X109**	15	1240	29	Fixed	Normal	2E+06
2	Top Column	**W14X90**	13	999	29	Pin	Normal	2E+06
3	Beam Bottom	**W18X50**	25	800	29	Moment	Moment	928000
4	Beam Top	**W18X55**	25	890	29	Moment	Moment	1E+06

DETERMINATION OF EFFECTIVE LENGTH FACTOR K (MOMENT)

#	End	Support	C1	C2	B1	B2	Rotational Stiffness	K			
(#)	*(type)*	*Units*		*(Shape,*	*(ft)*	*(in^4)*	*(kip.ft)*				
1	A	Fixed					1.00	1.7	App. 7	Fig.	C-A-7.2
	B	Normal	1	2	0	3	4.98	1	Given, y		

EXTERIOR COLUMN E

Dead Load	DL=	250	kip	DL=	250	kip	Given

Live Load	LL=	500	kip	LL=	500	kip	Given
Leaning Column Load	LCL =	0	kip	LCL =	0	kip	No Leaning Columns
Factors	ϕ_t=	0.9		Ω_t=	1.5		
	ϕ_r=	0.75		Ω_t=	2		

LRFD				ASD			
1) Demand:				**Demand:**			
Leaning Load	Pu =	1100	kip	Pa =	750	kip	
Individual Load	Pu =	1100	kip	Pa =	750	kip	
Individual Load							
	KLy =	15.00	ft	KLy =	15.00	ft	*From factors above*
2) Capacity	ϕ_tPn=	1210.00	kip	Pn/Ω=	808.00	kip	**Capacity** Table 4-1
3) Design Check:				**Design Check:**			**Design Check**
Strength		OK			OK		
Leaning Load							
	Kx =	1.70		Kx =	1.70		
	rx/ry =	1.67		rx/ry =	1.67		
Equivalent K	KL/r =	15.27	ft	KL/r =	15.27	ft	
2) Capacity	ϕ_tPn=	1205.00	kip	Pn/Ω=	803.00	kip	**Capacity** Table 4-1
3) Design Check:				**Design Check:**			**Design Check**
Strength		OK			OK		
Use	**LRFD:**	**W14X109**		**ASD:**	**W14X109**		**ANSWER**

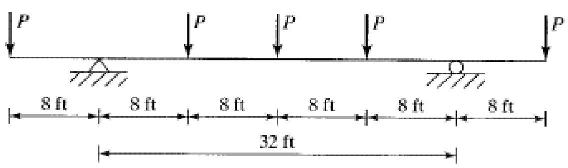
9.25 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
Shear Modulus	G =	11200	ksi
Yield Strength:	F_y =	50	ksi
Ultimate Strength	Fu =	70	ksi

Reference: AISC 14th
Section | *Eq/Fig/Table*

PROBLEM 9.25 | F

9-24. A beam of $F_y = 50$ ksi steel is used to support the loads shown in Fig. P9-24. Neglecting the beam self-weight, determine the lightest W shape to carry the loads if full lateral bracing is provided.



P: $P_D = 8.5$ k, $P_L = 6.0$ k

FIGURE P9-24

9-25. Redesign the beam of Prob. 9-24 if lateral bracing is only provided at the supports and at the concentrated loads. Determine C_b. (*Ans.* W16 × 26 LRFD, W14 × 30 ASD)

Required:

a) Design Beam of Figure P9-24

Method:

i) Determine Beam Flexural Demand
ii) Determine Moment Distribution on the beam
iii) Determine Cb
iv) Determine Beam Flexural Capacity

Solution:

Member Length	L =	38	ft	L =	38	ft
Number of Point Loads		5			5	
Number of Supports		2			2	
Dead Load	DL=	8.5	kip	DL=	8.5	kip
Live Load	LL=	6	kip	LL=	6	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	
Unbraced Length	L_{bx} =	8	ft	L_{bx} =	8	ft

Reference: AISC 14th
Section | *Eq/Fig/Table*

LRFD | ASD | F

1) Demand: | **Demand:**

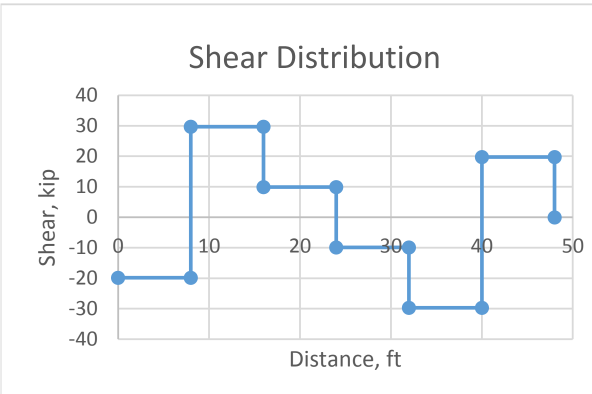
Load	Pu =	19.8	kip		Pa =	14.5	0
Reactions	Ry =	49.5	kip	Reactions	Ry =	36.25	0
	Rx =	0	kip		Rx =	0	0

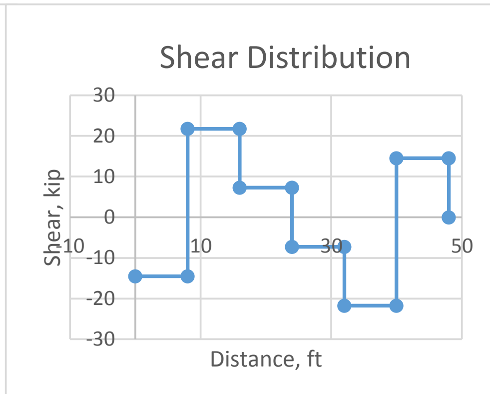
Shear Distribution:

Load	Load Value	Distance	Shear
(#)	(kip)	(ft)	(kip)
1	-19.8	0	-19.8
Ra	49.5	8	29.7
2	-19.8	16	9.9
3	-19.8	24	-9.9
4	-19.8	32	-29.7
Rb	49.5	40	19.8
5	-19.8	48	0

Shear Distribution:

Load	Load Value	d	Shear
(#)	(kip)	(ft)	(kip)
1	-14.5	0	-14.5
Ra	36.25	8	21.75
2	-14.5	16	7.25
3	-14.5	24	-7.25
4	-14.5	32	-21.75
Rb	36.25	40	14.5
5	-14.5	48	0



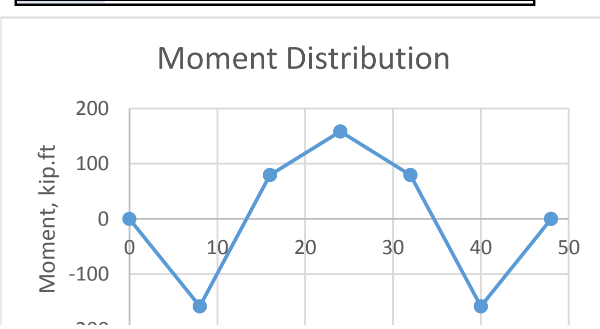


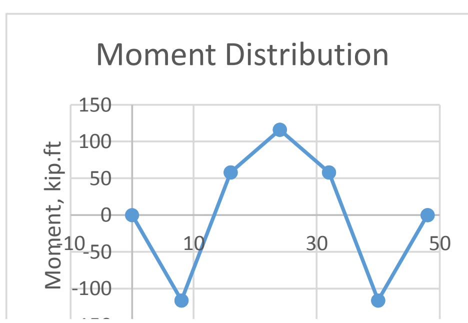
Moment Distribution:

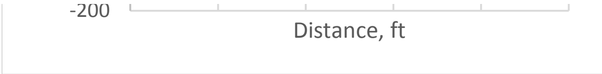
Load	Shear	Distance	Moment
(#)	(kip)	(ft)	(kip.ft)
1	-19.8	0	0
Ra	29.7	8	-158.4
2	9.9	16	79.2
3	-9.9	24	158.4
4	-29.7	32	79.2
Rb	19.8	40	-158.4
5	0	48	0

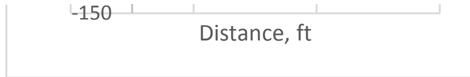
Moment Distribution:

Load	Shear	Distance	Moment
(#)	(kip)	(ft)	(kip.ft)
1	-14.5	0	0
Ra	21.75	8	-116
2	7.25	16	58
3	-7.25	24	116
4	-21.75	32	58
Rb	14.5	40	-116
5	0	48	0









ii) Determine Cb (LRFD)

M_{max} (kip.ft)	$M_{.25}$ (kip.ft)	$M_{.5}$ (kip.ft)	$M_{.75}$ (kip.ft)	C_b
158.4	39.6	79.2	118.8	1.67
158.4	99	39.6	19.8	2.17
158.40	99	118.8	138.6	1.25
158.4	19.8	39.6	99	2.17
158.4	118.8	79.2	39.6	1.67

ii) Determine Cb (ASD)

M_{max} (kip.ft)	$M_{.25}$ (kip.ft)	$M_{.5}$ (kip.ft)	$M_{.75}$ (kip.ft)	C_b
116	29	58	87	1.67
116	72.5	29	14.5	2.17
116	72.5	87	101.5	1.25
116	14.5	29	72.5	2.17
116	87	58	29	1.67

		LRFD		ASD		
USE	C_b =	1.25		1.25		**ANSWER**
Maximum Moment	M_{max} =	158.4	kip.ft	116	kip.ft	
Effective Moment	M_{ueff} =	126.72	kip.ft	92.8	kip.ft	
Unbraced Length	L_{bx} =	8	ft	8	ft	
Beam Selection		**W16X26**		**W14X30**		Table 3-2
Full plastic yield Lengtl	L_p =	3.96	ft	5.26	ft	Table 3-2
LTB Length:	L_r =	11.2	ft	14.9	ft	Table 3-3
	$\phi_b BF$ =	8.98	kips	4.63	kips	Table 3-4
	$\phi_b M_{px}$ =	166	kip.ft	118	kip.ft	Table 3-5
	Zone =	2		2		Table 3-6

	LRFD		ASD		
	$\phi_b M_{nx}$ =	162.2	M_{nx}/Ω_b =	131.6	*Zone 2 Moment*
Check	$\phi_b M_{nx} < \phi_b M_{px}$?	OK	$M_{nx}/\Omega_b < M_{px}/\Omega_b$?	N.G	*Plastic Check*
	$\phi_b M_{nx} > M_u$?	OK	$M_{nx}/\Omega_b > Ma$?	OK	*Check*

USE | **LRFD: W16X26** | **ASD: W16X26** | **ANSWER**

9.32 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
Shear Modulus	G =	11200	ksi
Yield Strength:	F_y =	50	ksi
Ultimate Strength	Fu =	70	ksi

Reference: AISC 14th
Section *Eq/Fig/Table*

PROBLEM **9.32** F

9-32. A W21 × 93 has been specified for use on your design project. By mistake, a W21 × 73 was shipped to the field. This beam must be erected today. Assuming that ½ in thick plates are obtainable immediately, select cover plates to be welded to the top and bottom flanges to obtain the necessary section capacity. Use F_y = 50 ksi steel for all materials and assume that full bracing is supplied for the compression flange. Use LRFD and ASD methods.

Required:

a) Select Plates to be welded on member

Method:

i) Determine information on members
ii) Specify trial plate thickness
iii) Determine plate width

Referenc AISC 14th
Section | *Eq/Fig/Table*
E

Solution:

LRFD				ASD			
1) Demand:				**Demand:**			
Previous Selection:		**W21X93**				**W21X93**	
Plastic Modulus	Z =	221	in^3		Z =	221	in^3
New Section:		**W21X73**				**W21X73**	
Plastic Modulus	Z =	172	in^3		Z =	172	in^3
Depth:	d =	21.2	in		d =	21.2	in
Reinforcement:		**Plates**				**Plates**	
Number of Plates:		2				2	
Enter trial thickness:	t =	1/2	in		t =	1/2	in
Min Width	w_{min} =	4.52	in		w_{min} =	4.52	in
USE	w =	5.00	in		w =	5.00	in

USE | **Plates** | **1/2** | **5.00** | in | | **Plates** | **1/2** | **5.00** | in | **ANSWER**

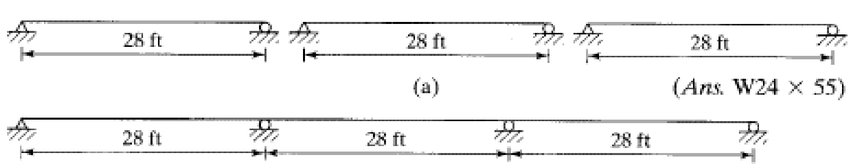
10.09 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
Shear Modulus	G =	11200	ksi
Yield Strength:	F_y =	50	ksi
Ultimate Strength	Fu =	70	ksi

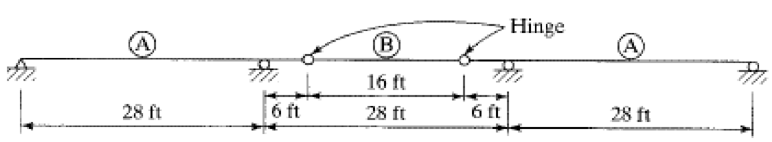
Referenc AISC 14th
Section | *Eq/Fig/Table*
E

PROBLEM 10.09

10-9. Three methods of supporting a roof are shown in Fig. P10-9. Using an elastic analysis with factored loads, F_y = 50 ksi, and assuming full lateral support in each case, select the lightest section if a dead uniform service load (including the beam self-weight) of 1.5 k/ft and a live uniform service load of 2.0 k/ft is to be supported. Consider moment only.



(b) (*Ans.* W21 × 44)



(c) (*Ans.* W16 × 26 (B), W21 × 44 (A))

FIGURE P10-9

Required:

a) Select member when using single span, simply supported beams
b) Select member when using continuous span, simply supported beams
c) Select member when using continuous span, with hinges

Method:

i) Determine load demand on member
ii) Determine largest moment
iii) Enter table to obtain member with moment capacity to support demand

Solution:

Reference: AISC 14th
Section *Eq/Fig/Table*

PART A: SINGLE SPAN, SIMPLY SUPPORTED

F

Member Length	L =	28	ft	L =	28	ft	
Type of Support:		Simply			Simply		
Dead Load	DL=	1.5	kip/ft	DL=	1.5	kip/ft	
Live Load	LL=	2	kip/ft	LL=	2	kip/ft	
Factors	ϕ_t=	0.9		Ω_t=	1.5		
	ϕ_r=	0.75		Ω_t=	2		

LRFD | ASD

1) Demand:				**Demand:**			
Load	Pu =	5	kip/ft	Pa =	3.5	kip/ft	
Moment	Mu =	490	kip.ft	Ma =	343	kip.ft	
1) Capacity:				**Capacity:**			
Beam Selection		**W24X55**			**W21X62**		Table 3-2
Capacity	$\phi_b M_{px}$ =	503	kip.ft	M_{px}/Ω_b =	359	kip.ft	Table 3-2
Check	$\phi_b M_{nx} > M_u$?	OK		M_{nx}/Ω_b > Ma ?	OK		*Check*
USE	**LRFD:**	**W24X55**		**ASD:**	**W24X55**		**ANSWER**

PART B: CONTINUOUS SPAN, SIMPLY SUPPORTED

F

Total Length		84					
Number of Spans		3					
Individual Length	L =	28	ft	40	L =	28	ft
Type of Support:		Case 39				Case 39	

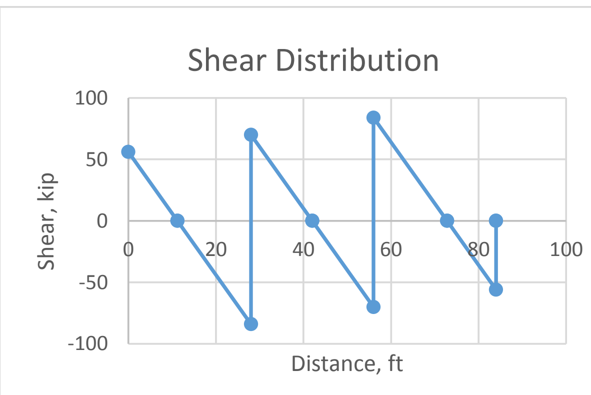
Dead Load	DL=	1.5	kip/ft	DL=	1.5	kip/ft
Live Load	LL=	2	kip/ft	LL=	2	kip/ft
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	

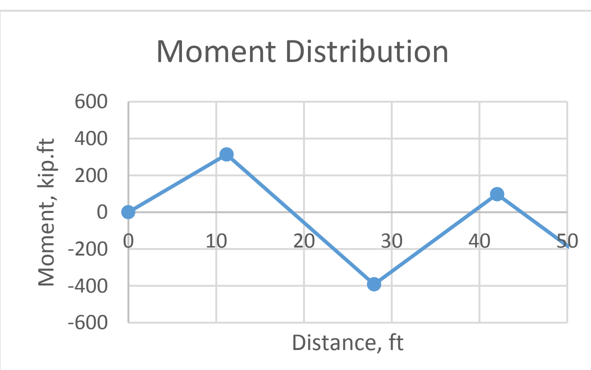
LRFD

1) Demand:

Load   Pu = 5 kip/ft

d (ft)	Reaction (kip)	Shear (kip)	Moment (kip.ft)
0	56	56	0
11.2		0	313.6
28	154	70	-392
42		0	98
56	154	84	-392
72.8		0	313.6
84			
84	56	0	0





Max (-) Moment M_{min} = -392 kip.ft

Max (+) Moment M_{max} = 313.6 kip.ft

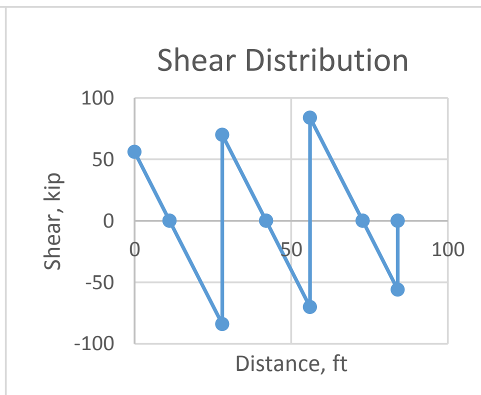
Max absolute   Mu = 392 kip.ft

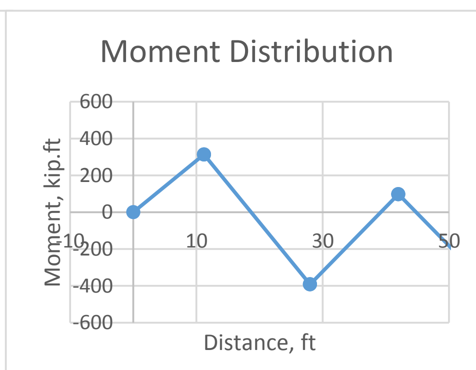
ASD

Demand:

Pa = 3.5 kip/ft

d (ft)	Reaction (kip)	Shear (kip)	Moment (kip.ft)
0	39.2	39.2	0
11.2		0	219.52
28	107.8	49	-274.4
42		0	68.6
56	107.8	58.8	-274.4
72.8		0	219.52
84			
84	39.2	0	0





M_{min} = -274.4 kip.ft

M_{max} = 219.52 kip.ft

Mu = 274.4 kip.ft

1) Capacity:				**Capacity:**				
Beam Selection		**W21X48**			**W21X55**			Table 3-2
Capacity	$\phi_b M_{px}$ =	14.1	kip.ft	M_{px}/Ω_b =	314	kip.ft		Table 3-2
Check	$\phi_b M_{nx} > M_u$?	N.G		M_{nx}/Ω_b > Ma ?	OK		*Check*	
USE	**LRFD:**	**W21X48**		**ASD:**	**W21X55**		**ANSWER**	

PART C: CONTINUOUS SPAN, WITH HINGES IN THE MIDDLE F

Total Length		84	ft			84	ft
Number of Spans		3				3	
Individual Length	L =	28	ft	40	L =	28	ft
Type of Support:		Case 39h				Case 39h	
Distance to hinge		6	ft			6	ft
Distance between hinges		16	ft			16	ft
Dead Load	DL=	1.5	kip/ft		DL=	1.5	kip/ft
Live Load	LL=	2	kip/ft		LL=	2	kip/ft
Factors	ϕ_t=	0.9			Ω_t=	1.5	
	ϕ_r=	0.75			Ω_t=	2	

LRFD | ASD

A - MIDDLE MEMBER:

1) Demand:							
Hinge Member Load	Pu =	80	kip	Pa =	56	kip	
Reaction/Shear:	Vu =	40	kip	Va =	28	kip	
Moment	Mu =	160	kip.ft	Ma =	112	kip.ft	Check Moment

2) Capacity:				**Capacity:**				
Beam Selection		**W16X26**			**W14X30**			Table 3-2
Capacity	$\phi_b M_{px}$ =	166	kip.ft	M_{px}/Ω_b =	118	kip.ft		Table 3-2
Check	$\phi_b M_{nx} > M_u$?	OK		M_{nx}/Ω_b > Ma ?	OK		*Check*	
USE	**LRFD:**	**W16X26**		**ASD:**	**W14X30**		**ANSWER**	

B - END MEMBERS:

1) Demand:				**Demand:**			
Load	Pu =	5	kip/ft		Pa =	3.5	kip/ft

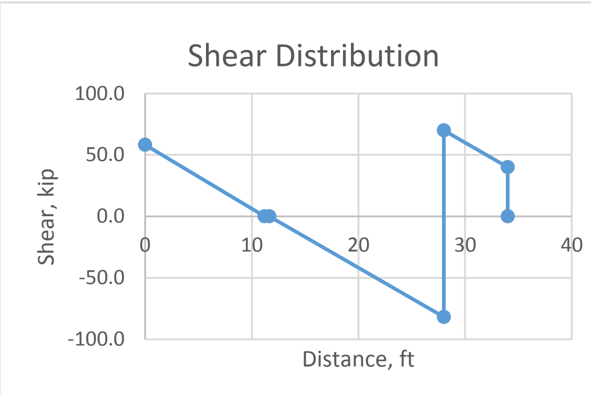
d	Reaction	Shear	Moment

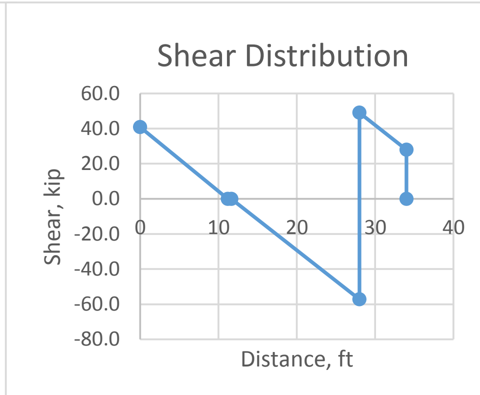
d	Reactior	Shear	Moment

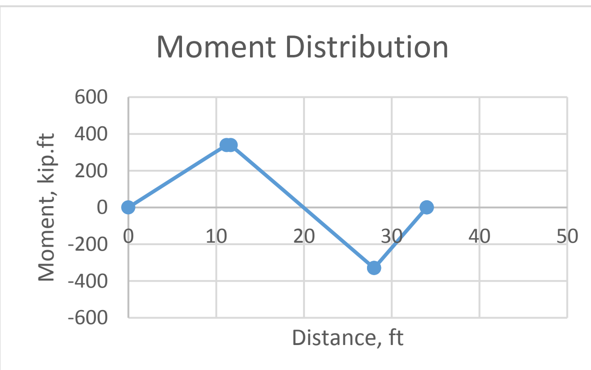
(ft)	*(kip)*	*(kip)*	*(kip.ft)*
0	58.2	58.2	0
11.2		0	338.9
11.64		0	338.9
28	151.8	-81.8	-330
28	151.8	70.0	-330
34		40	0
34		0	0

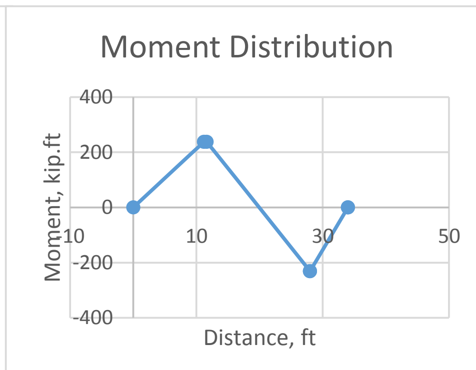
(ft)	*(kip)*	*(kip)*	*(kip.ft)*
0	40.8	40.8	0
11.2		0	237.2
11.64		0	237.2
28	106.3	-57.3	-231
28	106.3	49.0	-231
34		28	0
34		0	0

Distance calculated









Max (-) Moment	M_{min} =	-330	kip.ft	M_{min} =	-231	kip.ft
Max (+) Moment	M_{max} =	338.9	kip.ft	M_{max} =	237.22	kip.ft
Max absolute	Mu =	338.9	kip.ft	Ma =	237.22	kip.ft

1) Capacity: | **Capacity:**

Beam Selection		**W21X44**			**W21X44**		Table 3-2
Capacity	$\phi_b M_{px}$ =	358	kip.ft	M_{px}/Ω_b =	238	kip.ft	Table 3-2
Check	$\phi_b M_{nx} > M_u$?	OK		M_{nx}/Ω_b > Ma ?	OK		*Check*

USE **LRFD: W21X44** **ASD: W21X44** **ANSWER**

10.17 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
Shear Modulus	G =	11200	ksi
Yield Strength:	F_y =	50	ksi

Reference: AISC 14th

Ultimate Strength F_u = 70 ksi

Section | Eq/Fig/Table
F

PROBLEM 10.17

10-17. A 24-ft, simply supported beam must support a moving concentrated service live load of 50 k in addition to a uniform service dead load of 2.5 k/ft. Using 50 ksi steel, select the lightest section considering moments and shear only. Use LRFD and ASD methods and neglect the beam self-weight. (*Ans.* W24 × 76 LRFD and ASD)

Required:

a) Select lightest section considering moments and shear only

Method:

i) Determine load demand on member
ii) Determine largest shear and moment
iii) Enter table to obtain member with moment and shear capacity to support demand

Solution:

Reference: AISC 14th
Section q/Fig/Table
F

PART A: SINGLE SPAN, SIMPLY SUPPORTED

Member Length	L =	24	ft	L =	24	ft	
Type of Support:		Simply			Simply		
Dead Load	DL=	2.5	kip/ft	DL=	2.5	kip/ft	
Live Load	LL=	50	kip	LL=	50	kip	
Factors	ϕ_t=	0.9		Ω_t=	1.5		
	ϕ_r=	0.75		Ω_t=	2		

LRFD				ASD			
1) Demand:				**Demand:**			
Load	Pu =	80	kip	Pa =	50	kip	
Uniform Load	wu =	3	kip/ft	wa =	2.5	kip/ft	
Shear	Vu =	116	kip	Va =	80	kip	
Moment	Mu =	696	kip.ft	Ma =	480	kip.ft	
1) Capacity:				**Capacity:**			
Beam Selection		**W24X76**			**W24X76**		Table 3-2
Capacity	$\phi_b M_{px}$ =	22.4	kip.ft	M_{px}/Ω_b =	76	kip.ft	Table 3-2
	$\phi_v V_{nx}$ =	0		V_{nx}/Ω_v =	0		
Check	$\phi_v V_{nx} > V_u$?	OK		V_{nx}/Ω_v > Va ?	OK		*Check Shear*
	$\phi_b M_{nx} > M_u$?	N.G		M_{nx}/Ω_b > Ma ?	N.G		*Check Flexure*

USE | **LRFD: W24X76** | **ASD: W24X76** | **ANSWER**

10.24 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi
Shear Modulus	G =	11200	ksi	G =	11200	ksi
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi

Reference: AISC 14th

Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi

Section | *Eq/Fig/Table*

PROBLEM **10.24** F

10-23. Select the lightest available W sections (F_y = 50 ksi) for the beams and girders shown in Fig. P10-23. The floor slab is 6 in reinforced concrete (weight = 145 lb/ft^3) and supports a 125 psf uniform live load. Assume that continuous lateral bracing of the compression flange is provided. The maximum permissible TL deflection is L/240. (*Ans.* Beam = W21 × 44 LRFD and ASD, Girder = W24 × 62 LRFD and ASD)

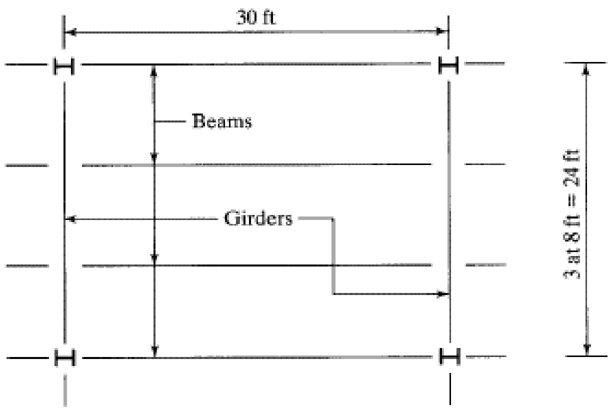


FIGURE P10-23

10-24. Repeat Prob. 10-23 if the live load is 250 psf.

Required:

a) Select lightest section considering moments, shear and TL deflection < L/240

Method:

i) Determine load demand on member
ii) Determine largest shear and moment
iii) Enter table to obtain member with moment and shear capacity to support demand

Solution:

PART A: BEAMS E

Beam Length	L =	30	ft	L =	30	ft
Beam Spacing	s =	8	ft	s =	8	ft
Concrete Weight	yc =	145	pcf	yc =	145	pcf
Concrete Slab Thickness		6	in		6	in
Dead Load	DL=	580	kip	DL=	580	kip
Uniform Live Load	Llu=	250	psf			
Live Load	LL=	2000	kip	LL=	2000	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	

LRFD | ASD

1) **Demand:** | **Demand:**

Load	wu =	3896	lb/ft	wa =	2580	lb/ft	
Demand Values:				**Demand Values:**			
Ultimate Moment,	M_u =	**438.3**	kip.ft	Ma =	**290.3**	kip.ft	
Ultimate Shear,	V_u =	**58.4**	kip	Va =	**38.7**	kip	
Allowed Deflection	Δa =	**1.5**	in	Δa =	**1.5**	in	
Beam Selection,	W:	**W21X55**			**W21X55**		**Capacity**
	ϕM_n	**473.0**	kip.ft	M_{px}/Ω_b =	**314.0**	kip.ft	*AISC 14th* Table 3-2
	ϕV_n	**234.0**	kip	V_{nx}/Ω_v =	**156.0**	kip	*AISC 14th* Table 3-2
Beam Depth:	d	**20.8**	in	d	**20.8**	in	
Moment of Inertia	I_x =	**1140.0**	in^4	I_x =	**1140.0**	in^4	
	I_y =	**48.4**	in^4	I_y =	**48.4**	in^4	
Largest M.I =	Ix =	**1140.0**	in^4	Ix =	**1140.0**	in^4	
Beam Deflection:	ΔTL =	1.42	in	ΔTL =	1.42	in	
Design Check:	$\phi M_n > M_u$?	**YES**		M_{nx}/Ω_b > Ma ?	**YES**		**Design Check**
	$\phi V_n > V_u$?	**YES**		V_{nx}/Ω_v > Va ?	**YES**		
	$\Delta_{TL} < \Delta_a$?	**YES**		$\Delta_{TL} < \Delta_a$?	**YES**		
Use	**LRFD:**	**W21X55**		**ASD:**	**W21X55**		**ANSWER**

PART B: GIRDERS E

Girder Length	Lg =	24	ft	Lg =	24	ft
Tributary Area	At =	192	ft	At =	192	ft

LRFD | ASD

1) Demand:				**Demand:**			
Uniform Load	wu =	3896	lb/ft	wa =	2580	lb/ft	*From beams above*
Load	Pu =	116.88	lb/ft	Pa =	77.4	lb/ft	
Demand Values:				**Demand Values:**			
Ultimate Moment,	M_u =	**935.0**	kip.ft	Ma =	**619.2**	kip.ft	
Ultimate Shear,	V_u =	**116.9**	kip	Va =	**77.4**	kip	
Allowed Deflection	Δa =	**1.2**	in	Δa =	**1.2**	in	
Beam Selection,	W:	**W30X90**			**W30X90**		**Capacity**
	ϕM_n	**1060.0**	kip.ft	M_{px}/Ω_b =	**706.0**	kip.ft	*AISC 14th* Table 3-2
	ϕV_n	**374.0**	kip	V_{nx}/Ω_v =	**249.0**	kip	*AISC 14th* Table 3-2
Beam Depth:	d	**29.5**	in	d	**29.5**	in	
Moment of Inertia	I_x =	**3610.0**	in^4	I_x =	**3610.0**	in^4	
	I_y =	**115.0**	in^4	I_y =	**115.0**	in^4	

Largest M.I =	Ix =	3610.0	in^4	Ix =	3610.0	in^4	
Beam Deflection:	ΔTL =	0.00	in	ΔTL =	0.61	in	

Design Check:					
Flexure:	$\phi M_n > M_u$?	**YES**	M_{nx}/Ω_b > Ma ?	**YES**	**Design Check**
Shear:	$\phi V_n > V_u$?	**YES**	V_{nx}/Ω_v > Va ?	**YES**	
Deflection:	$\Delta_{TL} < \Delta_a$?	**YES**	$\Delta_{TL} < \Delta_a$?	**YES**	

Use	**LRFD: W30X90**	**ASD: W30X90**	**ANSWER**

Check with Self-Weight

Ultimate Moment,	M_u =	**942.8**	kip.ft	Ma =	**627.0**	kip.ft
Ultimate Shear,	V_u =	**118.2**	kip	Va =	**78.7**	kip
Allowed Deflection	Δa =	**0.01**	in	Δa =	**0.62**	in

Design Check:	$\phi M_n > M_u$?	**YES**	M_{nx}/Ω_b > Ma ?	**YES**	**Design Check**
	$\phi V_n > V_u$?	**YES**	V_{nx}/Ω_v > Va ?	**YES**	
	$\Delta_{TL} < \Delta_a$?	**YES**	$\Delta_{TL} < \Delta_a$?	**YES**	

Use	**LRFD: W30X90**	**ASD: W30X90**	**ANSWER**

10.27 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi	
Shear Modulus	G =	11200	ksi	G =	11200	ksi	
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi	
Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi	Reference: AISC 14th
							Section *q/Fig/Table*
PROBLEM	**10.27**						F

10-27. The beam shown in Fig. P10-27 is a W14 × 34 of A992 steel and has lateral support of the compression flange at the ends and at the points of the concentrated loads. The two concentrated loads are service live loads. Check the beam for shear and for Web Local Yielding and Web Crippling at the concentrated load if l_b = 6 in. Neglect the self-weight of the beam. (*Ans.* Shear and web crippling N.G., web local yielding OK)

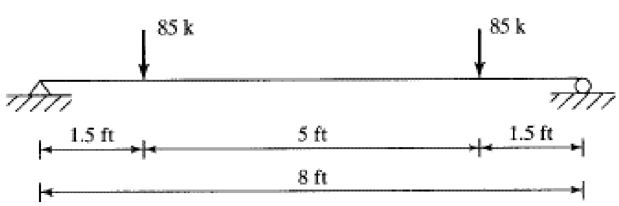


FIGURE P10-27

Required:

a) Select lightest section considering moments, shear
b) Web Local Yielding and Crippling at the concentrated load

Method:

i) Determine load demand on member
ii) Determine largest shear and moment
iii) Enter table to obtain member with moment and shear capacity to support demand

Solution:

BEAM SHEAR, MOMENT & DEFLECTION E

Beam Length	L =	8	ft	L =	8	ft
x, Point Load:	Lx =	1.5	ft	Lx =	1.5	
Dead Load	DL =	0	kip	DL =	0	
Live Load	LL=	85	kip	LL=	85	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	
Bearing Length:	lb =	6	in	lb =	6	in

LRFD				ASD			
1) Demand:				**Demand:**			
Load	Pu =	136	kip	Pa =	85	kip	
Demand Values:				**Demand Values:**			
Ultimate Moment,	M_u =	**204.0**	kip.ft	Ma =	**127.5**	kip.ft	
Ultimate Shear,	V_u =	**136.0**	kip	Va =	**85.0**	kip	
Allowed Deflection	Δa =	**0.4**	in	Δa =	**0.4**	in	
Beam Selection,	**W:**	**W14X34**			**W14X34**		**Capacity**
	ϕM_n	**205.0**	kip.ft	$M_{px/}\Omega_b$ =	**136.0**	kip.ft	*AISC 14th* Table 3-2
	ϕV_n	**120.0**	kip	$V_{nx/}\Omega_v$ =	**79.8**	kip	*AISC 14th* Table 3-2
Beam Depth:	d	**14.0**	in	d	**14.0**	in	
Moment of Inertia	I_x =	**340.0**	in^4	I_x =	**340.0**	in^4	
	I_y =	**23.3**	in^4	I_y =	**23.3**	in^4	
Largest M.I =	Ix =	**340.0**	in^4	Ix =	**340.0**	in^4	
Beam Deflection:	ΔTL =	0.00	in	ΔTL =	0.00	in	*Case 9* Pg 3-215
Design Check:							
Flexure:	$\phi M_n > M_u$?	**YES**		$M_{nx/}\Omega_b$ > Ma ?	**YES**		**Design Check**
Shear:	$\phi V_n > V_u$?	**YES**		$V_{nx/}\Omega_v$ > Va ?	**NO**		**ANSWER**
Deflection:	$\Delta_{TL} < \Delta_a$?	**YES**		$\Delta_{TL} < \Delta_a$?	**YES**		
Use	**LRFD: W14X34**			**ASD: W14X34**			**ANSWER**

BEAM: WEB LOCAL YIELDING & WEB CRIPPLING E

Web Local Yielding

Factor	ϕ=	1		Ω=	1.5		
Web Yield Strength:	Fyw =	50	ksi	Fyw =	50	ksi	
Beam Selection,	**W:**	**W14X34**			**W14X34**		
Beam Depth:	d	**14.0**	in	d	**14.0**	in	
Moment of Inertia	k =	**0.855**	in	k =	**0.855**	in	
Thickness of Web	t_w =	**0.285**	in	t_w =	**0.285**	in	
Thickness of Flange	t_f =	0.455	in	t_f =	0.455	in	
Bearing Length:	N =	6	in	N =	6	in	
Location of P load		18	in		18	in	
Location x > beam d ?		Yes			Yes		
Web Yield Capacity							
	Rn =	146.42		Rn =	146.42		5*k+N * Fyw * tw
	ϕR_n =	146.42		R_n/Ω =	97.61		
Check	Rn < Vu ?	YES		Rn < Va ?	YES		**ANSWER**

Web Crippling

Factor	ϕ_r=	0.75		Ω_t=	2		
Location of P load		18	in		18	in	
Location x > beam d/2 ?		Yes			Yes		
Web Resistance Capacity							
	Rn =	161.88		Rn =	161.88		.8*tw^2
Web Res. Strength	ϕR_n =	121.41		R_n/Ω =	80.94		
Check	Rn < Vu ?	N.G.		Rn < Va ?	N.G.		**ANSWER**

10.28 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi	
Shear Modulus	G =	11200	ksi	G =	11200	ksi	
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi	
Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi	Reference: AISC 14th
							Section *q/Fig/Table*

PROBLEM **10.28** F

10-28. A 7-ft beam with full lateral support for its compression flange is supporting a moving concentrated live load of 58 k. Using 50 ksi steel, select the lightest W section. Assume the moving load can be placed anywhere in the middle 5 ft of the beam span. Choose a member based on moment then check if it is satisfactory for shear, and compute the minimum length of bearing required at the supports from the standpoint of web local yielding and web crippling. Neglect self-weight.

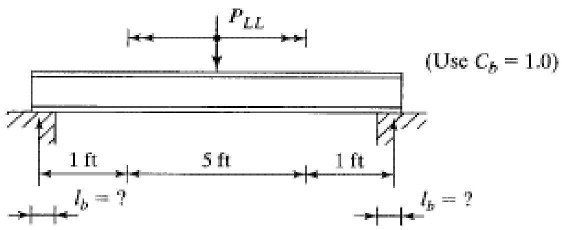


FIGURE P10-28

Required:

a) Select lightest section considering moments, shear
b) Web Local Yielding and Crippling at the concentrated load

Method:

i) Determine load demand on member
ii) Determine largest shear and moment
iii) Enter table to obtain member with moment and shear capacity to support demand

Solution:

BEAM SHEAR, MOMENT & DEFLECTION E

Beam Length	L =	7	ft	L =	7	ft	
x, Point Load:	Lx =	3.5	ft	Lx =	3.5		
Dead Load	DL =	0	kip	DL =	0		
Live Load	LL=	58	kip	LL=	58	kip	
Factors	ϕ_t=	0.9		Ω_t=	1.5		
	ϕ_r=	0.75		Ω_t=	2		
Bearing Length:	Ib =	12	in	Ib =	6	in	

LRFD | ASD

1) Demand: | **Demand:**

Load	Pu =	92.8	kip	Pa =	58	kip	
Demand Values:				**Demand Values:**			
Ultimate Moment,	M_u =	**162.4**	kip.ft	Ma =	**101.5**	kip.ft	*Max @ Mid-Span*
Ultimate Shear,	V_u =	**79.5**	kip	Va =	**49.7**	kip	*Max closest to edge*
Allowed Deflection	Δa =	**0.4**	in	Δa =	**0.4**	in	
Beam Selection,	**W:**	**W16X26**			**W16X26**		**Capacity**
	ϕM_n	**166.0**	kip.ft	M_{px}/Ω_b =	**110.0**	kip.ft	*AISC 14th* Table 3-2
	ϕV_n	**106.0**	kip	V_{nx}/Ω_v =	**70.5**	kip	*AISC 14th* Table 3-2
Beam Depth:	d	**15.7**	in	d	**15.7**	in	

Moment of Inertia	I_x =	**301.0**	in^4	I_x =	**301.0**	in^4	
	I_y =	**9.6**	in^4	I_y =	**9.6**	in^4	
Largest M.I =	Ix =	**301.0**	in^4	Ix =	**301.0**	in^4	
Beam Deflection:	ΔTL =	0.01	in	ΔTL =	0.01	in	*Case 9* Pg 3-215

Design Check:					
Flexure:	$\phi M_n > M_u$?	**YES**	M_{nx}/Ω_b > Ma ?	**YES**	**Design Check**
Shear:	$\phi V_n > V_u$?	**YES**	V_{nx}/Ω_v > Va ?	**YES**	**ANSWER**
Deflection:	$\Delta_{TL} < \Delta_a$?	**YES**	$\Delta_{TL} < \Delta_a$?	**YES**	

Use **LRFD: W16X26** **ASD: W16X26** **ANSWER**

BEAM: WEB LOCAL YIELDING & WEB CRIPPLING

E

Web Local Yielding

Factor	ϕ=	1		Ω=	1.5		
Web Yield Strength:	Fyw =	50	ksi	Fyw =	50	ksi	
Beam Selection,	**W:**	**W16X26**			**W16X26**		
Beam Depth:	d	**15.7**	in	d	**15.7**	in	
Moment of Inertia	k =	**0.747**	in	k =	**0.747**	in	
Thickness of Web	t_w =	**0.250**	in	t_w =	**0.250**	in	
Thickness of Flange	t_f =	0.345	in	t_f =	0.345	in	
Bearing Length:	N =	2.65	in	N =	2.24	in	**ANSWER**
Location of P load		42	in		42	in	
Location x > beam d ?		Yes			Yes		
Web Yield Capacity							
	Rn =	79.81	kip	Rn =	74.69	kip	*Goal Seek*
	ϕR_n =	79.81	kip	R_n/Ω =	49.79	kip	
Check	Rn > Vu ?	YES		Rn < Va ?	YES		**ANSWER**

Web Crippling

Factor	ϕ_r=	0.75		Ω_t=	2		
Location of P load		42	in		42	in	
Location x > beam d/2 ?		Yes			Yes		
Bearing Length:	N =	4.25	in	N =	3.45	in	**ANSWER**

Web Resistance Capacity

	Rn =	106.16	kip	Rn =	99.49	kip	.8*tw^2
Web Res. Strength	ϕR_n =	79.62	kip	R_n/Ω =	49.75	kip	*Goal Seek*
Check	Rn < Vu ?	OK		Rn < Va ?	OK		**ANSWER**

USE | W16X26 Nmin = 4.25 in | W16X26 Nmin = 3.5 in | **ANSWER**

10.30 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi
Shear Modulus	G =	11200	ksi	G =	11200	ksi
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi
Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi

Reference: AISC 14th
Section *Eq/Fig/Table*

PROBLEM **10.30** F

10-30. A W21 × 68 member is used as a simply supported beam with a span length of 12 ft. Determine C_b, since the lateral support of the compression flange is provided only at the ends. The member is uniformly loaded. The loads will produce factored moments of M_{Dx} = 75 ft-k, M_{Lx} = 90 ft-k and M_{Dy} = 15 ft-k, M_{Ly} = 18 ft-k. Is this member satisfactory for bending strength based on the interaction equation in Chapter H of the AISC Specification?

Required:

a) Determine Cb for the member

Method:

i) Determine Member Demand

ii) Determine Member Capacity

Solution:

Reference: AISC 14th
Section *Eq/Fig/Table*

PART A: SINGLE SPAN, SIMPLY SUPPORTED F

Member Length	L =	12	ft	L =	12	ft
Type of Support:		Simply			Simply	
Dead Moment, x	DLx=	75	kip/ft	DLx=	75	kip/ft
Live Moment, x	LLx=	90	kip/ft	LLx=	90	kip/ft
Dead Moment, y	DLy=	15	kip/ft	DLy=	15	kip/ft
Live Moment, y	LLy=	18	kip/ft	LLy=	18	kip/ft
Factors	ϕ_t=	0.9		Ω_t=	1.669	
	ϕ_r=	0.75		Ω_t=	2	

LRFD | ASD

1) Demand:				**Demand:**		
Moment, x	Mu =	234	kip/ft	Ma =	165	kip/ft
Moment, y	Muy =	46.8	kip/ft	May =	33	kip/ft

1) Capacity: | **Capacity:**

Plastic Zones Lengths and Info:

		LRFD			ASD		
Beam Selection		**W21X68**			**W21X68**		Table 3-2
Full plastic yield Length	L_p =	6.36	ft	L_p =	6.36	ft	
LTB Length:	L_r =	18.7	ft	L_r =	18.7	ft	
	$\phi_b BF$ =	18.8	kips	$\phi_b BF$ =	18.8	kips	
Capacity	$\phi_b M_{px}$ =	600	kip.ft	M_{px}/Ω_b =	399	kip.ft	Table 3-2
	Fy.Zy =	101.67	kip.ft	Fy.Zy =	101.67	kip.ft	
	1.6Fy.Sy =	104.67	kip.ft	1.6Fy.Sy =	104.67	kip.ft	
	M_{cy} =	91.5	kip.ft	M_{cy} =	60.9	kip.ft	
Check	$\phi_b M_{nx} > M_u$?	**OK**		M_{nx}/Ω_b > Ma ?	**OK**		*Check*
Determine Cb:							
Uniform Load	Cb =	1.14		Cb =	1.14		Table 3-1
	Zone =	2		Zone =	2		
	ϕM_{nx} =	563.12		ϕM_{nx} =	374.49		*BF Equation*
Check	$\phi_b M_{nx} <=$ Mpx ?	**OK**		M_{nx}/Ω_b < Mpx ?	**OK**		*Check*
Equation H1-1b	Ratio =	0.93		Ratio =	0.98		H Eq H1-1b
Check	Eq H1-1b < 1	OK		Eq H1-1b < 1	OK		*Check*

USE **LRFD: W21X68** **ASD: W21X68** **ANSWER**

10.31 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi	
Shear Modulus	G =	11200	ksi	G =	11200	ksi	
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi	
Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi	Reference: AISC 14th

Section *Eq/Fig/Table*

PROBLEM **Capacity** F

10-31. The 30-ft, simply supported beam shown in Fig. P10-31 has full support of its compression flange and is A992 steel. The beam supports a gravity service dead load of 132 lb/ft (includes beam weight) and gravity live load of 165 lb/ft. The loads are assumed to act through the c.g. of the section. Select the lightest available W10 section. (*Ans.* W10 × 22 LRFD, W10 × 26 ASD)

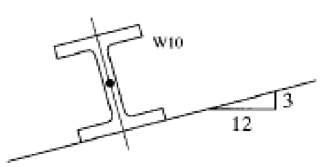


FIGURE P10-31

Required:

a) Determine lightest W10 section

Method:

i) Determine Member Demand

ii) Determine Member Capacity

Solution:

Reference: AISC 14th
Section *Eq/Fig/Table*

PART A: SINGLE SPAN, SIMPLY SUPPORTED

F

Member Length	L =	30	ft	L =	30	ft	
Type of Support:		Simply			Simply		
Dead Load	DL=	132	lb/ft	DLx=	132	lb/ft	
Live Load	LL=	165	lb/ft	LLx=	165	lb/ft	
Factors	ϕ_t=	0.9		Ω_t=	1.669		
	ϕ_r=	0.75		Ω_t=	2		
Datum Rise		3			3		
Datum Run		12			12		
Slope		12.37			12.37		

LRFD | ASD

1) Demand:				**Demand:**				
Load	wu =	422.40	lb/ft		wa =	297.00	lb/ft	
Load in local x	wux =	409.79	lb/ft		wax =	288.13	lb/ft	
Load in local y	wuy =	102.45	lb/ft		way =	72.03	lb/ft	
Moment, x	Mu =	46.10	k.ft		Ma =	32.41	k.ft	
Moment, y	Muy =	11.53	k.ft		May =	8.10	k.ft	
1) Capacity:				**Capacity:**				
Plastic Zones Lengths and Info:								
Beam Selection		W10X22				W10X26		Table 3-2
Unbraced Length	Lb =	0	ft		Lb =	0	ft	
Full plastic yield Lengtl	L_p =	4.7	ft		L_p =	4.8	ft	Table 3-2
LTB Length:	L_r =	13.8	ft		L_r =	14.9	ft	Table 3-2
	ϕ_bBF =	4.02	kips		ϕ_bBF =	4.34	kips	Table 3-2
Compression Zone	Zone =	1			Zone =	1		
Zone 1 Capacity	ϕM_{nx} =	97.50			ϕM_{nx} =	78.14		Zone 1
Zone 2 Capacity	$\phi_b M_{px}$ =	97.5	kip.ft		M_{px}/Ω_b =	78.1	kip.ft	Zone 2 Table 3-2
Along y axis	Fy.Zy =	25.42	kip.ft		Fy.Zy =	31.25	kip.ft	

	1.6Fy.Sy =	26.47	kip.ft	1.6Fy.Sy =	32.60	kip.ft	
Y axis capacity	M_{cy} =	22.9	kip.ft	M_{cy} =	18.7	kip.ft	*Capacity in y*
Check	$\phi_b M_{nx} > M_u$?	**OK**		M_{ny}/Ω_b > Ma ?	**OK**		*Check*
	$\phi_b M_{nx} <=$ Mpx ?	**OK**		M_{ny}/Ω_b < Mpy ?	**OK**		*Check*
	$\phi_b M_{ny} > M_u$?	**OK**		M_{ny}/Ω_b > Ma ?	**OK**		*Check*
	$\phi_b M_{ny} <=$ Mpy ?	**OK**		M_{ny}/Ω_b < Mpy ?	**OK**		*Check*
Equation H1-1b	Ratio =	0.98		Ratio =	0.85		H Eq H1-1b
Check	Eq H1-1b < 1	OK		Eq H1-1b < 1	OK		*Check*

USE **LRFD: W10X22** **ASD: W10X26** **ANSWER**

10.32 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi	
Shear Modulus	G =	11200	ksi	G =	11200	ksi	
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi	
Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi	Reference: AISC 14th
							Section *q/Fig/Table*

PROBLEM **10.32** F

10-32. Design a steel bearing plate from A572 (Grade 50) steel for a W18 × 35 beam, with end reactions of $R_D = 12$ k and $R_L = 16$ k. The beam will bear on a reinforced concrete wall with $f'_c = 3$ ksi. In the direction perpendicular to wall, the bearing plate maximum length of end bearing may not be longer than 6 in. W18 is A992 steel.

Required:

a) Bearing Plate for concrete wall

Method:

i) Determine Member Demand

ii) Determine Member Capacity

Solution:

Beam Length	L =	30	ft	L =	30	ft
Concrete Strength	fc' =	3	ksi	fc' =	3	ksi
Dead Load	DL=	12	kip	DL=	12	kip
Live Load	LL=	16	kip	LL=	16	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	
	ϕ_c=	0.6		Ω_c=	2.5	

LRFD ASD

1) Demand:

Demand:				**Demand:**				
Load	wu =	40	kip		wa =	28	kip	**LOAD**
Bearing Plate		**Plates**				**Plates**		**BEARING PLATE**
Number of Plates:		1				1		
Area of Plate:	A =	26.14	in^2		A =	27.45	in^2	
Enter trial length:	l =	7	in		l =	7	in	
Min Width	w_{min} =	3.73	in		w_{min} =	3.92	in	
USE	N =	4.00	in		N =	4.00	in	**BEARING LENGTH**
Check Limit	N < 6 in ?	**OK**			N < 6 in ?	**OK**		
Plate Thickness	h =	2.67						
	tmin =	**0.67**	in					
Use Thickness:	t =	**0.75**	in					
USE	**Plates** 7	**4.00**	in		**Plates** 7	**4.00**	in	**ANSWER**
	thickness	0.75	in		thickness	0.00	0	

BEAM: WEB LOCAL YIELDING & WEB CRIPPLING

E

Web Local Yielding

Factor	ϕ=	1		Ω=	1.5		
Web Yield Strength:	Fyw =	50	ksi	Fyw =	50	ksi	
Beam Selection,	W:	**W18X35**			**W18X35**		
Beam Depth:	d	**17.7**	in	d	**17.7**	in	
Moment of Inertia	k =	**0.827**	in	k =	**0.827**	in	
Thickness of Web	t_w =	**0.300**	in	t_w =	**0.300**	in	
Thickness of Flange	t_f =	0.425	in	t_f =	0.425	in	
Bearing Length:	N =	4.00	in	N =	4.00	in	

Web Yield Capacity

	Rn =	91.01	kip	Rn =	91.01	kip	5*k+N * Fyw * tw
	ϕR_n =	91.01	kip	R_n/Ω =	60.68	kip	
Check	Rn < Vu ?	YES		Rn < Va ?	YES		**ANSWER**

Web Crippling

Factor	ϕ_r=	0.75		Ω_t=	2		

Web Resistance Capacity

	Rn =	72.34	kip	Rn =	72.34	kip	.8*tw^2
Web Res. Strength	ϕR_n =	54.26	kip	R_n/Ω =	36.17	kip	
Check	Rn < Vu ?	OK		Rn < Va ?	OK		**ANSWER**

1. BUILDING LOADS AS REQUIRED BY CODE:

Loads are in accordance to:	2012	IBC	
Modified by:	Massachusetts Building Code		(CMR780)
Snow	Lowell	MA	
Wind	Lowell	MA	
Seismic	Lowell	MA	
Construction Live Load:	20	psf	
Uniform Live Load:	100	psf	

2.1 BUILDING VERTICAL LOADS:

	Roof		1st Floor	
2.1. Dead Load (D)	26.0	psf	128	psf
2.2. Live Load (L)		psf	100	psf
2.3. Roof Live Load (Lr)	20.0	psf		psf
2.4. Snow Load (S)	31.5	psf		psf
2.5. Rain Load (R)		psf		psf
2.6. Seismic Load (E)		psf		psf
2.7. Wind Load (W)	-23.0	psf		psf
Total / Service Load:	54.5	psf	228	psf

2.2 LOAD COMBINATIONS PER LRFD SPECIFICATIONS:

	Roof		1st Floor	
1. 1.4D	36.4	psf	179.2	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	47.0	psf	313.6	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)	70.1	psf	253.6	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	24.0	psf	153.6	psf
5. 1.2D + 1.0E + L + .2S	37.5	psf	253.6	psf
6. 0.9D + 1.0W	0.4	psf	115.2	psf
7. 0.9D + 1.0E	23.4	psf	115.2	psf
Controlling Load:	70.1	psf	313.6	psf

3.1 BUILDING LATERAL LOAD ON LONGITUDINAL DIRECTION: BRACED-FRAME

	Roof		1st Floor	
2.1. Dead Load (D)		psf		psf
2.2. Live Load (L)		psf		psf
2.3. Roof Live Load (Lr)		psf		psf
2.4. Snow Load (S)		psf		psf
2.5. Rain Load (R)		psf		psf
2.6. Seismic Load (E)	20.8	psf	20.4	psf
2.7. Wind Load (W)	6.4	psf	12.9	psf
Total / Service Load:	27.2	psf	33.3	psf

3.2 LOAD COMBINATIONS PER LRFD SPECIFICATIONS:

	Roof		1st Floor	
1. 1.4D	0.0	psf	0.0	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	0.0	psf	0.0	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)	3.2	psf	6.4	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	6.4	psf	12.9	psf
5. 1.2D + 1.0E + L + .2S	20.8	psf	20.4	psf
6. 0.9D + 1.0W	6.4	psf	12.9	psf
7. 0.9D + 1.0E	20.8	psf	20.4	psf
Controlling Load:	20.8	psf	20.4	psf

4.1 BUILDING LATERAL LOAD ON TRANSVERSE DIRECTION: MOMENT-FRAME

	Roof		1st Floor	
2.1. Dead Load (D)		psf		psf
2.2. Live Load (L)		psf		psf
2.3. Roof Live Load (Lr)		psf		psf
2.4. Snow Load (S)		psf		psf
2.5. Rain Load (R)		psf		psf
2.6. Seismic Load (E)	17.6	psf	17.0	psf
2.7. Wind Load (W)	9.8	psf	19.6	psf
Total / Service Load:	27.3	psf	36.6	psf

4.2 LOAD COMBINATIONS PER LRFD SPECIFICATIONS:

	Roof		1st Floor	
1. 1.4D	0.0	psf	0.0	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	0.0	psf	0.0	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)	4.9	psf	9.8	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	9.8	psf	19.6	psf
5. 1.2D + 1.0E + L + .2S	17.6	psf	17.0	psf
6. 0.9D + 1.0W	9.8	psf	19.6	psf
7. 0.9D + 1.0E	17.6	psf	17.0	psf
Controlling Load:	17.6	psf	19.6	psf

Reference:	ASCE	7-10
Section	*Eq/Fig/Table/Notes*	

DEAD LOAD 3

Floor:	Roof					2nd				
	Item	**Quantity** *(Area)*	**Units**	**Unit Weight** *(ksf or klf)*	**Weight** *(kip)*	**Item**	**Quantity** *(Area)*	**Units**	**Unit Weight** *(ksf or klf)*	**Weight** *(kip)*
						Concrete Slab	7776	*sf*	0.075	583
	Metal Deck	7776	*sf*	0.014	109	**Metal Deck 18 g.a**	7776	*sf*	0.014	109
	EPDM Membrane	7776	*sf*	0.001	8	**Cladding**	7776	*sf*	0.01	78
	Insulation	7776	*sf*	0.006	47	**Partitions**	7776	*sf*	0.01	78
	Mechanical Equipment	7776	*sf*	0.005	39	**Mechanical Equipment**	7776	*sf*	0.007	54
						Steel Structure	7776	*sf*	0.012	93
Subtotal	*202*			0.026	202	*995*			0.128	995
Cummulative	*202*				202	*1198*				**1198**

***Unit Weights per ASCE 7-10**

Reference:	ASCE	7-10
Section	*Eq/Fig/Table/Notes*	

SNOW LOAD 7

Exposure Factor	C_e =	0.90		Table	7-2
Thermal Factor	C_t =	1.00		Table	7-3
Importance Factor	I_s =	1.00		Table	1.5-2
Ground Snow Load	ρ_g =	50.00	psf	Figure	7-1
Flat Roof Snow Load	ρ_f =	31.50	psf	Eq	7.3-1

Reference: ASCE 7-10

Section	*Eq/Fig/Table/Notes*
12	

SEISMIC LOAD

Number of Floors: 2

Floor:	**Roof**					**2nd**				
	Item	**Quantity** *(Area)*	**Units**	**Unit Weight** *(ksf or klf)*	**Weight** *(kip)*	**Item**	**Quantity** *(Area)*	**Units**	**Unit Weight** *(ksf or klf)*	**Weight** *(kip)*
						Concrete Slab	7776	*sf*	0.075	583
	Metal Deck	7776	*sf*	0.014	109	**Metal Deck**	7776	*sf*	0.014	109
	EPDM Membrane	7776	*sf*	0.001	8	**Cladding**	7776	*sf*	0.01	78
	Insulation	7776	*sf*	0.006	47	**Partitions**	7776	*sf*	0.01	78
	Mechanical Equipment	7776	*sf*	0.005	39	**Mechanical Equipment**	7776	*sf*	0.007	54
	Snow	7776	*sf*	0.0315	245	**Steel Structure**	7776	*sf*	0.012	93
Subtotal	*447*				**447**	*995*				**995**
Cummulative	*447*				**447**	*1442*				**1442**
Mass (kip*s^2/ft)					107					58

WIND LOAD ANALYSIS

				Reference:	ASCE	7-10
				Section	*Eq/Fig/Table/Notes*	
1. BUILDING INFORMATION RELATED TO WIND LOAD ANALYSIS				26/27/28		
Mean roof height	H_{roof} =	30	ft	*Height of heighest level of structure*		
Floor-Floor Height	h_n =	15	ft			
Building Length	L =	112	ft			
Building Width	W =	76	ft			
Number of Braces/Level		2				
Number of Moment Frames/ Level		2				
2. WIND EXPOSURE, ROUGHNESS AND OCCUPANCY CATEGORY				26.4		
Occupancy Category:		B			Table	1-1
Ground Surface Roughness:		B				26.7.2
Exposure Category:		C				26.7.2
3. ENVIRONMENTAL CHARACTERISTICS AND FACTORS				26.5		
Wind Speed	V =	120	mph	26.5	Figure	26.5-1A
Zone A	P_{s30} =	22.8	psf		Figure	28.6-1
Zone C	P_{s30} =	15.1	psf		Figure	28.6-1
	a_1 =	7.6	ft	*.1*W*		
	a_2 =	12	ft	*.4*H_{roof}*		
	a =	7.6	ft	*Min Value*		
	2.a =	15.2	ft			
Weighted Average for P_{s30}:						
Longitudinal		16.1	psf			
Transverse		16.6	psf			
4. DESIGN WIND PRESSURE				26.8		
Adjustment Factor	λ =	1.4			Figure	28.6-1
	K_{zt} =	1		*26.8*		
Design wind pressure,	$P_{s\text{-longitudinal}}$ =	22.6	kip			
Design wind pressure,	$P_{s\text{-transverse}}$ =	23.3	kip			
5. LOAD APPLIED TO EACH LEVEL				26.8		
Roof	$F_{u\text{-longitudinal}}$ =	12.9	kip		Figure	28.6-1
	$F_{u\text{-transverse}}$ =	19.6	kip	*26.8*		

Level 1	$F_{u\text{-longitudinal}} =$	25.8	kip
	$F_{u\text{-transverse}} =$	39.1	kip

6. LATERAL LOAD APPLIED TO BRACED AND MOMENT FRAME

		Roof	Level 1	
Braced Frame	(Longitudinal)	6.44	12.88	kip
Moment Frame	(Transverse)	9.78	19.57	kip

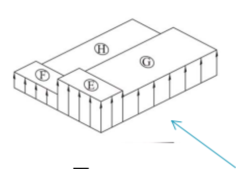
7. VERTICAL UPLIFT PRESSURES ON ROOF

Zone E P_{s30}	-27.4	psf	Figure	28.6-1
Zone F P_{s30}	-15.6	psf	Figure	28.6-1
Zone G P_{s30}	-19.1	psf	Figure	28.6-1
Zone H P_{s30}	-12.1	psf	Figure	28.6-1

Design wind pressure Zone E,	P_s	-38.36	psf
Design wind pressure Zone F,	P_s	-21.84	psf
Design wind pressure Zone G,	P_s	-26.74	psf
Design wind pressure Zone H,	P_s	-16.94	psf

7. UPLIFT PRESSURE (TRANSVERSE LOADING)

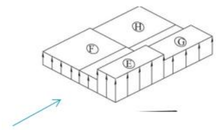
Area, Zone E	577.6	ft^2
Area, Zone F	577.6	ft^2
Area, Zone G	3678.4	ft^2
Area, Zone H	3678.4	ft^2
Total Roof Area	8512	ft^2



Weighted Uplift Pressure from Transverse Wind Load -22.96 psf

8. UPLIFT PRESSURE (LONGITUDINAL LOADING)

Area, Zone E	851.2	ft^2
Area, Zone F	851.2	ft^2
Area, Zone G	3404.8	ft^2
Area, Zone H	3404.8	ft^2
Total Roof Area	8512	ft^2



Weighted Uplift Pressure from Longitudinal Wind Load	-23.49	psf

9. MAXIMUM UPLIFT PRESSURE

Controlling Uplift Pressure	-23.49	psf	Largest Absolute Value

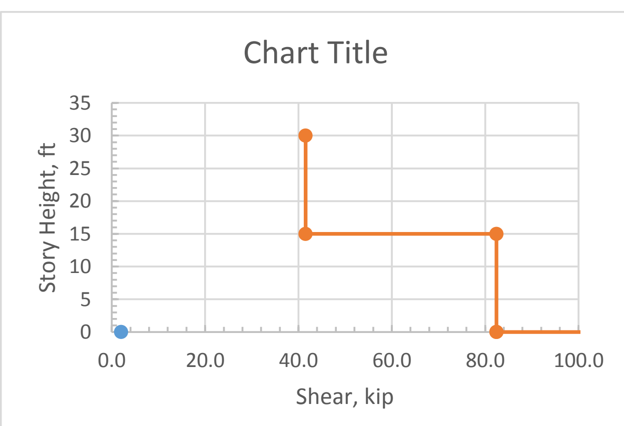
ASSUMPTIONS:

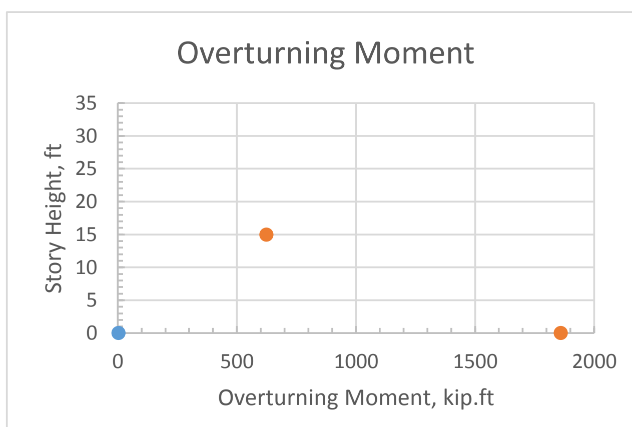
Building Frame System: Eccentrically braced steel frame

				Reference:	ASCE	7-10
				Section	*Eq/Fig/Table/Notes*	
1. SEISMIC GROUD MOTION VALUES				11.4		
Seismic Site Class:		C		11.4.2	Soil Properties / Ch. 20	
Maximum Considered Earthquake Spectral Response:						
	$S_s =$	0.250		11.4.1	Fig	22-1 / 22-4
	$S_1 =$	0.077		11.4.1	Fig	22-1 / 22-4
Adjusted MCE Spectral Response:						
	$F_a =$	1.200		11.4.3	Table	11.4-1
	$F_v =$	1.700		11.4.3	Table	11.4-2
	$S_{MS} = F_a S_s =$	0.300		11.4.3	Eq	11.4-1
	$S_{M1} = F_v S_1 =$	0.131		11.4.3	Eq	11.4-2
Design Spectral Response Acceleration Parameters:						
	$S_{DS} = 2/3\ S_{MS} =$	0.2		11.4.4	Eq	11.4-3
	$S_{D1} = 2/3\ S_{M1} =$	0.087		11.4.4	Eq	11.4-4
Design Response Spectrum:						
	$T_O = 0.2\ S_{D1}/S_{DS} =$	0.087	s	11.4.5		
	$T_S = S_{D1}/S_{DS} =$	0.436	s	11.4.5		
Long Period Transition	$T_L =$	6	s	11.4.5	Fig	22-15
	$T =$	0.54	s	Fundamental Period of Structure		
	$S_a =$ if $T < T_O : S_{DS}(0.4+0.6T/T_O) =$			11.4.5	Eq	11.4-5
	if $T_O < T < T_S : S_{DS} =$			11.4.5		
	if $T_S < T < T_L : S_{D1}/T =$	0.162		11.4.5	Eq	11.4-6
	if $T > T_L : S_{D1} * T_L / T^2 =$			11.4.5	Eq	11.4-7
2. IMPORTANCE FACTOR AND OCCUPANCY CATEGORY				11.5		
Occupancy Category:		II			Table	1-1
Importance Factor:		1			Table	11.5-1
3. SEISMIC DESIGN CATEGORY				11.6		
SDC based on short period:		B			Table	11.6-1
SDS based on 1-s period:		B			Table	11.6-2
	SDC =	**B**		*Maximum from values above*		
4. EQUIVALENT LATERAL FORCE PROCEDURE				12.8		
	$R =$	3.25		12.8.1	Table	12.2-1
	$\Omega_O =$	2		12.8.1	Table	12.2-1
	$C_D =$	3.25		12.8.1	Table	12.2-1

Approximate Fundamental Period, T_a:			12.8.2.1		
C_t =	0.03		*Dependent on structure*	Table	12.8-2
x =	0.75			Table	12.8-2
h_n =	30	ft	*Height of heighest level of structure*		
$T_a = C_t h_n^x$ =	0.385	s	12.8.2.1	Eq	12.8-7
Seismic Response Coefficient:			12.8.1.1		
$C_{Scalc} = S_{DS}/(R/I)$ =	0.062			Eq	12.8-2
C_{Smax} = if T <= T_L : $S_{D1}/(T*(R/I))$ =	0.050			Eq	12.8-3
if T > T_L : $S_{D1*}T_L/(T^2*(R/I))$ =				Eq	12.8-4
C_{Smin} =	0.01		*Revised Sup. 2*	Eq	12.8-5/12.8-6
C_S =	0.050				
Seismic Base Shear:			12.8.1		
Seismic Weight W =	1652	kip	12.7.2		Table Below
Seismic Base Shear V =	82.4	kip	12.8.1	Eq	12.8-1
Vertical Distribution of Seismic Forces:			12.8.3		
Lateral force per level	$F_x = C_{vx}V$			Eq	12.8-11
$C_{vx} = (w_x h_x^k)/(\Sigma\ w_i h_i^k)$			*Vertical Distribution Factor*		
k =	0.94		12.8.3		
Horizontal Distribution of Seismic Forces:			12.8.4		
$V_x = \Sigma\ F_i$ =				Eq	12.8-13

Floor	Height (ft)	Weight (kip)	$w_x h_x^k$	C_{vx}	F_x (kip)	V_x (kip)	Overturning Moment (kip.ft)	Total Height (ft)
Roof	15	572	14090	0.504	41.5	41.5		30
						41.5		15
2nd	15	1081	13867	0.496	40.9	82.4	623	15
						82.4		0
Podium	0	0	0	0	0	82.4	1859	0
SUM	30	1652	27957	1	82.4	123.9		0





ASSUMPTIONS:

Building Frame System: Steel moment-resisting frame

			Reference:	ASCE	7-10
			Section	Eq/Fig/Table/Notes	
1. SEISMIC GROUD MOTION VALUES			11.4		
Seismic Site Class:	C		11.4.2	Soil Properties / Ch. 20	
Maximum Considered Earthquake Spectral Response:					
S_s =	0.250		11.4.1	Fig	22-1 / 22-4
S_1 =	0.077		11.4.1	Fig	22-1 / 22-4
Adjusted MCE Spectral Response:					
F_a =	1.200		11.4.3	Table	11.4-1
F_v =	1.700		11.4.3	Table	11.4-2
$S_{MS} = F_a S_s$ =	0.3		11.4.3	Eq	11.4-1
$S_{M1} = F_v S_1$ =	0.131		11.4.3	Eq	11.4-2
Design Spectral Response Acceleration Parameters:					
$S_{DS} = 2/3\ S_{MS}$ =	0.2		11.4.4	Eq	11.4-3
$S_{D1} = 2/3\ S_{M1}$ =	0.087		11.4.4	Eq	11.4-4
Design Response Spectrum:					
$T_O = 0.2\ S_{D1}/S_{DS}$ =	0.087	s	11.4.5		
$T_S = S_{D1}/S_{DS}$ =	0.436	s	11.4.5		
Long Period Transition T_L=	6	s	11.4.5	Fig	22-15
T =	0.60	s	*Fundamental Period of Structure*		
S_a = if $T < T_O : S_{DS}(0.4+0.6T/T_O)$ =			11.4.5	Eq	11.4-5
if $T_O < T < T_S : S_{DS}$ =			11.4.5		
if $T_S < T < T_L : S_{D1}/T$ =	0.147		11.4.5	Eq	11.4-6
if $T > T_L : S_{D1} {*} T_L / T^2$=			11.4.5	Eq	11.4-7

2. IMPORTANCE FACTOR AND OCCUPANCY CATEGORY 11.5

Occupancy Category:	II	Table	1-1
Importance Factor:	1	Table	11.5-1

3. SEISMIC DESIGN CATEGORY 11.6

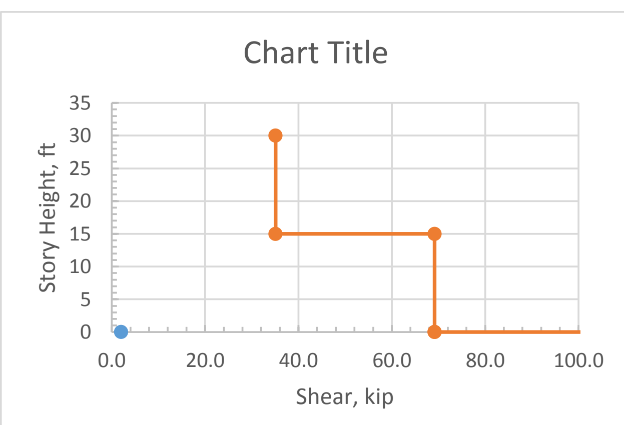
SDC based on short period:	B	Table	11.6-1
SDS based on 1-s period:	B	Table	11.6-2
SDC =	**B**	*Maximum from values above*	

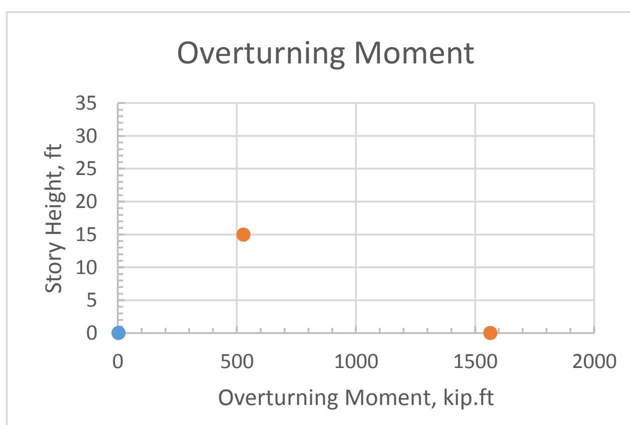
4. EQUIVALENT LATERAL FORCE PROCEDURE 12.8

R =	3.5	12.8.1	Table	12.2-1
Ω_O =	3	12.8.1	Table	12.2-1
C_D =	3	12.8.1	Table	12.2-1

Approximate Fundamental Period, T_a:			12.8.2.1		
C_t =	0.028		*Dependent on structure*	Table	12.8-2
x =	0.8			Table	12.8-2
h_n =	30	ft	*Height of heighest level of structure*		
$T_a = C_t h_n^{\ x}$ =	0.425	s	12.8.2.1	Eq	12.8-7
Seismic Response Coefficient:			12.8.1.1		
$C_{Scalc} = S_{DS}/(R/I)$ =	0.057			Eq	12.8-2
C_{Smax} = if $T <= T_L : S_{D1}/(T^*(R/I))$ =	0.042			Eq	12.8-3
if $T > T_L : S_{D1*}T_L/(T^2*(R/I))$ =				Eq	12.8-4
C_{Smin} =	0.01		*Revised Sup. 2*	Eq	12.8-5/12.8-6
C_S =	0.042		*Beware of mins and max*		
Seismic Base Shear:			12.8.1		
Seismic Weight W =	1652	kip	12.7.2		Table Below
Seismic Base Shear V =	69.2	kip	12.8.1	Eq	12.8-1
Vertical Distribution of Seismic Forces:			12.8.3		
Lateral force per level	$F_x = C_{vx}V$			Eq	12.8-11
$C_{vx} = (w_x h_x^{\ k})/(\Sigma\ w_i h_i^{\ k})$			*Vertical Distribution Factor*		
k =	0.96		12.8.3		
Horizontal Distribution of Seismic Forces:			12.8.4		
$V_x = \Sigma\ F_i$ =				Eq	12.8-13

Floor	Height (ft)	Weight (kip)	$w_x h_x^{\ k}$	C_{vx}	F_x (kip)	V_x (kip)	Overturning Moment (kip.ft)	Total Height (ft)
Roof	15	572	15105	0.508	35.1	35.1		30
						35.1		15
2nd	15	1081	14656	0.492	34.1	69.2	527	15
						69.2		0
Podium	0	0	0	0	0	69.2	1564	0
SUM	30	1652	29761	1	69.2	104.3		0





1. MOMENT FRAME INFORMATION

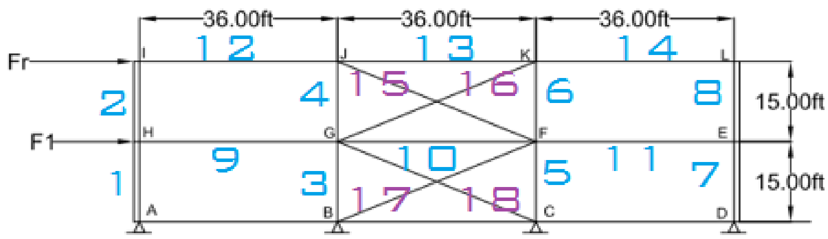


Figure 1 - Braced-Frame Member Reference

Diagonal, D	39	ft	
Story Height, H	15	ft	
Bay Width, W	36	ft	
F_r	9.78	kip	6.44
F_1	19.57	kip	0.00

2.1 MEMBER FORCES DISTRIBUTION

Find Reactions

Cy=		kip	up
By=	0.00	kip	down
Cx=	-14.68	kip	west
Bx=	-14.68	kip	west

Joint B

(Fr+F1)/2	V_1	14.68	kip	
	F_{BFy}	-6.12	kip	
Brace Force	F_{BF}	15.90	kip	T
Vertical Force	F_{BG}	-6.1152	kip	T

Joint C

V_1	14.68	kip	
F_{GCy}	-6.12	kip	C
F_{GC}	15.90	kip	C
F_{FC}	-6.12	kip	C

To solve system:

Moment Equation	15	F_{GF}	27.69	F_{GK}	-36	F_{GJ}	=	-72.06	513.69
Forces in X	-1	F_{GF}	-0.923	F_{GK}	0	F_{GJ}	=	-4.8	-4.89
Forces in Y	0	F_{GF}	-0.385	F_{GK}	1	F_{GJ}	=	4	-12

Inverse Matrix

61.533	922	2215.2
-66.666	-1000	-2400.0
-25.666	-385	-923.0

Solution

F_{GF}	1.13
F_{GK}	3.95
F_{GJ}	5.49

Joint G

Brace Force	F_{GK}	3.95	kip	C
Horizontal Force	F_{GF}	1.13	kip	C
Vertical Force	F_{GJ}	5.49	kip	T

Joint F

F_{FJ}	14.67	kip	C
F_{GF}	1.13	kip	C
F_{FK}	1.52	kip	T

Joint J

Joint K

Brace Force	F_{JF}	14.7	kip	C	F_{GK}	4.0	kip	C
Horizontal Force	$\mathbf{F_{JK}}$	3.6	kip	T	F_{KJ}	3.6	kip	T
Vertical Force	F_{JG}	5.49	kip	T	F_{KF}	1.5	kip	T

3. RESULTS

	Member *(#)*	**Frame** *(type)*	**Floor** *(Units)*	**Function**	**Force** *(kip)*	**T/C**	**Moment** *(kip.ft)*
GF	10	Braced	First	Beam	14.67	C	
JK	13	Braced	Roof	Beam	3.65	T	
JF	15	Braced	Roof	Brace	14.67	C	
GK	16	Braced	Roof	Brace	3.95	C	
BF	17	Braced	First	Brace	15.90	T	
GC	18	Braced	First	Brace	15.90	C	
BG	3	Braced	First	Column	-6.12	T	
JG	4	Braced	Roof	Column	5.49	T	
CF	5	Braced	First	Column	-6.12	C	
KF	6	Braced	Roof	Column	1.52	T	

1. MOMENT FRAME INFORMATION

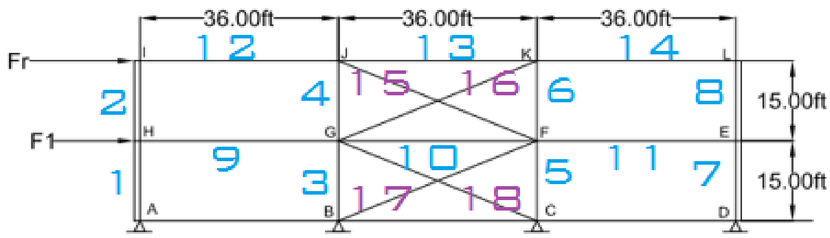


Figure 1 - Braced-Frame Member Reference

Diagonal, D	39	ft
Story Height, H	15	ft
Bay Width, W	36	ft
F_r	20.77	kip
F_1	20.44	kip

2.1 MEMBER FORCES DISTRIBUTION

Find Reactions

Cy=		kip	up
By=	0	kip	down
Cx=	-20.603	kip	west
Bx=	-20.603	kip	west

Joint B

(Fr+F1)/2	V_1	20.60	kip	
	F_{BFy}	-8.58	kip	
Brace Force	F_{BF}	22.32	kip	T
Vertical Force	F_{BG}	-8.58	kip	T

Joint C

V_1	20.60	kip	
F_{GCy}	-8.58	kip	C
F_{GC}	22.32	kip	C
F_{FC}	-8.58	kip	C

To solve system:

Moment Equation	15	F_{GF}	27.69	F_{GK}	-36	F_{GJ}	=	29.1	615.63
Forces in X	-1	F_{GF}	-0.923	F_{GK}	0	F_{GJ}	=	-6.25	0.16
Forces in Y	0	F_{GF}	-0.385	F_{GK}	1	F_{GJ}	=	1.796	-17.17

Inverse Matrix

61.533	922	2215.2
-66.666	-1000	-2400.0
-25.666	-385	-923.0

Solution

F_{GF}	6.61
F_{GK}	-0.38
F_{GJ}	1.66

Joint G

Brace Force	F_{GK}	-0.38	kip	C
Horizontal Force	F_{GF}	6.61	kip	C
Vertical Force	F_{GJ}	1.66	kip	T

Joint F

F_{FJ}	15.16	kip	C
F_{GF}	6.61	kip	C
F_{FK}	-0.15	kip	T

Joint J

Joint K

Brace Force	F_{JF}	15.2	kip	C	F_{GK}	-0.4	kip	C
Horizontal Force	$\mathbf{F_{JK}}$	-0.4	kip	T	F_{KJ}	-0.4	kip	T
Vertical Force	F_{JG}	1.66	kip	T	F_{KF}	-0.1	kip	T

3. RESULTS

	Member	**Frame**	**Floor**	**Function**	**Force**	**T/C**	**Moment**
	(#)	*(type)*	*(Units)*		*(kip)*		*(kip.ft)*
GF	10	Braced	First	Beam	15.16	C	
JK	13	Braced	Roof	Beam	-0.35	T	
JF	15	Braced	Roof	Brace	15.16	C	
GK	16	Braced	Roof	Brace	-0.38	C	
BF	17	Braced	First	Brace	22.32	T	
GC	18	Braced	First	Brace	22.32	C	
BG	3	Braced	First	Column	-8.58	T	
JG	4	Braced	Roof	Column	1.66	T	
CF	5	Braced	First	Column	-8.58	C	
KF	6	Braced	Roof	Column	-0.15	T	

1. MOMENT FRAME INFORMATION

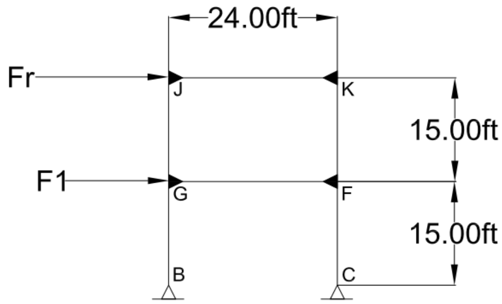


Figure 1 - Moment Frame

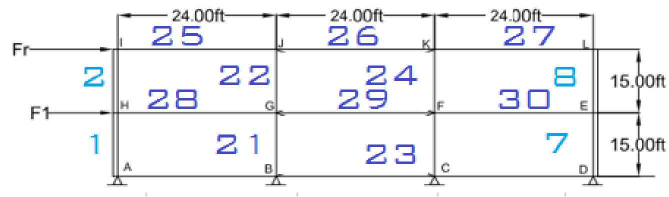


Figure 2 - Member Reference

Story Height, H =	15	ft
Bay Width, W =	24	ft
$F_{r} =$	9.78	kip
$F_{1} =$	19.57	kip

2.1 MEMBER FORCES DISTRIBUTION

Compute Shear Forces

$\sum F_x=0$

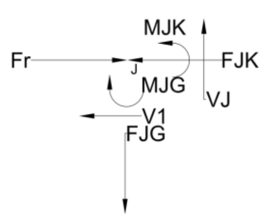
$F_{r} =$	9.78	kips
# of shear forces	2	
$V_{1} =$	4.89	kips

Compute moments at top of columns

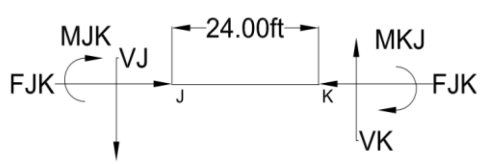
$M_{JG} =$	36.69	kip*ft
$M_{KF} =$	36.69	kip*ft

METHOD OF JOINTS:

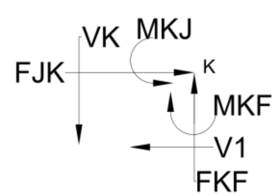
Joint J



Member J-K



Joint K



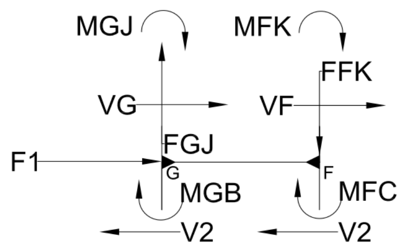
Joint J:

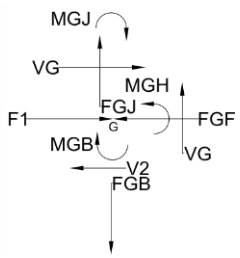
	$M_{JK} =$	36.69	kip*ft	
$\sum Fx=0$	$F_{JK} =$	4.89	kips	C
$\sum M/W$	$V_{J} =$	3.06	kips	
$\sum Fy=0$	$F_{JG} =$	3.06	kips	T

Joint K:

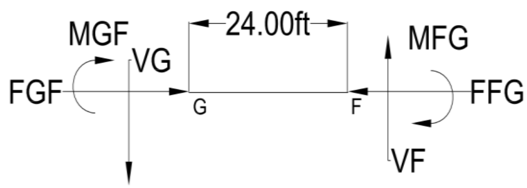
$M_{KJ} =$	36.69	kip*ft	
$F_{JK} =$	4.89	kips	C
$V_{K} =$	3.06	kips	
$F_{KF} =$	3.06	kips	C
$M_{KF} =$	36.69	kips	

Section 2

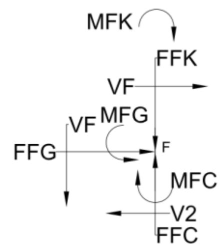




Joint G



Member G-F



Joint F

Joint G:

(F1+FR)/2	$V_{2} =$	14.68	kips	
FJG	$F_{GJ} =$	3.06	kips	T
V2*H/2	$M_{GB} =$	110.1	kip*ft	
(MJG)	$M_{GJ} =$	36.69	kip*ft	
(FJG)	$F_{GJ} =$	3.06	kips	
$\sum$Fx=0	$F_{GF} =$	4.89	kips	C
$\sum$M	$M_{GF} =$	146.76	kip*ft	
$\sum$M/W	$V_{G} =$	12.23	kips	
$\sum$Fy=0	$F_{GB} =$	15.29	kips	

Joint F

(F1+FR)/2	$V_{2} =$	14.68	kips	
FKF	$F_{FK} =$	3.06	kips	T
V2*H/2	$M_{FC} =$	110.1	kip*ft	
MKF	$M_{FK} =$	36.69	kip*ft	
FKF	$F_{FK} =$	3.06	kips	
FGF	$F_{FG} =$	4.89	kips	C
$\sum$M	$M_{FG} =$	146.76	kip*ft	
$\sum$M/W	$V_{G} =$	12.23	kips	
$\sum$Fy=0	$F_{FC} =$	15.29	kips	

3. RESULTS

	Member (#)	Frame (type)	Floor (Units)	Function	Force (kip)	T/C	Moment (kip.ft)
GF	29	Moment	First	Beam	4.89	C	146.76
JK	26	Moment	Roof	Beam	4.89	C	36.69
BG	21	Moment	First	Column	15.29	T	110.07
CF	23	Moment	First	Column	15.29	C	110.07
JG	22	Moment	Roof	Column	4.89	T	36.69
KF	24	Moment	Roof	Column	4.89	C	36.69

1. MOMENT FRAME INFORMATION

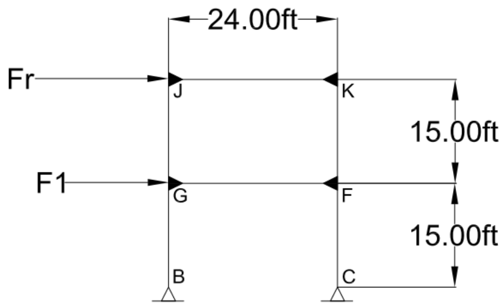


Figure 1 - Moment Frame

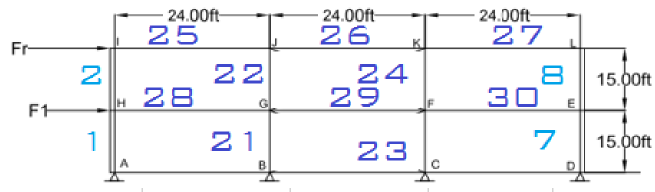


Figure 2 - Member Reference

Story Height, H =	15	ft
Bay Width, W =	24	ft
$F_{r=}$	17.55	kip
$F_{1=}$	17.03	kip

2.1 MEMBER FORCES DISTRIBUTION

Compute Shear Forces

$\sum F_x=0$

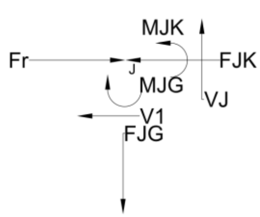
$F_{r=}$	17.55	kips
# of shear forces	2	
$V_{1=}$	8.78	kips

Compute moments at top of columns

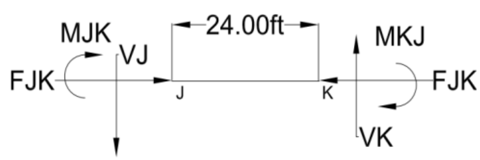
$M_{JG=}$	65.82	kip*ft
$M_{KF=}$	65.82	kip*ft

METHOD OF JOINTS:

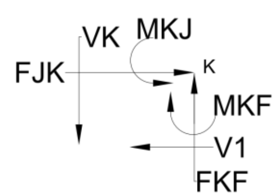
Joint J



Member J-K



Joint K



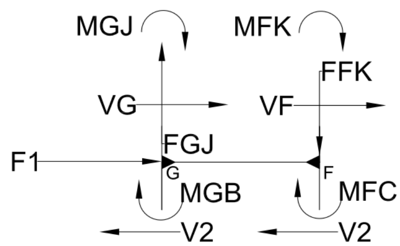
Joint J:

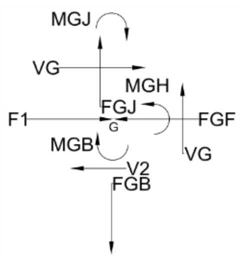
	$M_{JK=}$	65.82	kip*ft	
$\sum Fx=0$	$F_{JK=}$	8.78	kips	C
$\sum M/W$	$V_{J=}$	5.49	kips	
$\sum Fy=0$	$F_{JG=}$	5.49	kips	T

Joint K:

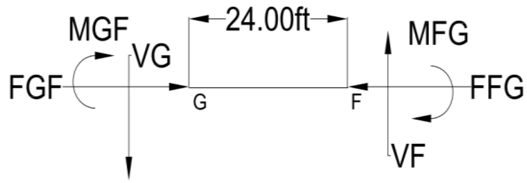
$M_{KJ=}$	65.82	kip*ft	
$F_{JK=}$	8.78	kips	C
$V_{K=}$	5.49	kips	
$F_{KF=}$	5.49	kips	C
$M_{KF=}$	65.82	kips	

Section 2

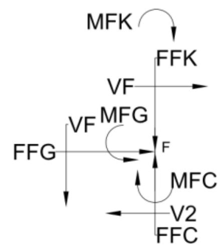




Joint G



Member G-F



Joint F

Joint G:

(F1+FR)/2	V_{2} =	7.9	kips	
FJG	F_{GJ} =	5.49	kips	T
V2*H/2	M_{GB} =	59.44	kip*ft	
(MJG)	M_{GJ} =	65.82	kip*ft	
(FJG)	F_{GJ} =	5.49	kips	
$\sum$Fx=0	F_{GF} =	9.11	kips	T
$\sum$M	M_{GF} =	125.3	kip*ft	
$\sum$M/W	V_{G} =	10.4	kips	
$\sum$Fy=0	F_{GB} =	15.9	kips	

Joint F

(F1+FR)/2	V_{2} =	7.9	kips	
FKF	F_{FK} =	5.5	kips	C
V2*H/2	M_{FC} =	59.4	kip*ft	
MKF	M_{FK} =	65.8	kip*ft	
FKF	F_{FK} =	5.49	kips	
FGF	F_{FG} =	9.11	kips	T
$\sum$M	M_{FG} =	125.3	kip*ft	
$\sum$M/W	V_{G} =	10.4	kips	
$\sum$Fy=0	F_{FC} =	15.9	kips	

3. RESULTS

	Member	Frame	Floor	Function	Force	T/C	Moment
	(#)	*(type)*	*(Units)*		*(kip)*		*(kip.ft)*
GF	29	Moment	First	Beam	9.11	T	125.26
JK	26	Moment	Roof	Beam	8.78	C	65.82
BG	21	Moment	First	Column	15.92	T	59.44
CF	23	Moment	First	Column	15.92	C	59.44
JG	22	Moment	Roof	Column	9.11	T	65.82
KF	24	Moment	Roof	Column	9.11	C	65.82

GIVEN:

					First	**Roof**
Number of Floors:	2	**Number of Bays/Row:**	3	**Number of Rows:**	2	3

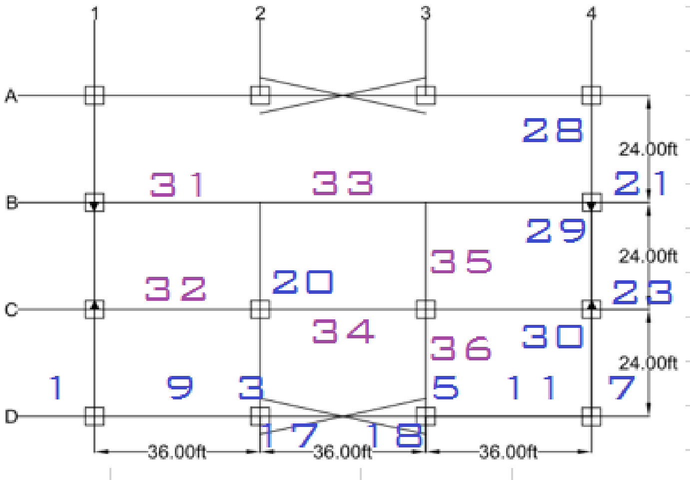


Figure 1 - First Floor Plan

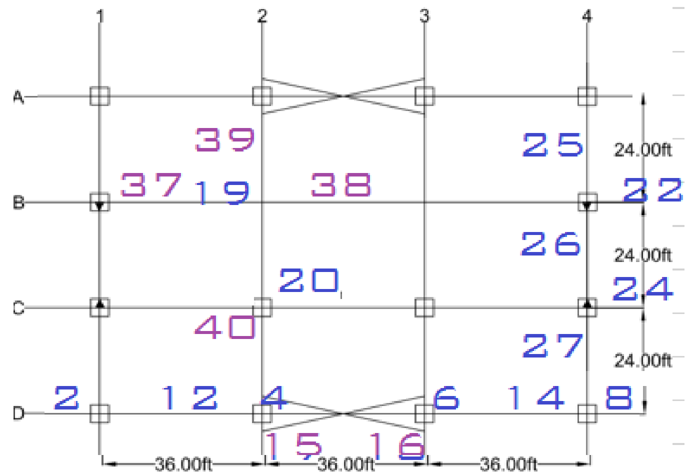


Figure 2 - Roof Floor Plan

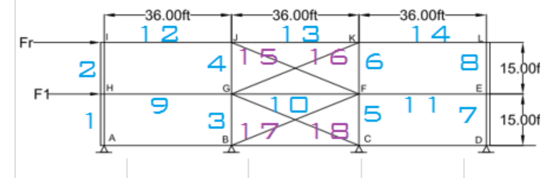


Figure 3 - Braced-Elevation

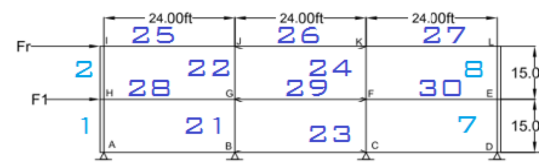


Figure 4 - Moment Frame - Elevation

Member Ref. (#)	Frame (type)	Floor (Units)	Member	Section (Shape)	Length (ft)	Unit Weight (plf)	Spacing O.C. (ft)	Qty of Members (Units)	Amount of Steel (kips)	DCR Moment (ratio)	DCR Shear (ratio)	Type of Support	Load Condition
1	Braced	First	Column	W8X40	15	40	12	2.00	1.20			Simply	Uniform
2	Braced	Roof	Column	W8X40	15	40		0.00	0.00			Simply	Uniform
3	Braced	First	Column	W8X40	15	40		3.00	1.80			Simply	Uniform
4	Braced	Roof	Column	W8X40	15	40		2.00	1.20			Simply	Uniform
5	Braced	First	Column	W8X40	15	40		2.00	1.20			Moment	Uniform
6	Braced	Roof	Column	W8X40	15	40		0.00	1.20			Simply	Uniform
7	Moment	First	Column	W8X40	15	40		0.00	2.40			Simply	Uniform

8	Moment	Roof	Column	W8X40	15	40	0.00	0.77	Simply	2-Point
9	Braced	First	Girder	W21X55	36	55	0.00	1.68	Simply	Uniform
10	Braced	First	Interior Girder	W21X55	36	55	0.00	1.54	Simply	Uniform
11	Braced	First	Beam	W21X55	36	55	0.00	2.11	Simply	Uniform
12	Braced	Roof	Girder	W14X34	36	34	0.00	4.22	Simply	Uniform
13	Braced	Roof	Interior Girder	W14X34	36	34	0.00	2.11	Simply	Uniform
14	Braced	Roof	Girder	W14X34	36	34	0.00	#N/A	Simply	Uniform
15	Braced	Roof	Braces	WT9X48.5	39	48.5	0.00	#N/A	Simply	2-Point
16	Braced	Roof	Braces	WT9X48.5	39	48.5	0.00	#N/A	Moment	Uniform
17	Braced	:Design!D4	Braces	WT9X48.5	39	48.5	0.00	#N/A	Simply	Uniform
18	Braced	First	Braces	WT9X48.5	39	48.5	0.00	3.36	Simply	2-Point
19	Interior	Roof	Roof Column	0	15	55	0.00	1.68	Simply	Uniform
20	Interior	First	Interior Column	0	30	55	0.00	#N/A	Simply	Uniform
21	Moment	First	Column	W8X40	15	40	0.00	#N/A	Simply	Axial
22	Moment	Roof	Column	W8X40	15	40	0.00	#N/A	Simply	Uniform
23	Moment	First	Column	W8X40	15	40	2.00	#N/A	Simply	Uniform
24	Moment	Roof	Column	W8X40	15	40	4.00	#N/A	0	0.00
25	Moment	Roof	Beam	W12X16	24	16	2.00	0.00	0	0.00
26	Moment	Roof	Interior Beam	W18X35	24	35	2.00	0.00	0	0.00
27	Moment	Roof	Beam	W12X16	24	16	4.00	0.00	0	0.00
28	Moment	First	Beam	W14X22	24	22	4.00	0.00	0	0.00
29	Moment	First	Interior Beam	W21X44	24	44	4.00	0.00	0	0.00
30	Moment	First	Beam	W14X22	24	22	4.00	0.00	0	0.00
31	Interior	First	Girder		36		24.00	0.00	0	0.00
32	Interior	First	Girder	0	36		6.00	0.00	0	0.00
33	Interior	First	Interior Girder		36		2.00	0.00	0	0.00
34	Interior	First	Interior Girder	0	36		4.00	0.00	0	0.00
35	Interior	First	Beam	W18X35	24	35	4.00	0.00	0	0.00
36	Interior	First	Beam	W18X35	24	35	2.00	0.00	0	0.00
37	Interior	Roof	Girder		36		2.00	0.00	0	0.00
38	Interior	Roof	Interior Girder	0	36		2.00	0.00	0	0.00
39	Interior	Roof	Beam	W12X19	48		2.00	0.00	0	0.00
40	Interior	Roof	Beam	W12X19	24		2.00	0.00	0	0.00

TOTAL	#N/A

GIVEN:

Number of Floors: 2

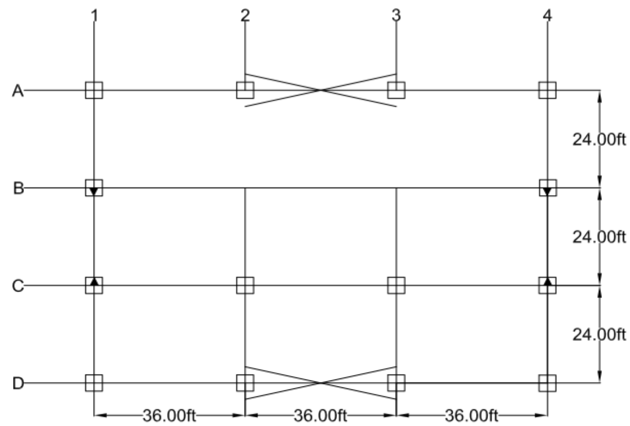


Figure 1 - First Floor Plan

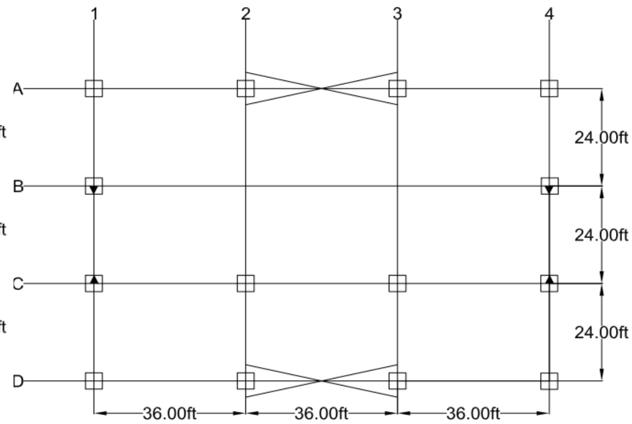


Figure 2 - Roof Floor Plan

ASSUMPTIONS:

Beams running parallel to moment frame for both levels

					Reference: Section Information	Excel *Eq/Fig/Table/Notes*	
1. FIRST FLOOR							
Member Ref:		36			*From Summary Sheet*		
Frame:		Interior					
Floor:		First					
Beam Length	L =	24	ft		*Project Information*		
Beam Spacing	s =	8	ft	O.C.	*Enter chosen spacing for beams*		
Support Condition		Simply					
Load Condition		Uniform			*AISC 14th*	Table	3-23
Case		1			*AISC 14th*	Table	3-24

Loads:

Dead Load	DL =	128	psf	*Project Loads*
Steel Selected	w =	35	psf	*Project Loads*
Total Dead Load	TDL =	163	psf	*Per Beam Selection Below*
Live Load	LL =	100	psf	
Snow Load	S =	0.0	psf	
Wind Load	W =	0.0	psf	
Seismic Load	E =	0.0	psf	

LRFD:	**1st Floor**	
1. 1.4D	228.2	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	355.6	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5V	295.6	psf

4. 1.2D + 1.0W + L + .5(Lr or S or R)	195.6	psf
5. 1.2D + 1.0E + L + .2S	295.6	psf
6. 0.9D + 1.0W	146.7	psf
7. 0.9D + 1.0E	146.7	psf

Controlling Load: 355.6 psf
Equivalent Linear Load: 2844.8 plf — *Given spacing chosen above*

Demand Values:

Ultimate Moment,	M_u =	**204.8**	kip.ft	*Given load and support conditions above*
Ultimate Shear,	V_u =	**34.1**	kip	*Given load and support conditions above*

Beam Selection,	**W:**	**W18X35**				
	ϕM_n	**249.0**	kip.ft	AISC	Table	3-2
	ϕV_n	**159.0**	kip	AISC	Table	3-2
Beam Depth:	d	**17.7**	in			

Design Check:	$\phi M_n > M_u$?	**YES**
	$\phi V_n > V_u$?	**YES**
Depth Clearance:		**OK**

DCR	**Moment**	0.82	OK	*Demand Capacity Ratio*
	Shear	0.21	OK	

MOMENT FRAME

CASE 1: GIRDER CD/AB ALONG COLUMN LINES 1 AND 4

Number of Floors: 2

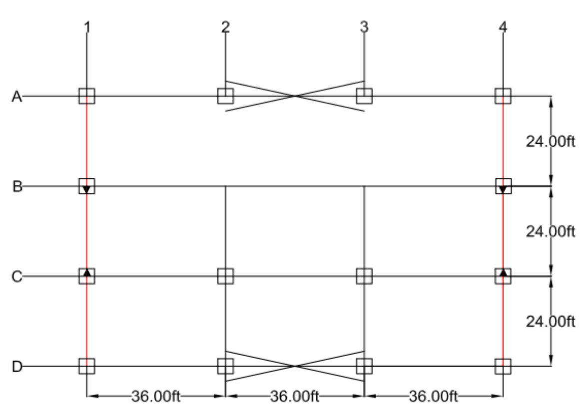


Figure 1 - First Floor Plan

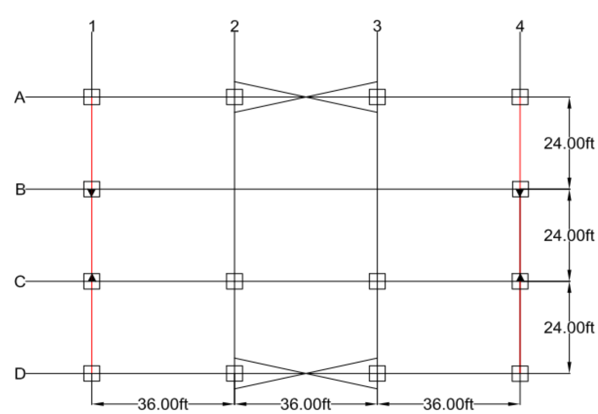
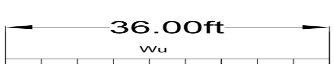


Figure 2 - Roof Floor Plan

ASSUMPTIONS:



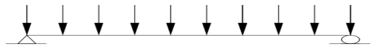


Figure 3 - Girder Loads Level 1 and Roof

1. FIRST FLOOR

				Reference: Section Information	Excel Eq/Fig/Table/Notes	
Member Ref:		28		*From Summary Sheet*		
Frame:		Moment				
Floor:		First				
Beam Length	L =	24	ft	*Project Information*		
Distance a	a =	0	ft	*AISC 14th*	Table	3-23
Support Condition		Simply				
Load Condition		Uniform		*AISC 14th*	Table	3-23
Case		1		*AISC 14th*	Table	3-24
Beam Tributary Area	A	288	ft^2	*Enter chosen spacing for beams*		

Loads:

Dead Load	DL =	128	psf	*Project Loads*
Steel Selected	w =	22	psf	
Total Dead Load	TDL =	150	psf	
Live Load	LL =	100	psf	
Snow Load	S =	0.0	psf	
Wind Load	W =	0.0	psf	
Seismic Load	E =	0.0	psf	

LRFD:	**1st Floor**	
1. 1.4D	210.0	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	340.0	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5V	280.0	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	180.0	psf
5. 1.2D + 1.0E + L + .2S	280.0	psf
6. 0.9D + 1.0W	135.0	psf
7. 0.9D + 1.0E	135.0	psf

Controlling Load:	340.0	psf	*Given spacing chosen above*
Equivalent Linear 1/2 Load:	1360	plf	

Demand Values:

Ultimate Moment,	M_u =	**97.9**	kip.ft	*Given load condition specified above*		
Ultimate Shear,	V_u =	**16.3**	kip	*Given load condition specified above*		
Beam Selection,	**W:**	**W14X22**				
	ϕM_n	**125.0**	kip.ft	AISC	Table	3-2
	ϕV_n	**94.5**	kip	AISC	Table	3-2
Beam Depth:	d	**13.7**	in			

Design Check:	$\phi M_n > M_u$?	**YES**		
	$\phi V_n > V_u$?	**YES**		
Depth Clearance:		**OK**		
DCR	**Moment**	0.78	OK	*Demand Capacity Ratio*
	Shear	0.17	OK	

CASE 2: BRACED GIRDER BC ALONG COLUMN LINES 1 AND 4

Number of Floors: 2

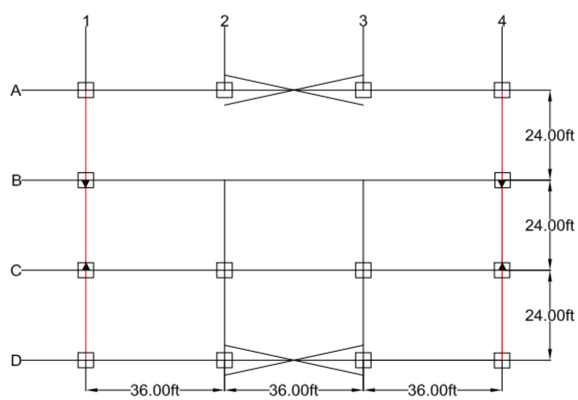


Figure 1 - First Floor Plan

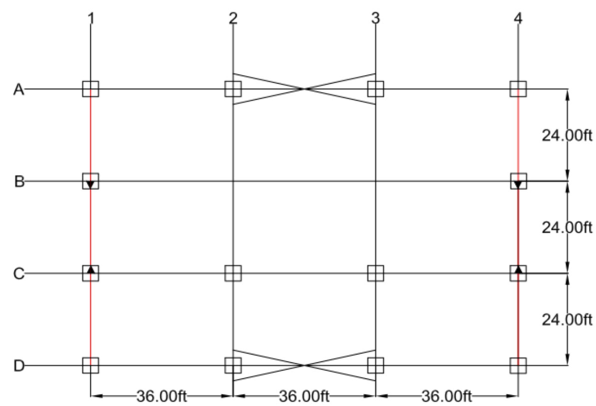


Figure 2 - Roof Floor Plan

ASSUMPTIONS:

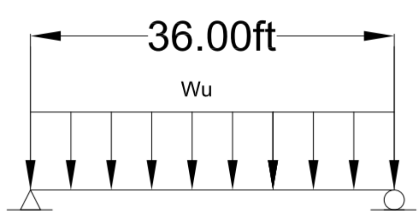


Figure 3 - Girder Loads Level 1 and Roof

1. FIRST FLOOR

				Reference: Section Information	Excel Eq/Fig/Table/Notes	
Member Ref:		29		*From Summary Sheet*		
Frame:		Moment				
Floor:		First				
Beam Length	L =	24	ft	*Project Information*		
Distance a	a =	8	ft	*AISC 14th*	Table	3-23
Support Condition		Simply				
Load Condition		Uniform		*AISC 14th*	Table	3-23
Case		1		*AISC 14th*	Table	3-24

Beam Tributary Area	A	288	ft^2	*Enter chosen spacing for beams*
Loads:				
Dead Load	DL =	128	psf	*Project Loads*
Steel Selected	w =	44	psf	
Total Dead Load	TDL =	172	psf	
Live Load	LL =	100	psf	*Project Loads*
Snow Load	S =	31.5	psf	*Project Loads*
Wind Load	W =	-23.5	psf	*Project Loads*
Seismic Load	E =		psf	

End Moments per Load:

Dead Load	M_{DL} =	99.1	kip.ft	
Live Load	M_{LL} =	57.6	kip.ft	
Snow Load	M_{SNOW} =	18.1	kip.ft	
Wind Load	M_{WIND} =	146.8	kip.ft	*From Moment Frame-Wind Calculations*
Seismic Load	$M_{seismic}$ =	125.3	kip.ft	*From Moment Frame-Seismic Calculations*

LRFD:	**1st Floor**	
1. 1.4D	138.7	kip.ft
2. 1.2D + 1.6L + .5(Lr or S or R)	220.1	kip.ft
3. 1.2D + 1.6(Lr or S or R) + (L or .5V	221.3	kip.ft
4. 1.2D + 1.0W + L + .5(Lr or S or R)	274.7	kip.ft
5. 1.2D + 1.0E + L + .2S	305.4	kip.ft
6. 0.9D + 1.0W	235.9	kip.ft
7. 0.9D + 1.0E	214.4	kip.ft

Controlling Moment:		**305.4**	kip.ft			
Ultimate Shear,	V_u =	**50.9**	kip	*Given load and support conditions above*		
Beam Selection,	**W:**	**W21X44**				
	ϕM_n	**358.0**	kip.ft	AISC	Table	3-2
	ϕV_n	**217.0**	kip	AISC	Table	3-2
Beam Depth:	d	**20.7**	in			

Design Check:	$\phi M_n > M_u$?	**YES**
	$\phi V_n > V_u$?	**YES**
Depth Clearance:		**OK**

DCR	**Moment**	0.85	OK	*Demand Capacity Ratio*
	Shear	0.23	OK	

MOMENT-FRAME EDGE COLUMNS SOUTH SIDE:

Number of Floors: 2

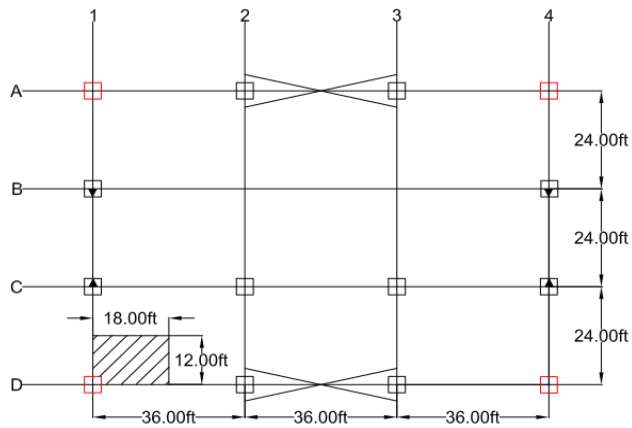


Figure 1 - First Floor Plan

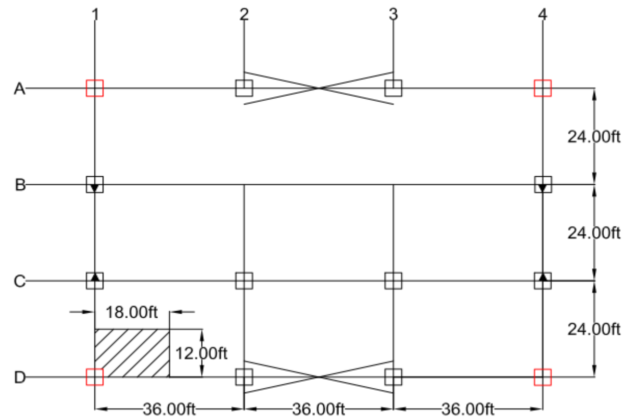


Figure 2 - Roof Floor Plan

ASSUMPTIONS:

Cummulative loads control design
South Side controls design for symmetrical frame design

1. FIRST FLOOR				Reference: Section Information	Excel /Fig/Table/Not	
Member Ref:		7		19 *From Summary Sheet*		
Frame:		Moment				
Floor:		First				
Beam Length	L =	15	ft	*Project Information*		
Distance a	a =	0	ft	*AISC 14th*	Table	3-23
Support Condition		Simply				
Load Condition		Uniform		*AISC 14th*	Table	3-23
Case		1		*AISC 14th*	Table	3-24
Tributary Area	A =	216	ft^2	*Per figure 1 and 2*		

Loads:

Dead Load	DL =	154	psf
Live Load	LL =	120	psf
Snow Load	S =	31.5	psf
Wind Load	W =	-23.5	psf
Seismic Load	E =		psf

LRFD:	**1st Floor**	
1. 1.4D	215.6	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	392.6	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)	355.2	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	177.1	psf
5. 1.2D + 1.0E + L + .2S	311.1	psf
6. 0.9D + 1.0W	115.1	psf

7. 0.9D + 1.0E — 138.6 psf

Controlling Load: 392.6 psf
Control: P_u 84.8 kip

Buckling Analysis:

K =	1		AISC 14th	Table C-A-7.1
KL =	15	ft		
KL/r =	60	ft	*Assumed*	
$\phi F_{cr} =$	34.6	ksi	AISC 14th	Table 4-22

Demand Value: $A_{reqd} =$ 2.45 in^2

Beam Selection, W: **W8X40**

$r_y =$	2.04	in^3		
A =	11.7	in^2		
KL/r =	88.24			
$\phi F_{cr} =$	25.2	ksi	AISC 14th	Table 4-22
$\phi P_n =$	294.84	kips		

Design Check: $\phi P_n > P_u$? **YES**

DCR **Compressive** 0.29 OK *Demand Capacity Ratio*

MOMENT-FRAME INTERIOR COLUMNS WEST/EAST SIDE:

Number of Floors: 2

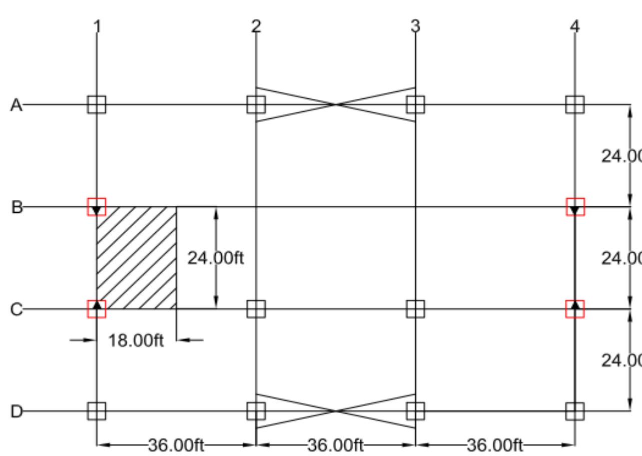


Figure 1 - First Floor Plan

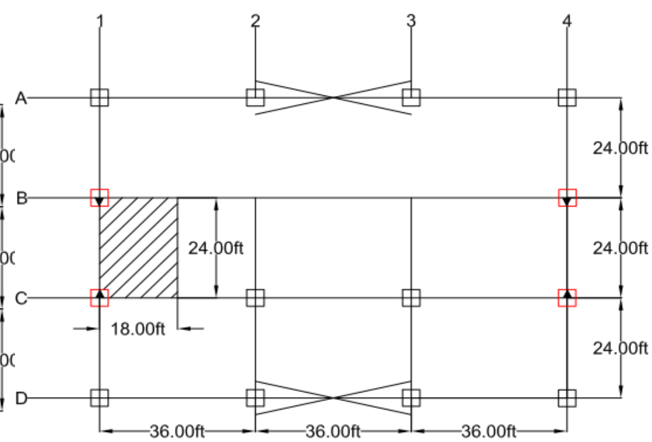


Figure 2 - Roof Floor Plan

ASSUMPTIONS:

Cummulative loads control design
South Side controls design for symmetrical frame design

Reference: Excel
Section /Fig/Table/Not
Information

1. FIRST FLOOR

Member Ref:		23		19	*From Summary Sheet*		
Frame:		Moment					
Floor:		First					
Beam Length	L =	15	ft		*Project Information*		
Distance a	a =	0	ft		*AISC 14th*	Table	3-23
Support Condition		Simply					
Load Condition		Uniform			*AISC 14th*	Table	3-23
Case		1			*AISC 14th*	Table	3-24
Tributary Area	A =	432	ft^2		*Per figure 1 and 2*		

Loads:

Dead Load	DL =	154	psf
Live Load	LL =	120	psf
Snow Load	S =	31.5	psf
Wind Load	W =	-23.5	psf
Seismic Load	E =		psf

LRFD:

	1st Floor	
1. 1.4D	215.6	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	392.6	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)	355.2	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	177.1	psf
5. 1.2D + 1.0E + L + .2S	311.1	psf
6. 0.9D + 1.0W	115.1	psf
7. 0.9D + 1.0E	138.6	psf

Controlling Load:		392.6	psf
Control:	P_u	169.6	kip

Buckling Analysis:

	K =	1		AISC 14th	Table	C-A-7.1
	KL =	15	ft			
	KL/r =	60	ft	*Assumed*		
	$\phi F_{cr} =$	34.6	ksi	AISC 14th	Table	4-22
Demand Value:	$A_{reqd} =$	4.90	in^2			

Beam Selection,	**W:**		**W8X40**				
		$r_y =$	2.04	in^3			
		A =	11.7	in^2			
		KL/r =	88.24				
		$\phi F_{cr} =$	25.2	ksi	AISC 14th	Table	4-22
		$\phi P_n =$	294.84	kips			

Design Check:	$\phi P_n > P_u$?	**YES**

DCR	**Compressive**	0.58	OK	*Demand Capacity Ratio*

BRACED FRAME

GIVEN:

Number of Floors: 2

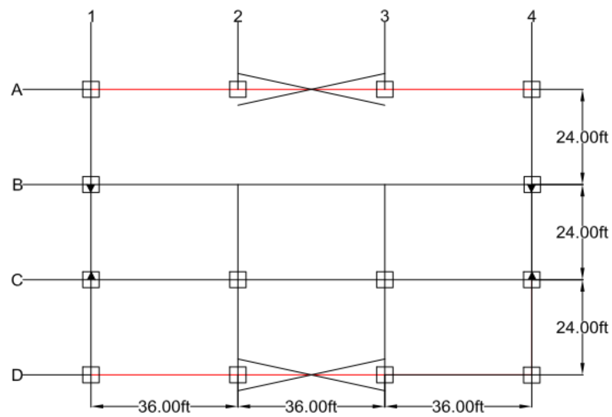


Figure 1 - First Floor Plan

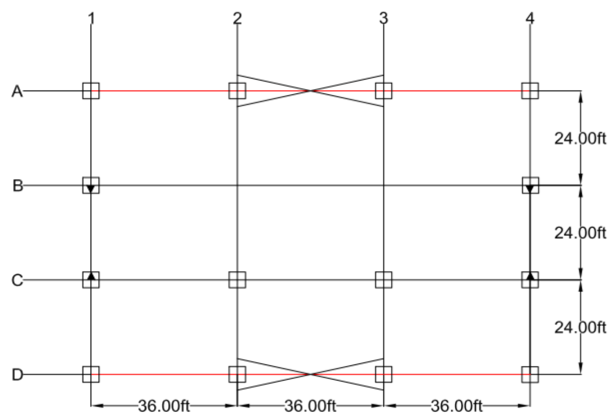


Figure 2 - Roof Floor Plan

ASSUMPTIONS:

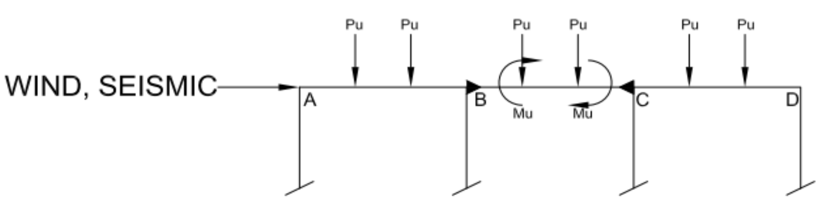


Figure 3 - Girder Loads Level 1 and Roof

					Reference: Section	Excel Eq/Fig/Table/Notes	
1. FIRST FLOOR					Information		
Member Ref:		9		8	*From Summary Sheet*		
Frame:		Braced					
Floor:		First					
Beam Length	L =	36	ft		*Project Information*		
Distance a	a =	12	ft		*AISC 14th*	Table	3-23
Support Condition		Simply					
Load Condition		2-Point			*AISC 14th*	Table	3-23
Case		9			*AISC 14th*	Table	3-24
Beam Tributary Area	A	432	ft^2		*Enter chosen spacing for beams*		

Loads:

Dead Load	DL =	128	psf	*Project Loads*

Steel Selected	w =	55	psf	*Project Loads*
Total Dead Load	TDL =	183	psf	*Per Beam Selection Below*
Live Load	LL =	100	psf	
Snow Load	S =	0.0	psf	
Wind Load	W =	0.0	psf	
Seismic Load	E =	0.0	psf	

LRFD:	**1st Floor**	
1. 1.4D	256.2	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	379.6	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5V	319.6	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	219.6	psf
5. 1.2D + 1.0E + L + .2S	319.6	psf
6. 0.9D + 1.0W	164.7	psf
7. 0.9D + 1.0E	164.7	psf

Controlling Load:	379.6	psf	
Equivalent Linear Load:	3.04	klf	*Given spacing chosen above*
Half Point Load:	36.44	kip	

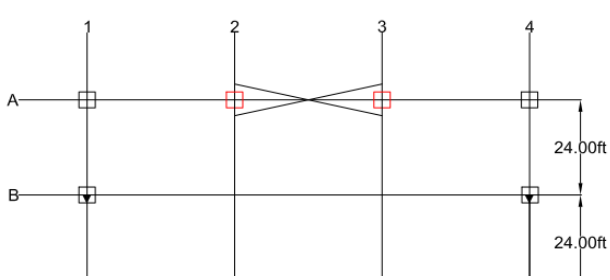
Ultimate Moment,	M_u =	**437.3**	kip.ft	*Given uniform distribued factored load*		
Ultimate Shear,	V_u =	**36.4**	kip	*Given uniform distribued factored load*		
Beam Selection,	**W:**	**W21X55**				
	ϕM_n	**473.0**	kip.ft	AISC	Table	3-2
	ϕV_n	**234.0**	kip	AISC	Table	3-2
Beam Depth:	d	**20.8**	in			

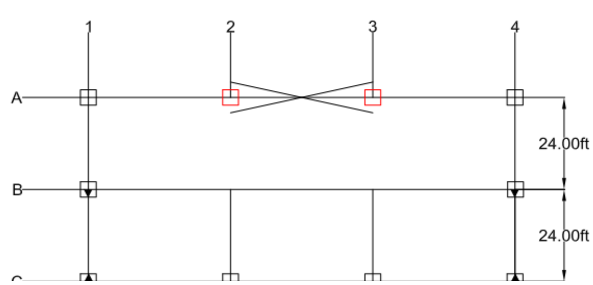
Design Check:	$\phi M_n > M_u$?	**YES**
	$\phi V_n > V_u$?	**YES**
Depth Clearance:		**OK**

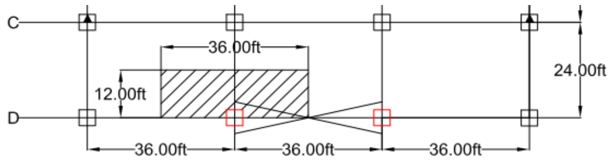
DCR	**Moment**	0.92	OK	*Demand Capacity Ratio*
	Shear	0.16	OK	

BRACED-FRAME INTERIOR COLUMN SOUTH SIDE:

Number of Floors: 2







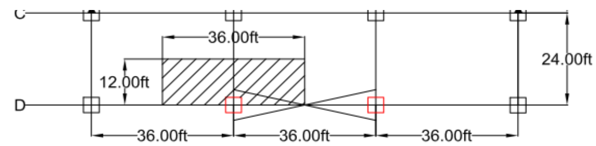


Figure 1 - First Floor Plan

Figure 2 - Roof Floor Plan

ASSUMPTIONS:

Cummulative loads control design
South Side controls design for symmetrical frame design

1. FIRST FLOOR

				Reference: Section Information	Excel /Fig/Table/Not	
Member Ref:		3		19 *From Summary Sheet*		
Frame:		Braced				
Floor:		First				
Member Length	L =	15	ft	*Project Information*		
Distance a	a =	5	ft	*AISC 14th*	Table	3-23
Support Condition		Simply				
Load Condition		Uniform		*AISC 14th*	Table	3-23
Case		1		*AISC 14th*	Table	3-24
Tributary Area	A =	432	ft^2	*Per figure 1 and 2*		

Loads:

Dead Load	DL =	154	psf
Live Load	LL =	120	psf
Snow Load	S =	31.5	psf
Wind Load	W =	-23.5	psf
Seismic Load	E =		psf

LRFD:	**1st Floor**	
1. 1.4D	215.6	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	392.6	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)	355.2	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	177.1	psf
5. 1.2D + 1.0E + L + .2S	311.1	psf
6. 0.9D + 1.0W	115.1	psf
7. 0.9D + 1.0E	138.6	psf

Controlling Load:		392.6	psf
Control:	P_u	169.6	kip

Buckling Analysis:

K =	1		AISC 14th	Table	C-A-7.1
KL =	15	ft			

	$KL/r =$	60	ft	*Assumed*		
	$\phi F_{cr} =$	34.6	ksi	AISC 14th	Table	4-22
Demand Value:	$A_{reqd} =$	4.90	in^2			

Beam Selection, **W:**		**W8X40**				
	$r_y =$	2.04	in^3			
	$A =$	11.7	in^2			
	$KL/r =$	88.24				
	$\phi F_{cr} =$	25.2	ksi	AISC 14th	Table	4-22
	$\phi P_n =$	294.84	kips			

Design Check: $\phi P_n > P_{u?}$ **YES**

DCR **Compressive** 0.58 OK *Demand Capacity Ratio*

BRACED-FRAME EDGE COLUMNS SOUTH SIDE:

Number of Floors: 2

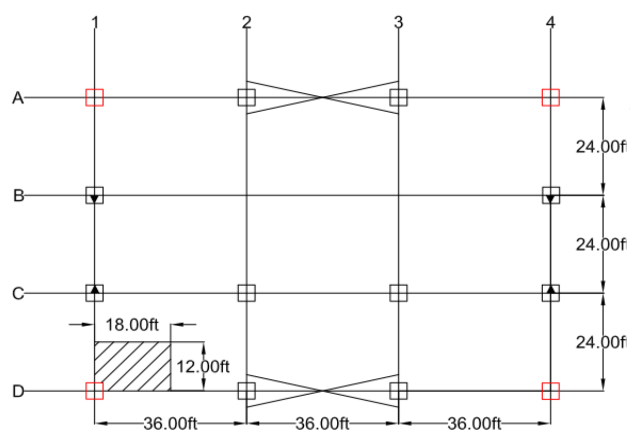


Figure 1 - First Floor Plan

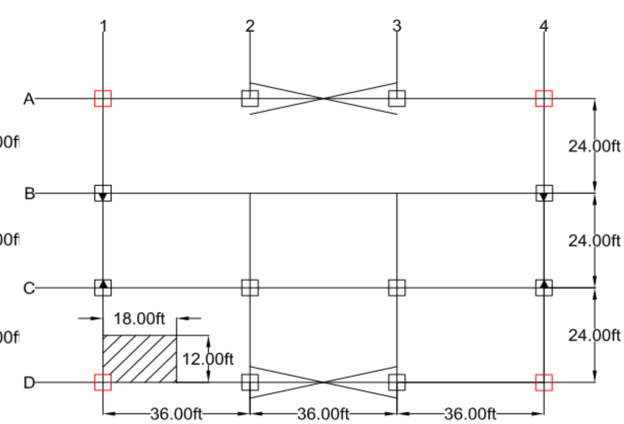


Figure 2 - Roof Floor Plan

ASSUMPTIONS:

Cummulative loads control design
South Side controls design for symmetrical frame design

				Reference: Section Information	Excel /Fig/Table/Not	

1. FIRST FLOOR

Member Ref:		1		19	*From Summary Sheet*		
Frame:		Braced					
Floor:		First					
Beam Length	$L =$	15	ft		*Project Information*		
Distance a	$a =$	12	ft		*AISC 14th*	Table	3-23

Support Condition		Simply				
Load Condition		Uniform		*AISC 14th*	Table	3-23
Case		1		*AISC 14th*	Table	3-24
Tributary Area	A =	216	ft^2	*Per figure 1 and 2*		

Loads:

Dead Load	DL =	154	psf
Live Load	LL =	120	psf
Snow Load	S =	31.5	psf
Wind Load	W =	-23.5	psf
Seismic Load	E =		psf

LRFD:	**1st Floor**	
1. 1.4D	215.6	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	392.6	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5W)	355.2	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	177.1	psf
5. 1.2D + 1.0E + L + .2S	311.1	psf
6. 0.9D + 1.0W	115.1	psf
7. 0.9D + 1.0E	138.6	psf

Controlling Load:		392.6	psf
Control:	P_u	84.8	kip

Buckling Analysis:

	K =	1		AISC 14th	Table	C-A-7.1
	KL =	15	ft			
	KL/r =	60	ft	*Assumed*		
	$\phi F_{cr} =$	34.6	ksi	AISC 14th	Table	4-22
Demand Value:	$A_{reqd} =$	2.45	in^2			

Beam Selection,	**W:**		**W8X40**				
		$r_y =$	2.04	in^3			
		A =	11.7	in^2			
		KL/r =	88.24				
		$\phi F_{cr} =$	25.2	ksi	AISC 14th	Table	4-22
		$\phi P_n =$	294.84	kips			

Design Check:		$\phi P_n > P_{u?}$	**YES**
DCR	**Compressive**	0.29	OK

Demand Capacity Ratio

GIVEN:

Number of Floors: 2

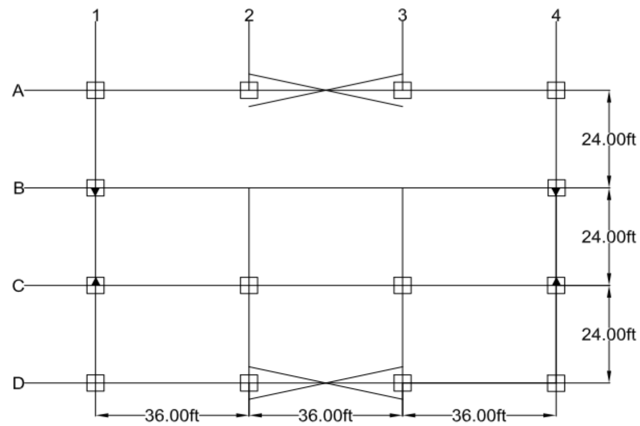


Figure 1 - First Floor Plan

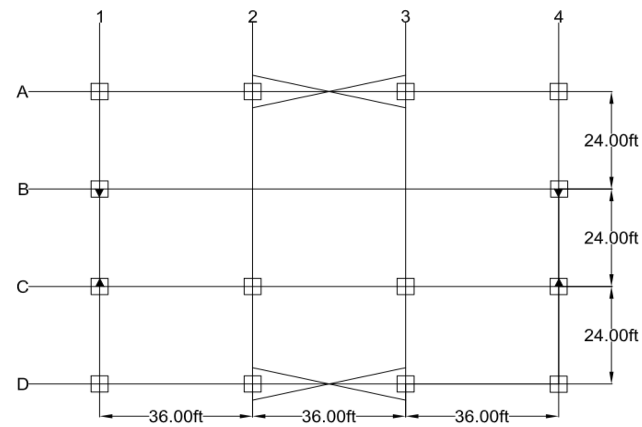


Figure 2 - Roof Floor Plan

ASSUMPTIONS:

Beams running parallel to moment frame for both levels

					Reference: Section Information	Excel *Eq/Fig/Table/Notes*	
2. ROOF							
Member Ref:		40			*From Summary Sheet*		
Frame:		Interior					
Floor:		Roof					
Beam Length	L =	24	ft		*Project Information*		
Beam Spacing	s =	0	ft	O.C.	*Enter chosen spacing for beams*		
Support Condition		Simply					
Load Condition		Uniform			*AISC 14th*	Table	3-23
Case		1			*AISC 14th*	Table	3-24

Loads:

Dead Load	DL =	26	psf	*Project Loads*
Steel Selected	w =	19	psf	*Project Loads*
Total Dead Load	TDL =	45	psf	*Per Beam Selection Below*
Live Load	LL =	20	psf	*Project Loads*
Snow Load	S =	31.5	psf	*Project Loads*
Wind Load	W =	-23.0	psf	*Project Loads*
Seismic Load	E =	0.0	psf	

LRFD:	**1st Floor**	
1. 1.4D	63.0	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	101.8	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5V	124.4	psf

4. 1.2D + 1.0W + L + .5(Lr or S or R)	46.8	psf
5. 1.2D + 1.0E + L + .2S	80.3	psf
6. 0.9D + 1.0W	17.5	psf
7. 0.9D + 1.0E	40.5	psf

Controlling Load: 124.4 psf
Equivalent Linear Load: 0.0 plf *Given spacing chosen above*

Demand Values:

Ultimate Moment,	M_u =	**0.0**	kip.ft	*Given uniform distribued factored load*
Ultimate Shear,	V_u =	**0.0**	kip.ft	*Given uniform distribued factored load*

						3-2
Beam Selection,	**W:**	**W12X19**				3-2
	ϕM_n	**92.6**	kip.ft	AISC	Table	
	ϕV_n	**86.0**	kip	AISC	Table	
Beam Depth:	d	**12.2**	in			

Design Check:	$\phi M_n > M_u$?	**YES**
	$\phi V_n > V_u$?	**YES**
Depth Clearance:		**OK**

DCR	**Moment**	0.00	OK	*Demand Capacity Ratio*
	Shear	0.00	OK	

MOMENT FRAME

CASE 1: GIRDER CD/AB ALONG COLUMN LINES 1 AND 4

Number of Floors: 2

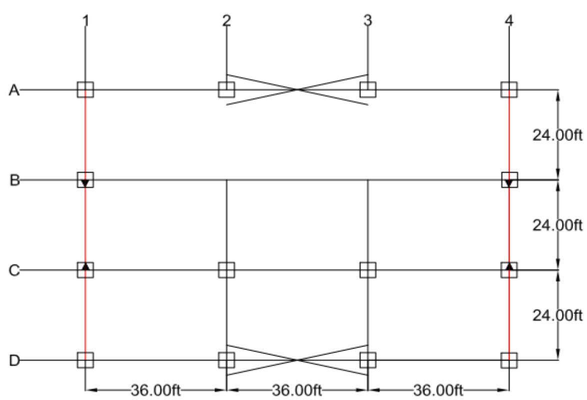


Figure 1 - First Floor Plan

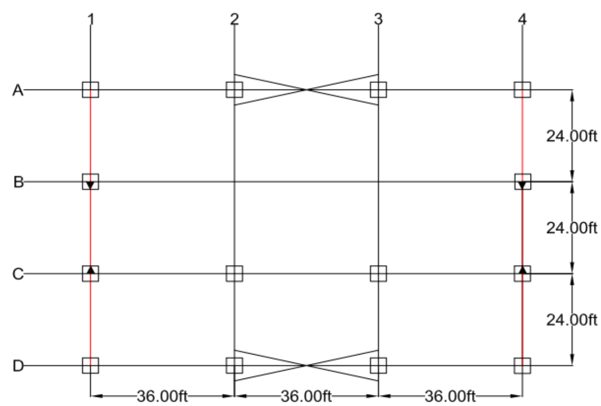
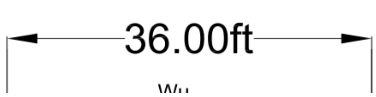


Figure 2 - Roof Floor Plan

ASSUMPTIONS:



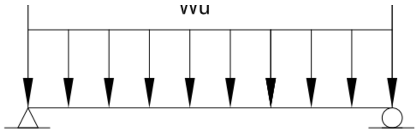


Figure 3 - Girder Loads Level 1 and Roof

				Reference: Section Information	Excel Eq/Fig/Table/Notes	
1. FIRST FLOOR						
Member Ref:		27		*From Summary Sheet*		
Frame:		Moment				
Floor:		Roof				
Beam Length	L =	24	ft	*Project Information*		
Distance a	a =	8	ft	*AISC 14th*	Table	3-23
Support Condition		Simply				
Load Condition		Uniform		*AISC 14th*	Table	3-23
Case		1		*AISC 14th*	Table	3-24
Beam Tributary Area	A	288	ft^2	*Enter chosen spacing for beams*		

Loads:						
Dead Load	DL =	26	psf	*Project Loads*		
Steel Selected	w =	16	psf			
Total Dead Load	TDL =	42	psf			
Live Load	LL =	20	psf			
Snow Load	S =	31.5	psf			
Wind Load	W =	-23.0	psf			
Seismic Load	E =	0.0	psf			

LRFD:	**1st Floor**	
1. 1.4D	58.8	psf
2. 1.2D + 1.6L + .5(Lr or S or R)	98.2	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5V	120.8	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)	43.2	psf
5. 1.2D + 1.0E + L + .2S	76.7	psf
6. 0.9D + 1.0W	14.8	psf
7. 0.9D + 1.0E	37.8	psf

Controlling Load:		120.8	psf	*Given spacing chosen above*		
Equivalent Linear 1/2 Load:		483.2	plf			
Demand Values:						
Ultimate Moment,	M_u =	**34.8**	kip.ft	*Given load condition specified above*		
Ultimate Shear,	V_u =	**5.8**	kip	*Given load condition specified above*		
Beam Selection,	**W:**	**W12X16**				
	ϕM_n	**75.4**	kip.ft	AISC	Table	3-2
	ϕV_n	**79.2**	kip	AISC	Table	3-2
Beam Depth:	d	**12.0**	in			

Design Check:	$\phi M_n>M_u$?	**YES**		
	$\phi V_n>V_u$?	**YES**		
Depth Clearance:		**OK**		
DCR	**Moment**	0.46	OK	*Demand Capacity Ratio*
	Shear	0.07	OK	

CASE 2: BRACED GIRDER BC ALONG COLUMN LINES 1 AND 4

Number of Floors: 2

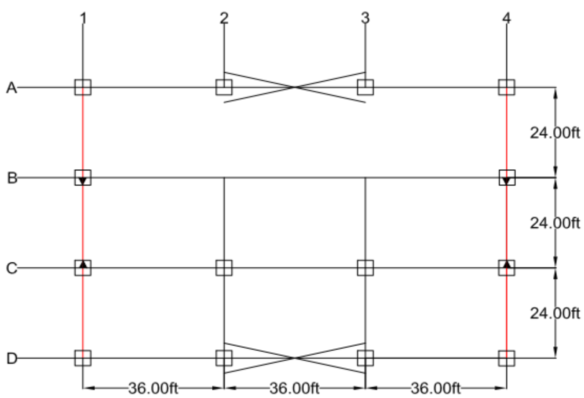


Figure 1 - First Floor Plan

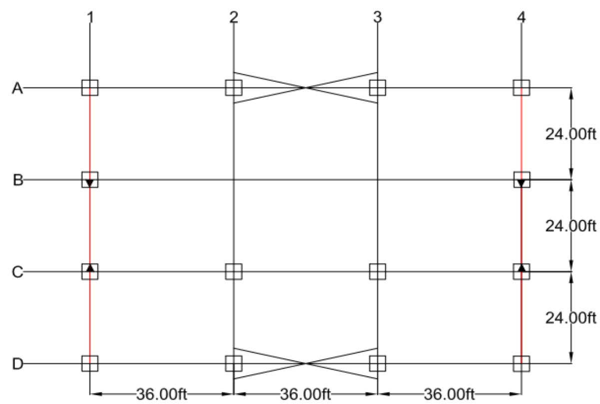


Figure 2 - Roof Floor Plan

ASSUMPTIONS:

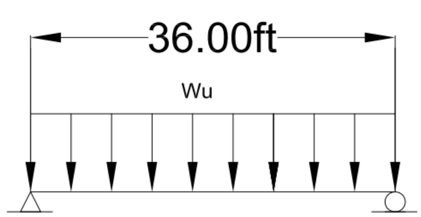


Figure 3 - Girder Loads Level 1 and Roof

				Reference:	Excel	
1. FIRST FLOOR				Section Information	*Eq/Fig/Table/Notes*	
Member Ref:		26		*From Summary Sheet*		
Frame:		Moment				
Floor:		Roof				
Beam Length	L =	24	ft	*Project Information*		
Distance a	a =	8	ft	*AISC 14th*	Table	3-23
Support Condition		Simply				
Load Condition		Uniform		*AISC 14th*	Table	3-23

Case		1		*AISC 14th*	Table	3-24
Beam Tributary Area	A	288	ft^2	*Enter chosen spacing for beams*		

Loads:

Dead Load	DL =	26	psf	*Project Loads*
Steel Selected	w =	35	psf	
Total Dead Load	TDL =	61	psf	
Live Load	LL =	20	psf	*Project Loads*
Snow Load	S =	31.5	psf	*Project Loads*
Wind Load	W =	-23.0	psf	*Project Loads*
Seismic Load	E =		psf	

End Moments per Load:

Dead Load	$M_{DL} =$	35.1	kip.ft	
Live Load	$M_{LL} =$	11.5	kip.ft	
Snow Load	$M_{SNOW} =$	18.1	kip.ft	
Wind Load	$M_{WIND} =$	146.8	kip.ft	*From Moment Frame-Wind Calculations*
Seismic Load	$M_{seismic} =$	125.3	kip.ft	*From Moment Frame-Seismic Calculations*

LRFD:	**1st Floor**	
1. 1.4D	49.2	kip.ft
2. 1.2D + 1.6L + .5(Lr or S or R)	69.7	kip.ft
3. 1.2D + 1.6(Lr or S or R) + (L or .5V	144.6	kip.ft
4. 1.2D + 1.0W + L + .5(Lr or S or R)	198.0	kip.ft
5. 1.2D + 1.0E + L + .2S	182.6	kip.ft
6. 0.9D + 1.0W	178.4	kip.ft
7. 0.9D + 1.0E	156.9	kip.ft

Controlling Moment:		**198.0**	kip.ft	
Ultimate Shear,	$V_u =$	**33.0**	kip	*Given load and support conditions above*

Beam Selection,	**W:**	**W18X35**				
	ϕM_n	**249.0**	kip.ft	AISC	Table	3-2
	ϕV_n	**159.0**	kip	AISC	Table	3-2
Beam Depth:	d	**17.7**	in			

Design Check:	$\phi M_n > M_u$?	**YES**
	$\phi V_n > V_u$?	**YES**
Depth Clearance:		**OK**

DCR	**Moment**	0.80	OK	*Demand Capacity Ratio*
	Shear	0.21	OK	

BRACED FRAME

GIVEN:

Number of Floors: 2

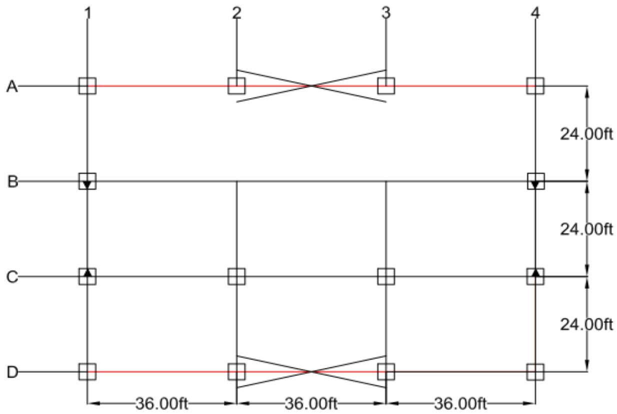


Figure 1 - First Floor Plan

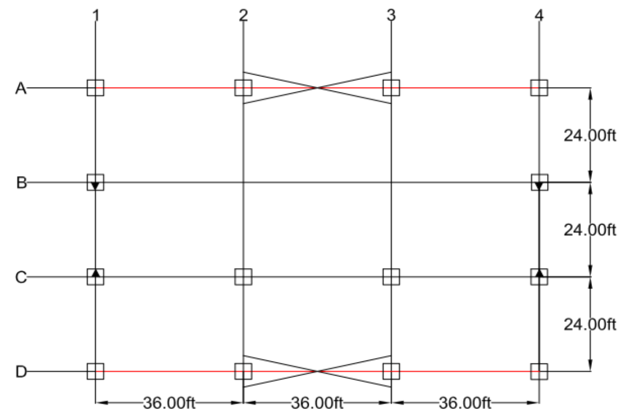


Figure 2 - Roof Floor Plan

ASSUMPTIONS:

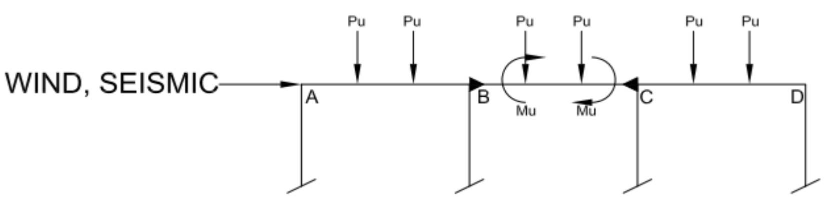


Figure 3 - Girder Loads Level 1 and Roof

					Reference:	Excel	
					Section	*Eq/Fig/Table/Notes*	
1. FIRST FLOOR					Information		
Member Ref:		13		8	*From Summary Sheet*		
Frame:		Braced					
Floor:		Roof					
Beam Length	L =	36	ft		*Project Information*		
Distance a	a =	12	ft		*AISC 14th*	Table	3-23
Support Condition		Simply					
Load Condition		2-Point			*AISC 14th*	Table	3-23
Case		9			*AISC 14th*	Table	3-24
Beam Tributary Area	A	432	ft^2		*Enter chosen spacing for beams*		
Loads:							
Dead Load	DL =	26	psf		*Project Loads*		
Steel Selected	w =	34	psf		*Project Loads*		
Total Dead Load	TDL =	60	psf		*Per Beam Selection Below*		
Live Load	LL =	20	psf				
Snow Load	S =	31.5	psf				
Wind Load	W =	-23.0	psf				
Seismic Load	E =	0.0	psf				

LRFD:		**1st Floor**	
1. 1.4D		84.0	psf
2. 1.2D + 1.6L + .5(Lr or S or R)		119.8	psf
3. 1.2D + 1.6(Lr or S or R) + (L or .5V		142.4	psf
4. 1.2D + 1.0W + L + .5(Lr or S or R)		64.8	psf
5. 1.2D + 1.0E + L + .2S		98.3	psf
6. 0.9D + 1.0W		31.0	psf
7. 0.9D + 1.0E		54.0	psf

Controlling Load:		142.4	psf			
Equivalent Linear Load:		1.14	klf	*Given spacing chosen above*		
Half Point Load:		13.67	kip			
Ultimate Moment,	M_u =	**164.0**	kip.ft	*Given uniform distribued factored load*		
Ultimate Shear,	V_u =	**13.7**	kip	*Given uniform distribued factored load*		
Beam Selection,	**W:**	**W14X34**				
	ϕM_n	**205.0**	kip.ft	AISC	Table	3-2
	ϕV_n	**120.0**	kip	AISC	Table	3-2
Beam Depth:	d	**14.0**	in			
Design Check:	$\phi M_n > M_u$?	**YES**				
	$\phi V_n > V_u$?	**YES**				
Depth Clearance:		**OK**				

DCR	**Moment**	0.80	OK	*Demand Capacity Ratio*
	Shear	0.11	OK	

INTERIOR COLUMNS:

Number of Floors: 2

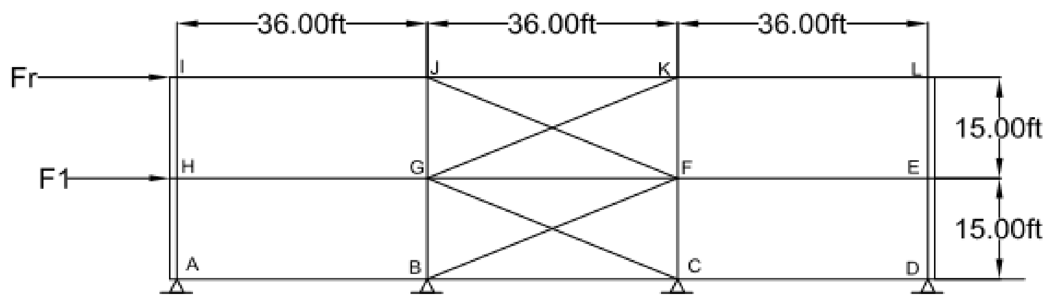


Figure 1 - Braced-Frame Elevation

ASSUMPTIONS:

			Reference: Section Information	Excel Eq/Fig/Table/Notes	
2. LOADS					
Column Length	L =	15 ft	*Project Information*		
Beam Length	L_{beam} =	36 ft			
Bracing Length	L_{brace} =	39 ft			
Support Condition		Pinned Both Ends			
Load Condition		Case 9	*AISC 14th*	Table	3-23
Member Condition		Compression			

Reference: Section Information — Excel Eq/Fig/Table/Notes

2. LOADS

Project Information

Floor	Member	Wind Load	Seismic Load	
Roof	GK (C/T)	4.00	0.40	kip
Roof	FJ (C)	14.4	4.27	kip
Level 1	BF (T)	15.60	11.50	kip
Level 1	CG (C)	15.60	11.50	kip

Controlling Load: 15.6 kip

Buckling Analysis:

K =	1		AISC 14th	Table	C-A-7.1
KL =	39	ft			
KL/r =	80	ft	*Assumed*		
ϕF_{cr} =	28.2	ksi	AISC 14th	Table	4-22

Demand Value:	$A_{reqd\,=}$	0.55	in^2			
Beam Selection,	**WT:**	**WT9X48.5**				
	$r_{y\,=}$	2.65	in^3			
	A =	14.3	in^2			
	KL/r =	176.60	in^{-2}			
	$\phi F_{cr\,=}$	6.75	ksi	AISC 14th	Table	4-22
	$\phi P_{n\,=}$	96.53	kips			
Design Check:	$\phi P_n > P_{u?}$	**YES**				

GIVEN:

Number of Floors: 2

Reference:	Excel
Section Information	*Eq/Fig/Table/Notes*

1. MEMBERS FOR ANALYSIS:

Member Ref.	Frame	Floor	Member	Section	Length	No translation Axial Load	Mom. (x)	Mom.(y)	Lateral Translation Axial Load	Mom. (x)	Mom.(y)	Lateral Deflection
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	$P_{nt\ (kip)}$	$M_{ntx\ (kip.ft)}$	$M_{nty\ (kip.ft)}$	$P_{lt\ (kip)}$	$Ml_{tx\ (kip.ft)}$	$Ml_{tx\ (kip.ft)}$	ΔH (in)
1	Braced	First	Column	W8X40	15	166	14	0	0	0	0	0
2	Braced	Roof	Column	W8X40	15							
3	Braced	First	Column	W8X40	15	166	14	0	0	0	0	0
4	Braced	Roof	Column	W8X40	15							
5	Braced	First	Column	W8X40	15							
6	Braced	Roof	Column	W8X40	15							
9	Braced	First	Beam	W21X55	36							
10	Braced	First	Interior Beam	W21X55	36							
11	Braced	First	Beam	W21X55	36							
12	Braced	Roof	Beam	W14X34	36							
13	Braced	Roof	Interior Beam	W14X34	36							
14	Braced	Roof	Beam	W14X34	36							
15	Braced	Roof	Braces	WT9X48.5	39							
16	Braced	Roof	Braces	WT9X48.5	39							
17	Braced	First	Braces	WT9X48.5	39							
18	Braced	First	Braces	WT9X48.5	39							
1	Moment	First	Column	W8X40	15							
2	Moment	Roof	Column	W8X40	15							
7	Moment	First	Column	W8X40	15							
8	Moment	Roof	Column	W8X40	15							

21	Moment	First	Column	W8X40	15							
22	Moment	Roof	Column	W8X40	15							
23	Moment	First	Column	W8X40	15	30	26	0	26	88	0	0.003
24	Moment	Roof	Column	W8X40	15							
25	Moment	Roof	Beam	W12X16	24							
26	Moment	Roof	Interior Beam	W18X35	24	2	11	0	1.35	32.7	0	0.003
27	Moment	Roof	Beam	W12X16	24							
28	Moment	First	Beam	W14X22	24							
29	Moment	First	Interior Beam	W21X44	24	5	54.3	0	4	98	0	0.003
30	Moment	First	Beam	W14X22	24							

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

2. LOADS

Member Ref.	Frame	Floor	P_{story}	P_{mf}	Lateral Shear	α	Gravity Load	Notional Load
(#)	*(type)*	*Units*	*(kip)*	*(kip)*	*(kip)*	*(LRFD)*	*(kip)*	*(kip)*
1-B	Braced	First	3643.2	607.6	31	1	214	0.43
1-M	Moment	First	3643.2	607.6	31	1	214	0.43
2-B	Braced	Roof	622.1	607.6	31	1	470	0.94
2-M	Moment	Roof	622.1	607.6	31	1	470	0.94

3. DETERMINATION OF LATERAL-TORSIONAL BUCKLING FACTOR C_b

F Eq. F1-1

Member Ref.	Frame	Floor	Member	Section	Length	M_{max}	$M_{.25}$	$M_{.5}$	$M_{.75}$	C_b	K
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(kip.ft)*	*(kip.ft)*	*(kip.ft)*	*(kip.ft)*		
1	Braced	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
3	Braced	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
9	Braced	First	Beam	W21X55	36	500.00	270.13	440.24	285.65	1.34	
7	Moment	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
8	Moment	Roof	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
23	Moment	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
24	Moment	Roof	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
26	Moment	Roof	Interior Beam	W18X35	24	500.00	270.13	440.24	285.65	1.34	
27	Moment	Roof	Beam	W12X16	24	500.00	270.13	440.24	285.65	1.34	
29	Moment	First	Interior Beam	W21X44	24	500.00	270.13	440.24	285.65	1.34	
30	Moment	First	Beam	W14X22	24	500.00	270.13	440.24	285.65	1.34	

4. PRE-DETERMINATION OF EFFECTIVE LENGTH FACTOR K

Appendix 7 Eq. C-A-7-1

Member Ref.	Frame	Floor	Member	Section	Length	Moment of Inertia	Modulus of Elasticity	Support end A	Support end B	Stiffness	Factor K
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(in^4)*	*(ksi)*	*(type)*	*(type)*	*(kip.ft)*	
1	Braced	First	Column	W8X40	15	146	29000	Pin	2/9	1960.2	0.8
2	Braced	Roof	Column	W8X40	15	146	29000	1*9	12	1960.2	
3	Braced	First	Column	W8X40	15	146	29000	Pin	4/9-10	1960.2	0.745
4	Braced	Roof	Column	W8X40	15	146	29000	3/9-10	12/13	1960.2	
9	Braced	First	Beam	W21X55	36	1140	29000	1*2/9	3-4*9-10	9566.0	0.66
10	Braced	First	Interior Beam	W21X55	36	1140	29000	Pin	Pin	9566.0	
12	Braced	Roof	Beam	W14X34	36	340	29000	Pin	Pin	2853.0	
13	Braced	Roof	Interior Beam	W14X34	36	340	29000	Pin	Pin	2853.0	
7	Moment	First	Column	W8X40	15	146	29000	Pin	8/30	1960.2	2.5
8	Moment	Roof	Column	W8X40	15	146	29000	7/30	27	1960.2	2.18
21	Moment	First	Column	W8X40	15	146	29000	Pin	21-22/28-29	1960.2	
22	Moment	Roof	Column	W8X40	15	146	29000	22-21/28-29	Pin	1960.2	
23	Moment	First	Column	W8X40	15	146	29000	Pin	24/29-30	1960.2	1.88
24	Moment	Roof	Column	W8X40	15	146	29000	23/29-30	26-27	1960.2	1.2
25	Moment	Roof	Beam	W12X16	24	103	29001	23/29-31	26-28	432.2	
26	Moment	Roof	Interior Beam	W18X35	24	510	29000	2*25	22/25-26	2139.8	1.25
27	Moment	Roof	Beam	W12X16	24	103	29000	24*26-27	8*27	432.1	1.6
28	Moment	First	Beam	W14X22	24	199	29001	1-2*28	21-22/28-29	835.0	
29	Moment	First	Interior Beam	W21X44	24	843	29000	Moment	Moment	3536.9	1.28
30	Moment	First	Beam	W14X22	24	199	29000	23-24/29-30	7*8/30	834.9	1.65

5. DETERMINATION OF EFFECTIVE LENGTH FACTOR K

Appendix 7 Eq. C-A-7-2

Member Ref.	End	Support	Column 1	Column 2	Beam 1	Beam 2	Beam 3	Beam 4	Rotational Stiffness (G)	K
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(in^4)*	*(ksi)*	*(kip.ft)*	*(kip.ft)*	

1	A	Pin					10.00	0.80
	B	2/9	2	1	9		0.41	
3		Pin					10.00	0.745
		4/9-10	4	3	9	10	0.20	
9		1*2/9	1	2	9		0.41	0.66
		3-4*9-10	3	4	9		0.41	
7		Pin					10.00	2.5
		8/30	8	7	30		4.70	
8		7/30	7	8	30		4.70	2.18
		27	8		27		4.54	
23		Pin					10.00	1.88
		24/29-30	23	24	29	30	0.90	
24		23/29-30	23	24	29	30	0.90	1.2
		26-27	24		26	27	0.76	
26		2*25	24		26	27	0.45	1.25
		22/25-26	22		25	26	0.76	
27		24*26-27	24		26	27	0.76	1.6
		8*27	8		27		4.54	
29		Moment	22	21	28	29	0.90	1.28
		Moment	23	24	29	30	0.90	
30		23-24/29-30	24	23	30	29	0.90	1.65
		7*8/30	7	8	30		4.70	

6. RESULTS

F

Member Ref.	Frame	Floor	Member	Pre-Section	Length	New-Section	Unit Weight	Spacing or a	Beams/Bay	Qty of Members	Amount of Steel
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(Shape)*	*(plf)*	*(ft)*	*(Units)*	*(Units)*	*(kips)*
1	Braced	First	Column	W8X40	15	**W8X40**	40	12.0	0.0	1.20	1.50
2	Braced	Roof	Column	W8X40	15	W8X40	40	1.7	0.0	1.20	1.50
3	Braced	First	Column	W8X40	15	**W8X40**	40	5.0	0.0	1.20	1.50
4	Braced	Roof	Column	W8X40	15	**W8X40**	40	0.0	0.0	1.20	1.20
5	Braced	First	Column	W8X40	15	W8X40	40	0.0	0.0	1.20	1.50
6	Braced	Roof	Column	W8X40	15	W8X40	40	0.0	0.0	1.20	1.50
9	Braced	First	Beam	W21X55	36	**W21X55**	55	0.0	0.0	2.16	2.16
10	Braced	First	Interior Beam	W21X55	36	W21X55	55	0.0	0.0	2.16	2.16
11	Braced	First	Beam	W21X55	36	W21X55	55	0.0	0.0	2.16	2.16
12	Braced	Roof	Beam	W14X34	36	W14X34	34	0.0	0.0	1.37	1.37
13	Braced	Roof	Interior Beam	W14X34	36	W14X34	34	0.0	0.0	1.37	1.37
14	Braced	Roof	Beam	W14X34	36	W14X34	34	0.0	0.0	1.37	1.37
15	Braced	Roof	Braces	WT9X48.5	39	W10X33	33	0.0	0.0	3.78	2.57
16	Braced	Roof	Braces	WT9X48.5	39	W10X33	33	0.0	0.0	3.78	2.57
17	Braced	First	Braces	WT9X48.5	39	**W10X33**	33	0.0	0.0	3.78	2.57
18	Braced	First	Braces	WT9X48.5	39	**W10X33**	33	0.0	0.0	3.78	2.57
1	Moment	First	Column	W8X40	15	**W8X40**	40	12.0	0.0	1.20	1.50
2	Moment	Roof	Column	W8X40	15	**W8X40**	40	1.7	0.0	1.20	1.50
7	Moment	First	Column	W8X40	15	W8X40	40	0.0	0.0	1.20	1.50
8	Moment	Roof	Column	W8X40	15	W8X40	40	0.0	0.0	1.20	1.50
21	Moment	First	Column	W8X40	15	W8X40	40	0.0	0.0	1.17	2.04
22	Moment	Roof	Column	W8X40	15	W8X40	40	0.0	0.0	1.17	2.04
23	Moment	First	Column	W8X40	15	W8X40	40	0.0	0.0	1.17	2.04
24	Moment	Roof	Column	W8X40	15	W8X40	40	5.0	0.0	1.17	2.04
25	Moment	Roof	Beam	W12X16	24	**W12X16**	16	8.0	0.0	1.44	1.44
26	Moment	Roof	Interior Beam	W18X35	24	W18X35	35	0.0	0.0	1.92	1.87
27	Moment	Roof	Beam	W12X16	24	W12X16	16	0.0	0.0	1.44	1.44
28	Moment	First	Beam	W14X22	24	**W14X22**	22	0.0	0.0	1.92	1.92
29	Moment	First	Interior Beam	W21X44	24	W21X44	44	0.0	0.0	1.92	2.54
30	Moment	First	Beam	W14X22	24	W14X22	22	0.0	0.0	1.92	1.92

7. CONNECTIONS

J

Member Ref.	Frame	Floor	Member	Pre-Section	Length	New-Section	Unit Weight	Spacing or a	Beams/Bay	Qty of Members	Amount of Steel
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(Shape)*	*(plf)*	*(ft)*	*(Units)*	*(Units)*	*(kips)*
0	0	0	0	0	0	**#N/A**	#N/A	#N/A	#N/A	#N/A	#N/A
0	0	0	0	0	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	0	0	0	0	0	**W12X50**	50	#N/A	#N/A	#N/A	#N/A

COLUMN-CAPACITY

Member Ref:	1
Frame:	Braced
Floor:	First
Member:	Column
Ref. 2:	1-B

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	15	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	15	ft	Global or Local System?
Unbraced Length, y:	$L_{by} =$	15	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		

Reference: Excel

		Section	*Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Information

Section:	W	**W8X40**					
Member is in:		Compression					
Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	d =	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
T	T =	0	in	Warping Constant	$C_w =$	726	in

3. PRELIMINARY ANALYSIS

Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x =$	51.0			
	$(KL/r)_y =$	88.2	AISC	Table	3-2
	$(KL)_z =$	180.0	AISC	Table	3-2
Largest Possible Ratio:		88.2			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-2**			

4. LOCAL SLENDERNESS CHECK:

Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	17.6	7.21
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	Nonslender	Nonslender

Reference: AISC 14th

		Section	*Eq/Fig/Table/Notes*

5. BUCKLING ANALYSIS:

E

Euler Buckling Stress:	$F_{e3} =$	36.8	ksi	*Eq. E3-4*
Torsional Buckling Stress:	$F_{e4} =$	97.2	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe3 =	36.8	ksi	
Critical Buckling Stress:	$F_{cr} =$	28.3	ksi	*Eq. E3-2*

6. COLUMN CAPACITY:

Eq. E3-1

Compressive Strength:	$P_{n} =$	331.1	ksi	*Eq. E3-1*
Factor:	$\Phi =$	0.9		
Column Capacity:	$\Phi.P_{n} =$	298.0	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	15	ft	*Global or Local System?*
Unbraced Length, y:	$L_{by} =$	15	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor Check:		**K<1**		*Check for values below*
Eff. Length Factor, x:	$K_{x} =$	0.8		
Eff. Length Factor, y:	$K_{y} =$	0.8		
Eff. Length Factor, z:	$K_{z} =$	1		

Plastic Zones Lengths and Info:

Full plastic yield Length:	$L_{p} =$	7.2	ft
LTB Length:	$L_{r} =$	29.9	ft
	$\phi_b BF =$	2.46	kips
	$\phi_b M_{px} =$	149	kip.ft

Reference:	Excel
Section Information	*Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W8X40**
Member is in:		Compression

Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	$d =$	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_{f} =$	8.07	in
Polar Moment of Inertia:	$J_{w} =$	1.12	in^4	Flange Thickness:	$t_{f} =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_{w} =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	$A =$	11.7	in^2
Section Modulus:	$S_{x} =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	$Z =$	39.8	in^3	Distance flange/centro	$h_{0} =$	11.60	in
Plastic Section Modulus, y:	$Z_{y} =$	18.5	in	Warping Constant	$C_{w} =$	726	in
T	$T =$	0	in	Section Modulus:	$S_{y} =$	12.2	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	Flange
Flexure	Compact	Compact
Compression	Mntx =	14

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	$Z_i =$	0.428	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	$P_{nt} =$	166	kips	
Ultimate Moment, NT, x	$M_{ntx} =$	14	kip.ft	
Ultimate Moment, NT, y	$M_{nty} =$	0	kip.ft	
Ultimate Axial Load, LT	$P_{lt} =$	0	kips	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Total V. load in story	$P_{story} =$	3643.2	kip	
	$P_{mf} =$	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	$\Delta H/L$ =	0.0012		

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n =$	298.0	ksi		
Flexure Capacity					
Along axis x:	Zone =	2			
	Cb =	1.34			
Flexure Capacity, x	$M_{cx} =$	149.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	$M_{cy} =$	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.97		A-8-8
	$P_{e\text{-}story}$ =	25304.2	kip	A-8-7
	B_{2x} =	1.17		A-8-6
2nd-Order Axial Strength	P_r =	166.0	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.97		A-8-8
	$P_{e\text{-}storyY}$ =	25304.2	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	166.0	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-14.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.557			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:	3
Frame:	Braced
Floor:	First
Member:	Column
Ref. 2:	1-B

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	F_{y} =	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	L =	15	ft	15	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	L_{bx} =	0.745	ft	Global or Local System?
Unbraced Length, y:	L_{by} =	0.745	ft	
Unbraced Length, z:	L_{bz} =	15	ft	
Eff. Length Factor, x:	K_{x} =	1		
Eff. Length Factor, y:	K_{y} =	1		
Eff. Length Factor, z:	K_{z} =	1		

Reference: Excel

	Section	*Eq/Fig/Table/Notes*
3. SECTION PROPERTIES	Information	

Section:	W	**W8X40**	
Member is in:		Compression	

Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	$d =$	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	$A =$	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	$Z =$	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
T	T =	0	in	Warping Constant	$C_w =$	726	in

3. PRELIMINARY ANALYSIS	Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x =$	2.5			
	$(KL/r)_y =$	4.4	AISC	Table	3-2
	$(KL)_z =$	180.0	AISC	Table	3-2
Largest Possible Ratio:		4.4			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-2**			

4. LOCAL SLENDERNESS CHECK:	Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	17.6	7.21
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	Nonslender	Nonslender

	Reference:	AISC 14th
	Section	*Eq/Fig/Table/Notes*
5. BUCKLING ANALYSIS:	E	

Euler Buckling Stress:	$F_{e3} =$	14903.3	ksi	*Eq. E3-4*
Torsional Buckling Stress:	$F_{e4} =$	97.2	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe4 =	97.2	ksi	
Critical Buckling Stress:	$F_{cr} =$	INSERT	ksi	*Eq. E3-2*

6. COLUMN CAPACITY:	Eq. E3-1

Compressive Strength:	$P_{n\,=}$	#VALUE!	ksi	*Eq. E3-1*
Factor:	$\Phi\,_{=}$	0.9		
Column Capacity:	$\Phi.P_{n\,=}$	#VALUE!	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	F_{y} =	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	L =	15	ft	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	L_{bx} =	15	ft	*Global or Local System?*
Unbraced Length, y:	L_{by} =	15	ft	
Unbraced Length, z:	L_{bz} =	15	ft	
Eff. Length Factor Check:		**K<1**		*Check for values below*
Eff. Length Factor, x:	K_{x} =	1		
Eff. Length Factor, y:	K_{y} =	1		
Eff. Length Factor, z:	K_{z} =	1		

Plastic Zones Lengths and Info:

Full plastic yield Length:	L_{p} =	7.2	ft
LTB Length:	L_{r} =	29.9	ft
	$\phi_b BF$ =	2.46	kips
	$\phi_b M_{px}$ =	149	kip.ft

Reference: Section Information | Excel *Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W8X40**
Member is in:		Compression

Moment of Inertia, x:	I_{xw} =	146	in^4	Depth:	d =	8.25	in
Moment of Inertia, y	I_{yw} =	49.1	in^4	Width:	b_f =	8.07	in
Polar Moment of Inertia:	J_w =	1.12	in^4	Flange Thickness:	t_f =	0.56	in
Radius of Gyration, x:	r_{xw} =	3.53	in	Web Thickness:	t_w =	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in^2
Section Modulus:	S_x =	35.5	in^3	r_{ts}	r_{ts} =	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	h_0 =	11.60	in
Plastic Section Modulus, y:	Z_y =	18.5	in	Warping Constant	C_w =	726	in
T	T =	0	in	Section Modulus:	S_y =	12.2	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	**Flange**
Flexure	Compact	Compact
Compression	Mntx =	14

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	Z_i =	0.428	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	P_{nt} =	166	kips	
Ultimate Moment, NT, x	M_{ntx} =	14	kip.ft	
Ultimate Moment, NT, y	M_{nty} =	0	kip.ft	
Ultimate Axial Load, LT	P_{lt} =	0	kips	
Ultimate Moment, LT, y	Ml_{tx} =	0	kip.ft	
Ultimate Moment, LT, y	Ml_{tx} =	0	kip.ft	
Total V. load in story	P_{story} =	3643.2	kip	
	P_{mf} =	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	ΔH/L =	0.0012		

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n$ =	#VALUE!	ksi		
Flexure Capacity					
Along axis x:	Zone =	2			
	Cb =	1.34			
Flexure Capacity, x	M_{cx} =	149.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	M_{cy} =	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.97		A-8-8
	$P_{e\text{-}story}$ =	25304.2	kip	A-8-7
	B_{2x} =	1.17		A-8-6
2nd-Order Axial Strength	P_r =	166.0	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.97		A-8-8
	$P_{e\text{-}storyY}$ =	25304.2	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	166.0	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-14.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	#VALUE!			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

GIVEN:

Number of Floors: 2

Reference: Section Information | Excel *Eq/Fig/Table/Notes*

1. MEMBERS FOR ANALYSIS:

Member Ref.	Frame	Floor	Member	Section	Length	No translation Axial Load	Mom. (x)	Mom.(y)	Lateral Translation Axial Load	Mom. (x)	Mom.(y)	Lateral Deflection
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	$P_{nt\ (kip)}$	$M_{ntx\ (kip.ft)}$	$M_{nty\ (kip.ft)}$	$P_{lt\ (kip)}$	$Ml_{tx\ (kip.ft)}$	$Ml_{tx\ (kip.ft)}$	ΔH (in)
1	Braced	First	Column	W8X40	15	130	14	0	0	0	0	0
2	Braced	Roof	Column	W8X40	15							
3	Braced	First	Column	W8X40	15	130	14	0	0	0	0	0
4	Braced	Roof	Column	W8X40	15							
5	Braced	First	Column	W8X40	15							
6	Braced	Roof	Column	W8X40	15							
9	Braced	First	Beam	W21X55	36							
10	Braced	First	Interior Beam	W21X55	36							
11	Braced	First	Beam	W21X55	36							
12	Braced	Roof	Beam	W14X34	36							
13	Braced	Roof	Interior Beam	W14X34	36							
14	Braced	Roof	Beam	W14X34	36							
15	Braced	Roof	Braces	WT9X48.5	39							
16	Braced	Roof	Braces	WT9X48.5	39							
17	Braced	First	Braces	WT9X48.5	39							
18	Braced	First	Braces	WT9X48.5	39							
1	Moment	First	Column	W8X40	15							
2	Moment	Roof	Column	W8X40	15							
7	Moment	First	Column	W8X40	15							
8	Moment	Roof	Column	W8X40	15							

21	Moment	First	Column	W8X40	15							
22	Moment	Roof	Column	W8X40	15							
23	Moment	First	Column	W8X40	15	27.2	6	0	3.4	71	0	0.003
24	Moment	Roof	Column	W8X40	15							
25	Moment	Roof	Beam	W12X16	24							
26	Moment	Roof	Interior Beam	W18X35	24	5.3	3.5	0	1	10.4	0	0.003
27	Moment	Roof	Beam	W12X16	24							
28	Moment	First	Beam	W14X22	24							
29	Moment	First	Interior Beam	W21X44	24	6.6	20	0	2	15	0	0.003
30	Moment	First	Beam	W14X22	24							

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

2. LOADS

Member Ref.	Frame	Floor	P_{story}	P_{mf}	Lateral Shear	α	Gravity Load	Notional Load
(#)	*(type)*	*Units*	*(kip)*	*(kip)*	*(kip)*	*(LRFD)*	*(kip)*	*(kip)*
1-B	Braced	First	3643.2	607.6	31	1	214	0.43
1-M	Moment	First	3643.2	607.6	31	1	214	0.43
2-B	Braced	Roof	622.1	607.6	31	1	470	0.94
2-M	Moment	Roof	622.1	607.6	31	1	470	0.94

3. DETERMINATION OF LATERAL-TORSIONAL BUCKLING FACTOR C_b

F Eq. F1-1

Member Ref.	Frame	Floor	Member	Section	Length	M_{max}	$M_{.25}$	$M_{.5}$	$M_{.75}$	C_b	K
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(kip.ft)*	*(kip.ft)*	*(kip.ft)*	*(kip.ft)*		
1	Braced	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
3	Braced	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
9	Braced	First	Beam	W21X55	36	500.00	270.13	440.24	285.65	1.34	
7	Moment	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
8	Moment	Roof	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
23	Moment	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
24	Moment	Roof	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
26	Moment	Roof	Interior Beam	W18X35	24	500.00	270.13	440.24	285.65	1.34	
27	Moment	Roof	Beam	W12X16	24	500.00	270.13	440.24	285.65	1.34	
29	Moment	First	Interior Beam	W21X44	24	500.00	270.13	440.24	285.65	1.34	
30	Moment	First	Beam	W14X22	24	500.00	270.13	440.24	285.65	1.34	

4. PRE-DETERMINATION OF EFFECTIVE LENGTH FACTOR K

Appendix 7 Eq. C-A-7-1

Member Ref.	Frame	Floor	Member	Section	Length	Moment of Inertia	Modulus of Elasticity	Support end A	Support end B	Stiffness	Factor K
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(in^4)*	*(ksi)*	*(type)*	*(type)*	*(kip.ft)*	
1	Braced	First	Column	W8X40	15	146	29000	Pin	2/9	1960.2	0.8
2	Braced	Roof	Column	W8X40	15	146	29000	1*9	12	1960.2	
3	Braced	First	Column	W8X40	15	146	29000	Pin	4/9-10	1960.2	0.745
4	Braced	Roof	Column	W8X40	15	146	29000	3/9-10	12/13	1960.2	
9	Braced	First	Beam	W21X55	36	1140	29000	1*2/9	3-4*9-10	9566.0	0.66
10	Braced	First	Interior Beam	W21X55	36	1140	29000	Pin	Pin	9566.0	
12	Braced	Roof	Beam	W14X34	36	340	29000	Pin	Pin	2853.0	
13	Braced	Roof	Interior Beam	W14X34	36	340	29000	Pin	Pin	2853.0	
7	Moment	First	Column	W8X40	15	146	29000	Pin	8/30	1960.2	2.5
8	Moment	Roof	Column	W8X40	15	146	29000	7/30	27	1960.2	2.18
21	Moment	First	Column	W8X40	15	146	29000	Pin	21-22/28-29	1960.2	
22	Moment	Roof	Column	W8X40	15	146	29000	22-21/28-29	Pin	1960.2	
23	Moment	First	Column	W8X40	15	146	29000	Pin	24/29-30	1960.2	1.88
24	Moment	Roof	Column	W8X40	15	146	29000	23/29-30	26-27	1960.2	1.2
25	Moment	Roof	Beam	W12X16	24	103	29001	23/29-31	26-28	432.2	
26	Moment	Roof	Interior Beam	W18X35	24	510	29000	2*25	22/25-26	2139.8	1.25
27	Moment	Roof	Beam	W12X16	24	103	29000	24*26-27	8*27	432.1	1.6
28	Moment	First	Beam	W14X22	24	199	29001	1-2*28	21-22/28-29	835.0	
29	Moment	First	Interior Beam	W21X44	24	843	29000	Moment	Moment	3536.9	1.28
30	Moment	First	Beam	W14X22	24	199	29000	23-24/29-30	7*8/30	834.9	1.65

5. DETERMINATION OF EFFECTIVE LENGTH FACTOR K Appendix 7 Eq. C-A-7-2

Member Ref.	End	Support	Column 1	Column 2	Beam 1	Beam 2	Beam 3	Beam 4	Rotational Stiffness (G)	K
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(in^4)*	*(ksi)*	*(kip.ft)*	*(kip.ft)*	

1	A	Pin					10.00	0.80
	B	2/9	2	1	9		0.41	
3		Pin					10.00	0.745
		4/9-10	4	3	9	10	0.20	
9		1*2/9	1	2	9		0.41	0.66
		3-4*9-10	3	4	9		0.41	
7		Pin					10.00	2.5
		8/30	8	7	30		4.70	
8		7/30	7	8	30		4.70	2.18
		27	8		27		4.54	
23		Pin					10.00	1.88
		24/29-30	23	24	29	30	0.90	
24		23/29-30	23	24	29	30	0.90	1.2
		26-27	24		26	27	0.76	
26		2*25	24		26	27	0.45	1.25
		22/25-26	22		25	26	0.76	
27		24*26-27	24		26	27	0.76	1.6
		8*27	8		27		4.54	
29		Moment	22	21	28	29	0.90	1.28
		Moment	23	24	29	30	0.90	
30		23-24/29-30	24	23	30	29	0.90	1.65
		7*8/30	7	8	30		4.70	

6. RESULTS

F

Member Ref.	Frame	Floor	Member	Pre-Section	Length	New-Section	Unit Weight	Spacing or a	Beams/Bay	Qty of Members	Amount of Steel
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(Shape)*	*(plf)*	*(ft)*	*(Units)*	*(Units)*	*(kips)*
1	Braced	First	Column	W8X40	15	**W8X40**	40	12.0	0.0	1.20	1.50
2	Braced	Roof	Column	W8X40	15	W8X40	40	1.7	0.0	1.20	1.50
3	Braced	First	Column	W8X40	15	**W8X40**	40	5.0	0.0	1.20	1.50
4	Braced	Roof	Column	W8X40	15	**W8X40**	40	0.0	0.0	1.20	1.20
5	Braced	First	Column	W8X40	15	W8X40	40	0.0	0.0	1.20	1.50
6	Braced	Roof	Column	W8X40	15	W8X40	40	0.0	0.0	1.20	1.50
9	Braced	First	Beam	W21X55	36	**W21X55**	55	0.0	0.0	2.16	2.16
10	Braced	First	Interior Beam	W21X55	36	W21X55	55	0.0	0.0	2.16	2.16
11	Braced	First	Beam	W21X55	36	W21X55	55	0.0	0.0	2.16	2.16
12	Braced	Roof	Beam	W14X34	36	W14X34	34	0.0	0.0	1.37	1.37
13	Braced	Roof	Interior Beam	W14X34	36	W14X34	34	0.0	0.0	1.37	1.37
14	Braced	Roof	Beam	W14X34	36	W14X34	34	0.0	0.0	1.37	1.37
15	Braced	Roof	Braces	WT9X48.5	39	W10X33	33	0.0	0.0	3.78	2.57
16	Braced	Roof	Braces	WT9X48.5	39	W10X33	33	0.0	0.0	3.78	2.57
17	Braced	First	Braces	WT9X48.5	39	**W10X33**	33	0.0	0.0	3.78	2.57
18	Braced	First	Braces	WT9X48.5	39	**W10X33**	33	0.0	0.0	3.78	2.57
1	Moment	First	Column	W8X40	15	**W8X40**	40	12.0	0.0	1.20	1.50
2	Moment	Roof	Column	W8X40	15	**W8X40**	40	1.7	0.0	1.20	1.50
7	Moment	First	Column	W8X40	15	W8X40	40	0.0	0.0	1.20	1.50
8	Moment	Roof	Column	W8X40	15	W8X40	40	0.0	0.0	1.20	1.50
21	Moment	First	Column	W8X40	15	W8X40	40	0.0	0.0	1.17	2.04
22	Moment	Roof	Column	W8X40	15	W8X40	40	0.0	0.0	1.17	2.04
23	Moment	First	Column	W8X40	15	W8X40	40	0.0	0.0	1.17	2.04
24	Moment	Roof	Column	W8X40	15	W8X40	40	5.0	0.0	1.17	2.04
25	Moment	Roof	Beam	W12X16	24	**W12X16**	16	8.0	0.0	1.44	1.44
26	Moment	Roof	Interior Beam	W18X35	24	W18X35	35	0.0	0.0	1.92	1.87
27	Moment	Roof	Beam	W12X16	24	W12X16	16	0.0	0.0	1.44	1.44
28	Moment	First	Beam	W14X22	24	**W14X22**	22	0.0	0.0	1.92	1.92
29	Moment	First	Interior Beam	W21X44	24	W21X44	44	0.0	0.0	1.92	2.54
30	Moment	First	Beam	W14X22	24	W14X22	22	0.0	0.0	1.92	1.92

COLUMN-CAPACITY

Member Ref:	1
Frame:	Braced
Floor:	First
Member:	Column
Ref. 2:	1-B

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	F_{y} =	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	L =	15	ft	15 *Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	L_{bx} =	15	ft	Global or Local System?
Unbraced Length, y:	L_{by} =	15	ft	
Unbraced Length, z:	L_{bz} =	15	ft	
Eff. Length Factor, x:	K_{x} =	1		
Eff. Length Factor, y:	K_{y} =	1		
Eff. Length Factor, z:	K_{z} =	1		

Reference: Excel

		Section	*Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section Information

Section: W **W8X40**
Member is in: Compression

Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	d =	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
T	T =	0	in	Warping Constant	$C_w =$	726	in

3. PRELIMINARY ANALYSIS

Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x =$	51.0			
	$(KL/r)_y =$	88.2	AISC	Table	3-2
	$(KL)_z =$	180.0	AISC	Table	3-2
Largest Possible Ratio:		88.2			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-2**			

4. LOCAL SLENDERNESS CHECK:

Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	17.6	7.21
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	Nonslender	Nonslender

		Reference:	AISC 14th
		Section	*Eq/Fig/Table/Notes*

5. BUCKLING ANALYSIS:

E

Euler Buckling Stress:	$F_{e3} =$	36.8	ksi	*Eq. E3-4*
Torsional Buckling Stress:	$F_{e4} =$	97.2	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe3 =	36.8	ksi	
Critical Buckling Stress:	$F_{cr} =$	28.3	ksi	*Eq. E3-2*

6. COLUMN CAPACITY:

Eq. E3-1

Compressive Strength:	$P_{n} =$	331.1	ksi	*Eq. E3-1*
Factor:	$\Phi =$	0.9		
Column Capacity:	$\Phi.P_{n} =$	298.0	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	*Project Information*
Column Slenderness Parameters:				
Unbraced Length, x:	$L_{bx} =$	15	ft	*Global or Local System?*
Unbraced Length, y:	$L_{by} =$	15	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor Check:		**K<1**		*Check for values below*
Eff. Length Factor, x:	$K_{x} =$	0.8		
Eff. Length Factor, y:	$K_{y} =$	0.8		
Eff. Length Factor, z:	$K_{z} =$	1		
Plastic Zones Lengths and Info:				
Full plastic yield Length:	$L_{p} =$	7.2	ft	
LTB Length:	$L_{r} =$	29.9	ft	
	$\phi_b BF =$	2.46	kips	
	$\phi_b M_{px} =$	149	kip.ft	

Reference: Section Information	Excel *Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W8X40**					
Member is in:		Compression					
Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	$d =$	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_{f} =$	8.07	in
Polar Moment of Inertia:	$J_{w} =$	1.12	in^4	Flange Thickness:	$t_{f} =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_{w} =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	$A =$	11.7	in^2
Section Modulus:	$S_{x} =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	$Z =$	39.8	in^3	Distance flange/centro	$h_{0} =$	11.60	in
Plastic Section Modulus, y:	$Z_{y} =$	18.5	in	Warping Constant	$C_{w} =$	726	in
T	$T =$	0	in	Section Modulus:	$S_{y} =$	12.2	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	Flange
Flexure	Compact	Compact
Compression	Mntx =	14

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	$Z_i =$	0.428	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	$P_{nt} =$	130	kips	
Ultimate Moment, NT, x	$M_{ntx} =$	14	kip.ft	
Ultimate Moment, NT, y	$M_{nty} =$	0	kip.ft	
Ultimate Axial Load, LT	$P_{lt} =$	0	kips	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Total V. load in story	$P_{story} =$	3643.2	kip	
	$P_{mf} =$	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	$\Delta H/L$ =	0.0012		

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n =$	298.0	ksi		
Flexure Capacity					
Along axis x:	Zone =	2			
	Cb =	1.34			
Flexure Capacity, x	$M_{cx} =$	149.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	$M_{cy} =$	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.97		A-8-8
	$P_{e\text{-}story}$ =	25304.2	kip	A-8-7
	B_{2x} =	1.17		A-8-6
2nd-Order Axial Strength	P_r =	130.0	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.97		A-8-8
	$P_{e\text{-}storyY}$ =	25304.2	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	130.0	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-14.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.436			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:	3
Frame:	Braced
Floor:	First
Member:	Column
Ref. 2:	1-B

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	15	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	0.745	ft	Global or Local System?
Unbraced Length, y:	$L_{by} =$	0.745	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		

Reference: Excel

	Section	*Eq/Fig/Table/Notes*
3. SECTION PROPERTIES	Information	

Section: W **W8X40**
Member is in: Compression

Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	$d =$	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	$A =$	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
T	T =	0	in	Warping Constant	$C_w =$	726	in

3. PRELIMINARY ANALYSIS — Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x =$	2.5			
	$(KL/r)_y =$	4.4	AISC	Table	3-2
	$(KL)_z =$	180.0	AISC	Table	3-2
Largest Possible Ratio:		4.4			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-2**			

4. LOCAL SLENDERNESS CHECK: — Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	17.6	7.21
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	Nonslender	Nonslender

	Reference:	AISC 14th
	Section	*Eq/Fig/Table/Notes*
5. BUCKLING ANALYSIS:	E	

Euler Buckling Stress:	$F_{e3} =$	14903.3	ksi	*Eq. E3-4*
Torsional Buckling Stress:	$F_{e4} =$	97.2	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe4 =	97.2	ksi	
Critical Buckling Stress:	$F_{cr} =$	INSERT	ksi	*Eq. E3-2*

6. COLUMN CAPACITY: — Eq. E3-1

Compressive Strength:	$P_{n} =$	#VALUE!	ksi	*Eq. E3-1*
Factor:	$\Phi =$	0.9		
Column Capacity:	$\Phi.P_{n} =$	#VALUE!	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	$F_{y =}$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	L =	15	ft	*Project Information*
Column Slenderness Parameters:				
Unbraced Length, x:	$L_{bx =}$	15	ft	*Global or Local System?*
Unbraced Length, y:	$L_{by =}$	15	ft	
Unbraced Length, z:	$L_{bz =}$	15	ft	
Eff. Length Factor Check:		**K<1**		*Check for values below*
Eff. Length Factor, x:	$K_{x =}$	1		
Eff. Length Factor, y:	$K_{y =}$	1		
Eff. Length Factor, z:	$K_{z =}$	1		
Plastic Zones Lengths and Info:				
Full plastic yield Length:	$L_{p =}$	7.2	ft	
LTB Length:	$L_{r =}$	29.9	ft	
	$\phi_b BF =$	2.46	kips	
	$\phi_b M_{px\ =}$	149	kip.ft	

Reference: Section Information | Excel *Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W8X40**					
Member is in:		Compression					
Moment of Inertia, x:	$I_{xw =}$	146	in^4	Depth:	d =	8.25	in
Moment of Inertia, y	$I_{yw =}$	49.1	in^4	Width:	$b_{f =}$	8.07	in
Polar Moment of Inertia:	$J_{w =}$	1.12	in^4	Flange Thickness:	$t_{f =}$	0.56	in
Radius of Gyration, x:	$r_{xw =}$	3.53	in	Web Thickness:	$t_{w =}$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in^2
Section Modulus:	$S_{x =}$	35.5	in^3	r_{ts}	$r_{ts =}$	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	$h_{0 =}$	11.60	in
Plastic Section Modulus, y:	$Z_{y =}$	18.5	in	Warping Constant	$C_{w =}$	726	in
T	T =	0	in	Section Modulus:	$S_{y =}$	12.2	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	Flange
Flexure	Compact	Compact
Compression	Mntx =	14

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	$Z_i =$	0.428	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	$P_{nt} =$	130	kips	
Ultimate Moment, NT, x	$M_{ntx} =$	14	kip.ft	
Ultimate Moment, NT, y	$M_{nty} =$	0	kip.ft	
Ultimate Axial Load, LT	$P_{lt} =$	0	kips	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Total V. load in story	$P_{story} =$	3643.2	kip	
	$P_{mf} =$	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	ΔH/L =	0.0012		

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n =$	#VALUE!	ksi		
Flexure Capacity					
Along axis x:	Zone =	2			
	Cb =	1.34			
Flexure Capacity, x	$M_{cx} =$	149.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	$M_{cy} =$	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	$\tau_b =$	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	$M_1 =$	-1				
Larger 1st-O End Mom:	$M_2 =$	1				
Modif. Coefficient, x:	$C_{mx} =$	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	$P_{ex} =$	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	$B_{1x} =$	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	$\tau_b =$	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	$M_1 =$	-1				
Larger 1st-O End Mom:	$M_2 =$	1				
Modif. Coefficient, y	$C_{my} =$	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	$P_{ey} =$	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	$B_{1y} =$	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	$R_m =$	0.97		A-8-8
	$P_{e\text{-}story} =$	25304.2	kip	A-8-7
	$B_{2x} =$	1.17		A-8-6
2nd-Order Axial Strength	$P_r =$	130.0	kip	A-8-2
2nd-Order Mom. Strength	$M_{rx} =$	0.0	kip.ft	A-8-1

Along axis y:

	$R_{my} =$	0.97		A-8-8
	$P_{e\text{-}storyY} =$	25304.2	kip	A-8-7
	$B_{2y} =$	1.00		A-8-6
2nd-Order Axial Strength	$P_{ry} =$	130.0	kip	A-8-2
2nd-Order Mom. Strength	$M_{ry} =$	-14.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	$P_r/P_c =$	#VALUE!			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:	1
Frame:	Braced
Floor:	First
Member:	Column
Ref. 2:	1-B

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	15	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	15	ft	Global or Local System?
Unbraced Length, y:	$L_{by} =$	15	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		

Reference: Excel

	Section	*Eq/Fig/Table/Notes*
3. SECTION PROPERTIES	Information	

Section: W **W8X40**
Member is in: Compression

Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	d =	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
T	T =	0	in	Warping Constant	$C_w =$	726	in

3. PRELIMINARY ANALYSIS Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x =$	51.0			
	$(KL/r)_y =$	88.2	AISC	Table	3-2
	$(KL)_z =$	180.0	AISC	Table	3-2
Largest Possible Ratio:		88.2			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-2**			

4. LOCAL SLENDERNESS CHECK: Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	17.6	7.21
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	Nonslender	Nonslender

	Reference:	AISC 14th
	Section	*Eq/Fig/Table/Notes*
5. BUCKLING ANALYSIS:	E	

Euler Buckling Stress:	$F_{e3} =$	36.8	ksi	*Eq. E3-4*
Torsional Buckling Stress:	$F_{e4} =$	97.2	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe3 =	36.8	ksi	
Critical Buckling Stress:	$F_{cr} =$	28.3	ksi	*Eq. E3-2*

6. COLUMN CAPACITY: Eq. E3-1

Compressive Strength:	$P_{n} =$	331.1	ksi	*Eq. E3-1*
Factor:	$\Phi =$	0.9		
Column Capacity:	$\Phi.P_{n} =$	298.0	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	*Project Information*
Column Slenderness Parameters:				
Unbraced Length, x:	$L_{bx} =$	15	ft	*Global or Local System?*
Unbraced Length, y:	$L_{by} =$	15	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor Check:		**K<1**		*Check for values below*
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		
Plastic Zones Lengths and Info:				
Full plastic yield Length:	$L_{p} =$	7.2	ft	
LTB Length:	$L_{r} =$	29.9	ft	
	$\phi_b BF =$	2.46	kips	
	$\phi_b M_{px} =$	149	kip.ft	

Reference: Section Information | Excel *Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W8X40**					
Member is in:		Compression					
Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	$d =$	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_{f} =$	8.07	in
Polar Moment of Inertia:	$J_{w} =$	1.12	in^4	Flange Thickness:	$t_{f} =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_{w} =$	0.36	in
Radius of Gyration, y	$r_{yw} =$	2.04	in	Area:	$A =$	11.7	in^2
Section Modulus:	$S_{x} =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	$Z =$	39.8	in^3	Distance flange/centro	$h_{0} =$	11.60	in
Plastic Section Modulus, y:	$Z_{y} =$	18.5	in	Warping Constant	$C_{w} =$	726	in
T	$T =$	0	in	Section Modulus:	$S_{y} =$	12.2	in^3

3. SLENDERNESS CHARACTERISTICS:
Table B4.1a

	Web	Flange
Flexure	Compact	Compact
Compression	Mntx =	14

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS:
C2.2(b)

Notional Load:	$Z_i =$	0.428	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES:
GTS

Ultimate Axial Load, NT	$P_{nt} =$	130	kips	
Ultimate Moment, NT, x	$M_{ntx} =$	14	kip.ft	
Ultimate Moment, NT, y	$M_{nty} =$	0	kip.ft	
Ultimate Axial Load, LT	$P_{lt} =$	0	kips	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Total V. load in story	$P_{story} =$	3643.2	kip	
	$P_{mf} =$	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	$\Delta H/L$ =	0.0012		

6. MEMBER CAPACITY:
Eq. E3-1

Axial Capacity	$\phi.P_n =$	298.0	ksi		
Flexure Capacity					
Along axis x:	Zone =	2			
	Cb =	1.34			
Flexure Capacity, x	$M_{cx} =$	149.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	$M_{cy} =$	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS:
C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.97		A-8-8
	$P_{e\text{-story}}$ =	25304.2	kip	A-8-7
	B_{2x} =	1.17		A-8-6
2nd-Order Axial Strength	P_r =	130.0	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.97		A-8-8
	$P_{e\text{-storyY}}$ =	25304.2	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	130.0	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-14.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.436			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:	3
Frame:	Braced
Floor:	First
Member:	Column
Ref. 2:	1-B

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y\,=}$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	15	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx\,=}$	15	ft	Global or Local System?
Unbraced Length, y:	$L_{by\,=}$	15	ft	
Unbraced Length, z:	$L_{bz\,=}$	15	ft	
Eff. Length Factor, x:	$K_{x\,=}$	1		
Eff. Length Factor, y:	$K_{y\,=}$	1		
Eff. Length Factor, z:	$K_{z\,=}$	1		

Reference: Excel

	Section	Eq/Fig/Table/Notes
3. SECTION PROPERTIES	Information	

Section: W **W8X40**
Member is in: Compression

Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	d =	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
T	T =	0	in	Warping Constant	$C_w =$	726	in

3. PRELIMINARY ANALYSIS Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x =$	51.0			
	$(KL/r)_y =$	88.2	AISC	Table	3-2
	$(KL)_z =$	180.0	AISC	Table	3-2
Largest Possible Ratio:		88.2			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-2**			

4. LOCAL SLENDERNESS CHECK: Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	17.6	7.21
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	Nonslender	Nonslender

	Reference:	AISC 14th
	Section	Eq/Fig/Table/Notes
5. BUCKLING ANALYSIS:	E	

Euler Buckling Stress:	$F_{e3} =$	36.8	ksi	*Eq. E3-4*
Torsional Buckling Stress:	$F_{e4} =$	97.2	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe3 =	36.8	ksi	
Critical Buckling Stress:	$F_{cr} =$	28.3	ksi	*Eq. E3-2*

6. COLUMN CAPACITY: Eq. E3-1

Compressive Strength:	P_{n} =	331.1	ksi	*Eq. E3-1*
Factor:	Φ =	0.9		
Column Capacity:	$\Phi.P_{n}$ =	298.0	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	15	ft	*Global or Local System?*
Unbraced Length, y:	$L_{by} =$	15	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor Check:		**K<1**		*Check for values below*
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		

Plastic Zones Lengths and Info:

Full plastic yield Length:	$L_{p} =$	7.2	ft
LTB Length:	$L_{r} =$	29.9	ft
	$\phi_b BF =$	2.46	kips
	$\phi_b M_{px} =$	149	kip.ft

Reference: Section Information | Excel *Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W8X40**	
Member is in:		Compression	

Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	$d =$	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	$A =$	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	$Z =$	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
Plastic Section Modulus, y:	$Z_y =$	18.5	in	Warping Constant	$C_w =$	726	in
T	$T =$	0	in	Section Modulus:	$S_y =$	12.2	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	**Flange**
Flexure	Compact	Compact
Compression	Mntx =	14

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	Z_i =	0.428	kip		Eq.	C2-1
Second/First order drift ratio:		2	in			
Is it applied at all levels in all combinations?			**YES**	Ref. to C.2.3(3)		

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	P_{nt} =	130	kips	
Ultimate Moment, NT, x	M_{ntx} =	14	kip.ft	
Ultimate Moment, NT, y	M_{nty} =	0	kip.ft	
Ultimate Axial Load, LT	P_{lt} =	0	kips	
Ultimate Moment, LT, y	Ml_{tx} =	0	kip.ft	
Ultimate Moment, LT, y	Ml_{tx} =	0	kip.ft	
Total V. load in story	P_{story} =	3643.2	kip	
	P_{mf} =	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	$\Delta H/L$ =	0.0012		

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n$ =	298.0	ksi		
Flexure Capacity					
Along axis x:	Zone =	2			
	Cb =	1.34			
Flexure Capacity, x	M_{cx} =	149.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	M_{cy} =	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.97		A-8-8
	$P_{e\text{-story}}$ =	25304.2	kip	A-8-7
	B_{2x} =	1.17		A-8-6
2nd-Order Axial Strength	P_r =	130.0	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.97		A-8-8
	$P_{e\text{-storyY}}$ =	25304.2	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	130.0	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-14.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.436			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:	23
Frame:	Moment
Floor:	First
Member:	Column
Ref. 2:	1-M

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	15	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	15	ft	Global or Local System?
Unbraced Length, y:	$L_{by} =$	15	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		

Reference: Excel

				Section	Eq/Fig/Table/Notes
3. SECTION PROPERTIES				Information	

Section: W **W8X40**
Member is in: Compression

Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	d =	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
T	T =	0	in	Warping Constant	$C_w =$	726	in

3. PRELIMINARY ANALYSIS — Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x =$	51.0			
	$(KL/r)_y =$	88.2	AISC	Table	3-2
	$(KL)_z =$	180.0	AISC	Table	3-2
Largest Possible Ratio:		88.2			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-2**			

4. LOCAL SLENDERNESS CHECK: — Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	17.6	7.21
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	Nonslender	Nonslender

				Reference:	AISC 14th
				Section	Eq/Fig/Table/Notes
5. BUCKLING ANALYSIS:				E	
Euler Buckling Stress:	$F_{e3} =$	36.8	ksi		*Eq. E3-4*
Torsional Buckling Stress:	$F_{e4} =$	97.2	ksi		*Eq. E4-4*
Controling Euler Stress:	Fe3 =	36.8	ksi		
Critical Buckling Stress:	$F_{cr} =$	28.3	ksi		*Eq. E3-2*

6. COLUMN CAPACITY: — Eq. E3-1

Compressive Strength:	$P_{n\,=}$	331.1	ksi	*Eq. E3-1*
Factor:	$\Phi_{\,=}$	0.9		
Column Capacity:	$\Phi.P_{n\,=}$	298.0	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi	
	$G =$	11200	ksi	
Yield Strength:	$F_y =$	50	ksi	

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	*Project Information*
Column Slenderness Parameters:				
Unbraced Length, x:	$L_{bx} =$	15	ft	*Global or Local System?*
Unbraced Length, y:	$L_{by} =$	15	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor Check:		**K>1**		*Check for values below*
Eff. Length Factor, x:	$K_x =$	1.88		
Eff. Length Factor, y:	$K_y =$	1.88		
Eff. Length Factor, z:	$K_z =$	1		
Plastic Zones Lengths and Info:				
Full plastic yield Length:	$L_p =$	7.2	ft	
LTB Length:	$L_r =$	29.9	ft	
	$\phi_b BF =$	2.46	kips	
	$\phi_b M_{px} =$	149	kip.ft	

3. SECTION PROPERTIES

Reference: Section Information | Excel *Eq/Fig/Table/Notes*

Section:	W	**W8X40**					
Member is in:		Compression					
Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	$d =$	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	$A =$	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	$Z =$	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
Plastic Section Modulus, y:	$Z_y =$	18.5	in	Warping Constant	$C_w =$	726	in
T	$T =$	0	in	Section Modulus:	$S_y =$	12.2	in^3

3. SLENDERNESS CHARACTERISTICS:
Table B4.1a

	Web	Flange
Flexure	Compact	Compact
Compression	Mntx =	26

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS:
C2.2(b)

Notional Load:	$Z_i =$	0.428	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES:
GTS

Ultimate Axial Load, NT	$P_{nt} =$	30	kips	
Ultimate Moment, NT, x	$M_{ntx} =$	26	kip.ft	
Ultimate Moment, NT, y	$M_{nty} =$	0	kip.ft	
Ultimate Axial Load, LT	$P_{lt} =$	26	kips	
Ultimate Moment, LT, y	$Ml_{tx} =$	88	kip.ft	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Total V. load in story	$P_{story} =$	3643.2	kip	
	$P_{mf} =$	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	ΔH/L =	0.0012		

6. MEMBER CAPACITY:
Eq. E3-1

Axial Capacity	$\phi.P_n =$	298.0	ksi		
Flexure Capacity					
Along axis x:	Zone =	2			
	Cb =	1.34			
Flexure Capacity, x	$M_{cx} =$	149.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	$M_{cy} =$	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS:
C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.97		A-8-8
	$P_{e\text{-}story}$ =	25304.2	kip	A-8-7
	B_{2x} =	1.17		A-8-6
2nd-Order Axial Strength	P_r =	60.4	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.97		A-8-8
	$P_{e\text{-}storyY}$ =	25304.2	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	56.0	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-26.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.203			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:	26
Frame:	Moment
Floor:	Roof
Member:	Interior Beam
Ref. 2:	2-M

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	24	ft	24	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	24	ft	Global or Local System?
Unbraced Length, y:	$L_{by} =$	24	ft	
Unbraced Length, z:	$L_{bz} =$	24	ft	
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		

Reference: Excel

				Section	Eq/Fig/Table/Notes

3. SECTION PROPERTIES

Information

Section: W **W18X35**

Member is in: Compression

Moment of Inertia, x:	I_{xw} =	510	in^4	Depth:	d =	17.7	in
Moment of Inertia, y	I_{yw} =	15.3	in^4	Width:	b_f =	6	in
Polar Moment of Inertia:	J_w =	0.506	in^4	Flange Thickness:	t_f =	0.425	in
Radius of Gyration, x:	r_{xw} =	7.04	in	Web Thickness:	t_w =	0.3	in
Radius of Gyration, y	ryw =	1.22	in	Area:	A =	10.3	in^2
Section Modulus:	S_x =	57.6	in^3	r_{ts}	r_{ts} =	2.81	in
Plastic Section Modulus, x:	Z =	66.5	in^3	Distance flange/centro	h_0 =	11.60	in
T	T =	0	in	Warping Constant	C_w =	1140	in

3. PRELIMINARY ANALYSIS

Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x$ =	40.9			
	$(KL/r)_y$ =	236.1	AISC	Table	3-2
	$(KL)_z$ =	288.0	AISC	Table	3-2
Largest Possible Ratio:		236.1			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-3**			

4. LOCAL SLENDERNESS CHECK:

Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	53.5	7.06
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	N.G	Nonslender

Reference:	AISC 14th
Section	*Eq/Fig/Table/Notes*

5. BUCKLING ANALYSIS:

E

Euler Buckling Stress:	F_{e3} =	5.1	ksi	*Eq. E3-4*
Torsional Buckling Stress:	F_{e4} =	18.3	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe3 =	5.1	ksi	
Critical Buckling Stress:	F_{cr} =	0.8	ksi	*Eq. E3-2*

6. COLUMN CAPACITY:

Eq. E3-1

Compressive Strength:	$P_{n} =$	8.8	ksi	*Eq. E3-1*
Factor:	$\Phi =$	0.9		
Column Capacity:	$\Phi.P_{n} =$	7.9	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_y =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	24	ft	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	24	ft	*Global or Local System?*
Unbraced Length, y:	$L_{by} =$	24	ft	
Unbraced Length, z:	$L_{bz} =$	24	ft	
Eff. Length Factor Check:		**K>1**		*Check for values below*
Eff. Length Factor, x:	$K_x =$	1.25		
Eff. Length Factor, y:	$K_y =$	1.25		
Eff. Length Factor, z:	$K_z =$	1		

Plastic Zones Lengths and Info:

Full plastic yield Length:	$L_p =$	4.31	ft
LTB Length:	$L_r =$	12.3	ft
	$\phi_b BF =$	12.3	kips
	$\phi_b M_{px} =$	249	kip.ft

Reference:	Excel
Section Information	*Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section: W **W18X35**

Member is in: Compression

Moment of Inertia, x:	$I_{xw} =$	510	in^4	Depth:	$d =$	17.7	in
Moment of Inertia, y	$I_{yw} =$	15.3	in^4	Width:	$b_f =$	6	in
Polar Moment of Inertia:	$J_w =$	0.506	in^4	Flange Thickness:	$t_f =$	0.425	in
Radius of Gyration, x:	$r_{xw} =$	7.04	in	Web Thickness:	$t_w =$	0.3	in
Radius of Gyration, y	$r_{yw} =$	1.22	in	Area:	$A =$	10.3	in^2
Section Modulus:	$S_x =$	57.6	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	$Z =$	66.5	in^3	Distance flange/centro	$h_0 =$	11.60	in
Plastic Section Modulus, y:	$Z_y =$	8.06	in	Warping Constant	$C_w =$	1140	in
T	$T =$	0	in	Section Modulus:	$S_y =$	5.12	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	Flange
Flexure	Compact	Compact
Compression	Mntx =	11

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	Z_i =	0.94	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	P_{nt} =	2	kips
Ultimate Moment, NT, x	M_{ntx} =	11	kip.ft
Ultimate Moment, NT, y	M_{nty} =	0	kip.ft
Ultimate Axial Load, LT	P_{lt} =	1.35	kips
Ultimate Moment, LT, y	Ml_{tx} =	32.7	kip.ft
Ultimate Moment, LT, y	Ml_{tx} =	0	kip.ft
Total V. load in story	P_{story} =	622.1	kip
	P_{mf} =	607.6	kip
Story Shear in Direction of	H =	31	kip
	α =	1	*LRFD*
Lateral Deflection	ΔH =	0.215	in
Fact. Story Drift Limit	$\Delta H/L$ =	0.0007	

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n$ =	7.9	ksi		
Flexure Capacity					
Along axis x:	Zone =	3			
	Cb =	1.34			
Flexure Capacity, x	M_{cx} =	249.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	M_{cy} =	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.85		A-8-8
	$P_{e\text{-}story}$ =	35441.9	kip	A-8-7
	B_{2x} =	1.02		A-8-6
2nd-Order Axial Strength	P_r =	3.4	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.85		A-8-8
	$P_{e\text{-}storyY}$ =	35441.9	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	3.4	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-11.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.428			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:	29
Frame:	Moment
Floor:	First
Member:	Interior Beam
Ref. 2:	1-M

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	L =	24	ft	24	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	24	ft	Global or Local System?
Unbraced Length, y:	$L_{by} =$	24	ft	
Unbraced Length, z:	$L_{bz} =$	24	ft	
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		

Reference: Excel

	Section	Eq/Fig/Table/Notes
3. SECTION PROPERTIES	Information	

Section: W **W21X44**
Member is in: Compression

Moment of Inertia, x:	I_{xw} =	843	in^4	Depth:	d =	20.7	in
Moment of Inertia, y	I_{yw} =	20.7	in^4	Width:	b_f =	6.5	in
Polar Moment of Inertia:	J_w =	0.77	in^4	Flange Thickness:	t_f =	0.45	in
Radius of Gyration, x:	r_{xw} =	8.06	in	Web Thickness:	t_w =	0.35	in
Radius of Gyration, y	ryw =	1.26	in	Area:	A =	13	in^2
Section Modulus:	S_x =	81.6	in^3	r_{ts}	r_{ts} =	2.81	in
Plastic Section Modulus, x:	Z =	95.4	in^3	Distance flange/centro	h_0 =	11.60	in
T	T =	0	in	Warping Constant	C_w =	2110	in

3. PRELIMINARY ANALYSIS Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x$ =	35.7			
	$(KL/r)_y$ =	228.6	AISC	Table	3-2
	$(KL)_z$ =	288.0	AISC	Table	3-2
Largest Possible Ratio:		228.6			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-3**			

4. LOCAL SLENDERNESS CHECK: Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	53.6	7.22
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	N.G	Nonslender

	Reference:	AISC 14th
	Section	Eq/Fig/Table/Notes
5. BUCKLING ANALYSIS:	E	

Euler Buckling Stress:	F_{e3} =	5.5	ksi	*Eq. E3-4*
Torsional Buckling Stress:	F_{e4} =	18.4	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe3 =	5.5	ksi	
Critical Buckling Stress:	F_{cr} =	1.1	ksi	*Eq. E3-2*

6. COLUMN CAPACITY: Eq. E3-1

Compressive Strength:	$P_{n\,=}$	14.3	ksi	*Eq. E3-1*
Factor:	$\Phi_{\,=}$	0.9		
Column Capacity:	$\Phi.P_{n\,=}$	12.8	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	F_y =	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	L =	24	ft	*Project Information*
Column Slenderness Parameters:				
Unbraced Length, x:	L_{bx} =	24	ft	*Global or Local System?*
Unbraced Length, y:	L_{by} =	24	ft	
Unbraced Length, z:	L_{bz} =	24	ft	
Eff. Length Factor Check:		**K>1**		*Check for values below*
Eff. Length Factor, x:	K_x =	1.28		
Eff. Length Factor, y:	K_y =	1.28		
Eff. Length Factor, z:	K_z =	1		
Plastic Zones Lengths and Info:				
Full plastic yield Length:	L_p =	4.45	ft	
LTB Length:	L_r =	13	ft	
	$\phi_b BF$ =	16.8	kips	
	$\phi_b M_{px}$ =	358	kip.ft	

Reference:	Excel
Section Information	*Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W21X44**					
Member is in:		Compression					
Moment of Inertia, x:	I_{xw} =	843	in^4	Depth:	d =	20.7	in
Moment of Inertia, y	I_{yw} =	20.7	in^4	Width:	b_f =	6.5	in
Polar Moment of Inertia:	J_w =	0.77	in^4	Flange Thickness:	t_f =	0.45	in
Radius of Gyration, x:	r_{xw} =	8.06	in	Web Thickness:	t_w =	0.35	in
Radius of Gyration, y	ryw =	1.26	in	Area:	A =	13	in^2
Section Modulus:	S_x =	81.6	in^3	r_{ts}	r_{ts} =	2.81	in
Plastic Section Modulus, x:	Z =	95.4	in^3	Distance flange/centro	h_0 =	11.60	in
Plastic Section Modulus, y:	Z_y =	10.2	in	Warping Constant	C_w =	2110	in
T	T =	0	in	Section Modulus:	S_y =	6.37	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	**Flange**
Flexure	Compact	Compact
Compression	Mntx =	54.3

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	$Z_i =$	0.428	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	$P_{nt} =$	5	kips	
Ultimate Moment, NT, x	$M_{ntx} =$	54.3	kip.ft	
Ultimate Moment, NT, y	$M_{nty} =$	0	kip.ft	
Ultimate Axial Load, LT	$P_{lt} =$	4	kips	
Ultimate Moment, LT, y	$Ml_{tx} =$	98	kip.ft	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Total V. load in story	$P_{story} =$	3643.2	kip	
	$P_{mf} =$	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	$\Delta H/L$ =	0.0007		

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n =$	12.8	ksi		
Flexure Capacity					
Along axis x:	Zone =	3			
	Cb =	1.34			
Flexure Capacity, x	$M_{cx} =$	358.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	$M_{cy} =$	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.97		A-8-8
	$P_{e\text{-}story}$ =	40486.8	kip	A-8-7
	B_{2x} =	1.10		A-8-6
2nd-Order Axial Strength	P_r =	9.4	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.97		A-8-8
	$P_{e\text{-}storyY}$ =	40486.8	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	9.0	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-54.3	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.732			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:	23
Frame:	Moment
Floor:	First
Member:	Column
Ref. 2:	1-M

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	F_{y} =	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	L =	15	ft	15	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	L_{bx} =	15	ft	Global or Local System?
Unbraced Length, y:	L_{by} =	15	ft	
Unbraced Length, z:	L_{bz} =	15	ft	
Eff. Length Factor, x:	K_{x} =	1		
Eff. Length Factor, y:	K_{y} =	1		
Eff. Length Factor, z:	K_{z} =	1		

Reference: Excel

					Section		*Eq/Fig/Table/Notes*	
3. SECTION PROPERTIES					Information			
Section:	W	**W8X40**						
Member is in:		Compression						
Moment of Inertia, x:	I_{xw} =	146	in^4	Depth:	d =	8.25	in	
Moment of Inertia, y	I_{yw} =	49.1	in^4	Width:	b_f =	8.07	in	
Polar Moment of Inertia:	J_w =	1.12	in^4	Flange Thickness:	t_f =	0.56	in	
Radius of Gyration, x:	r_{xw} =	3.53	in	Web Thickness:	t_w =	0.36	in	
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in^2	
Section Modulus:	S_x =	35.5	in^3	r_{ts}	r_{ts} =	2.81	in	
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	h_0 =	11.60	in	
T	T =	0	in	Warping Constant	C_w =	726	in	

3. PRELIMINARY ANALYSIS			Eq. E 6-2a/b		
Slenderness Ratios:	$(KL/r)_x$ =	51.0			
	$(KL/r)_y$ =	88.2	AISC	Table	3-2
	$(KL)_z$ =	180.0	AISC	Table	3-2
Largest Possible Ratio:		88.2			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-2**			

4. LOCAL SLENDERNESS CHECK: Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	17.6	7.21
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	Nonslender	Nonslender

				Reference:	AISC 14th
				Section	*Eq/Fig/Table/Notes*
5. BUCKLING ANALYSIS:				E	
Euler Buckling Stress:	F_{e3} =	36.8	ksi		*Eq. E3-4*
Torsional Buckling Stress:	F_{e4} =	97.2	ksi		*Eq. E4-4*
Controling Euler Stress:	Fe3 =	36.8	ksi		
Critical Buckling Stress:	F_{cr} =	28.3	ksi		*Eq. E3-2*

6. COLUMN CAPACITY: Eq. E3-1

Compressive Strength:	P_{n} =	331.1	ksi	*Eq. E3-1*
Factor:	Φ =	0.9		
Column Capacity:	$\Phi.P_{n}$ =	298.0	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	*Project Information*
Column Slenderness Parameters:				
Unbraced Length, x:	$L_{bx} =$	15	ft	*Global or Local System?*
Unbraced Length, y:	$L_{by} =$	15	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor Check:		**K>1**		*Check for values below*
Eff. Length Factor, x:	$K_{x} =$	1.88		
Eff. Length Factor, y:	$K_{y} =$	1.88		
Eff. Length Factor, z:	$K_{z} =$	1		
Plastic Zones Lengths and Info:				
Full plastic yield Length:	$L_{p} =$	7.2	ft	
LTB Length:	$L_{r} =$	29.9	ft	
	$\phi_b BF =$	2.46	kips	
	$\phi_b M_{px} =$	149	kip.ft	

Reference:	Excel
Section Information	*Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W8X40**					
Member is in:		Compression					
Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	$d =$	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	$r_{yw} =$	2.04	in	Area:	$A =$	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	$Z =$	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
Plastic Section Modulus, y:	$Z_y =$	18.5	in	Warping Constant	$C_w =$	726	in
T	$T =$	0	in	Section Modulus:	$S_y =$	12.2	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	**Flange**
Flexure	Compact	Compact
Compression	Mntx =	6

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	Z_i =	0.428	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	P_{nt} =	27.2	kips	
Ultimate Moment, NT, x	M_{ntx} =	6	kip.ft	
Ultimate Moment, NT, y	M_{nty} =	0	kip.ft	
Ultimate Axial Load, LT	P_{lt} =	3.4	kips	
Ultimate Moment, LT, y	Ml_{tx} =	71	kip.ft	
Ultimate Moment, LT, y	Ml_{tx} =	0	kip.ft	
Total V. load in story	P_{story} =	3643.2	kip	
	P_{mf} =	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	ΔH/L =	0.0012		

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n$ =	298.0	ksi		
Flexure Capacity					
Along axis x:	Zone =	2			
	Cb =	1.34			
Flexure Capacity, x	M_{cx} =	149.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	M_{cy} =	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.97		A-8-8
	$P_{e\text{-}story}$ =	25304.2	kip	A-8-7
	B_{2x} =	1.17		A-8-6
2nd-Order Axial Strength	P_r =	31.2	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.97		A-8-8
	$P_{e\text{-}storyY}$ =	25304.2	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	30.6	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-6.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.105			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:	26
Frame:	Moment
Floor:	Roof
Member:	Interior Beam
Ref. 2:	2-M

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	24	ft	24	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	24	ft	Global or Local System?
Unbraced Length, y:	$L_{by} =$	24	ft	
Unbraced Length, z:	$L_{bz} =$	24	ft	
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		

Reference: Excel

	Section	*Eq/Fig/Table/Notes*
3. SECTION PROPERTIES	Information	

Section:	W	**W18X35**					
Member is in:		Compression					
Moment of Inertia, x:	$I_{xw=}$	510	in^4	Depth:	d =	17.7	in
Moment of Inertia, y	$I_{yw=}$	15.3	in^4	Width:	$b_{f=}$	6	in
Polar Moment of Inertia:	$J_{w=}$	0.506	in^4	Flange Thickness:	$t_{f=}$	0.425	in
Radius of Gyration, x:	$r_{xw=}$	7.04	in	Web Thickness:	$t_{w=}$	0.3	in
Radius of Gyration, y	ryw =	1.22	in	Area:	A =	10.3	in^2
Section Modulus:	$S_{x=}$	57.6	in^3	r_{ts}	$r_{ts=}$	2.81	in
Plastic Section Modulus, x:	Z =	66.5	in^3	Distance flange/centro	$h_{0=}$	11.60	in
T	T =	0	in	Warping Constant	$C_{w=}$	1140	in

3. PRELIMINARY ANALYSIS Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_{x=}$	40.9			
	$(KL/r)_{y=}$	236.1	AISC	Table	3-2
	$(KL)_{z=}$	288.0	AISC	Table	3-2
Largest Possible Ratio:		236.1			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-3**			

4. LOCAL SLENDERNESS CHECK: Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	53.5	7.06
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	N.G	Nonslender

	Reference:	AISC 14th
	Section	*Eq/Fig/Table/Notes*
5. BUCKLING ANALYSIS:	E	

Euler Buckling Stress:	$F_{e3=}$	5.1	ksi	*Eq. E3-4*
Torsional Buckling Stress:	$F_{e4=}$	18.3	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe3 =	5.1	ksi	
Critical Buckling Stress:	$F_{cr=}$	0.8	ksi	*Eq. E3-2*

6. COLUMN CAPACITY: Eq. E3-1

Compressive Strength:	$P_{n\,=}$	8.8	ksi	*Eq. E3-1*
Factor:	$\Phi\,_=$	0.9		
Column Capacity:	$\Phi.P_{n\,=}$	7.9	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	
	G =	11200	ksi	
Yield Strength:	F_{y} =	50	ksi	

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	L =	24	ft	*Project Information*
Column Slenderness Parameters:				
Unbraced Length, x:	L_{bx} =	24	ft	*Global or Local System?*
Unbraced Length, y:	L_{by} =	24	ft	
Unbraced Length, z:	L_{bz} =	24	ft	
Eff. Length Factor Check:		**K>1**		*Check for values below*
Eff. Length Factor, x:	K_{x} =	1.25		
Eff. Length Factor, y:	K_{y} =	1.25		
Eff. Length Factor, z:	K_{z} =	1		
Plastic Zones Lengths and Info:				
Full plastic yield Length:	L_{p} =	4.31	ft	
LTB Length:	L_{r} =	12.3	ft	
	$\phi_b BF$ =	12.3	kips	
	$\phi_b M_{px}$ =	249	kip.ft	

Reference: Section Information — Excel *Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W18X35**					
Member is in:		Compression					
Moment of Inertia, x:	I_{xw} =	510	in^4	Depth:	d =	17.7	in
Moment of Inertia, y	I_{yw} =	15.3	in^4	Width:	b_f =	6	in
Polar Moment of Inertia:	J_w =	0.506	in^4	Flange Thickness:	t_f =	0.425	in
Radius of Gyration, x:	r_{xw} =	7.04	in	Web Thickness:	t_w =	0.3	in
Radius of Gyration, y	ryw =	1.22	in	Area:	A =	10.3	in^2
Section Modulus:	S_x =	57.6	in^3	r_{ts}	r_{ts} =	2.81	in
Plastic Section Modulus, x:	Z =	66.5	in^3	Distance flange/centro	h_0 =	11.60	in
Plastic Section Modulus, y:	Z_y =	8.06	in	Warping Constant	C_w =	1140	in
T	T =	0	in	Section Modulus:	S_y =	5.12	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	**Flange**
Flexure	Compact	Compact
Compression	Mntx =	3.5

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	Z_i =	0.94	kip		Eq.	C2-1
Second/First order drift ratio:		2	in			

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	P_{nt} =	5.3	kips	
Ultimate Moment, NT, x	M_{ntx} =	3.5	kip.ft	
Ultimate Moment, NT, y	M_{nty} =	0	kip.ft	
Ultimate Axial Load, LT	P_{lt} =	1	kips	
Ultimate Moment, LT, y	Ml_{tx} =	10.4	kip.ft	
Ultimate Moment, LT, y	Ml_{tx} =	0	kip.ft	
Total V. load in story	P_{story} =	622.1	kip	
	P_{mf} =	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	ΔH/L =	0.0007		

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n$ =	7.9	ksi		
Flexure Capacity					
Along axis x:	Zone =	3			
	Cb =	1.34			
Flexure Capacity, x	M_{cx} =	249.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	M_{cy} =	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.85		A-8-8
	$P_{e\text{-}story}$ =	35441.9	kip	A-8-7
	B_{2x} =	1.02		A-8-6
2nd-Order Axial Strength	P_r =	6.3	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.85		A-8-8
	$P_{e\text{-}storyY}$ =	35441.9	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	6.3	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-3.5	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.802			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:	29
Frame:	Moment
Floor:	First
Member:	Interior Beam
Ref. 2:	1-M

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	24	ft	24	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	24	ft	Global or Local System?
Unbraced Length, y:	$L_{by} =$	24	ft	
Unbraced Length, z:	$L_{bz} =$	24	ft	
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		

Reference: Excel

Section *Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Information

Section:	W	**W21X44**					
Member is in:		Compression					
Moment of Inertia, x:	$I_{xw} =$	843	in^4	Depth:	d =	20.7	in
Moment of Inertia, y	$I_{yw} =$	20.7	in^4	Width:	$b_f =$	6.5	in
Polar Moment of Inertia:	$J_w =$	0.77	in^4	Flange Thickness:	$t_f =$	0.45	in
Radius of Gyration, x:	$r_{xw} =$	8.06	in	Web Thickness:	$t_w =$	0.35	in
Radius of Gyration, y	ryw =	1.26	in	Area:	A =	13	in^2
Section Modulus:	$S_x =$	81.6	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	Z =	95.4	in^3	Distance flange/centro	$h_0 =$	11.60	in
T	T =	0	in	Warping Constant	$C_w =$	2110	in

3. PRELIMINARY ANALYSIS

Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x =$	35.7			
	$(KL/r)_y =$	228.6	AISC	Table	3-2
	$(KL)_z =$	288.0	AISC	Table	3-2
Largest Possible Ratio:		228.6			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-3**			

4. LOCAL SLENDERNESS CHECK:

Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	53.6	7.22
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	N.G	Nonslender

Reference: AISC 14th

Section *Eq/Fig/Table/Notes*

5. BUCKLING ANALYSIS:

E

Euler Buckling Stress:	$F_{e3} =$	5.5	ksi	*Eq. E3-4*
Torsional Buckling Stress:	$F_{e4} =$	18.4	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe3 =	5.5	ksi	
Critical Buckling Stress:	$F_{cr} =$	1.1	ksi	*Eq. E3-2*

6. COLUMN CAPACITY:

Eq. E3-1

Compressive Strength:	P_n =	14.3	ksi	*Eq. E3-1*
Factor:	Φ =	0.9		
Column Capacity:	$\Phi.P_n$ =	12.8	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	24	ft	*Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	24	ft	*Global or Local System?*
Unbraced Length, y:	$L_{by} =$	24	ft	
Unbraced Length, z:	$L_{bz} =$	24	ft	
Eff. Length Factor Check:		**K>1**		*Check for values below*
Eff. Length Factor, x:	$K_{x} =$	1.28		
Eff. Length Factor, y:	$K_{y} =$	1.28		
Eff. Length Factor, z:	$K_{z} =$	1		

Plastic Zones Lengths and Info:

Full plastic yield Length:	$L_{p} =$	4.45	ft
LTB Length:	$L_{r} =$	13	ft
	$\phi_b BF =$	16.8	kips
	$\phi_b M_{px} =$	358	kip.ft

Reference:	Excel
Section Information	*Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W21X44**
Member is in:		Compression

Moment of Inertia, x:	$I_{xw} =$	843	in^4	Depth:	$d =$	20.7	in
Moment of Inertia, y	$I_{yw} =$	20.7	in^4	Width:	$b_{f} =$	6.5	in
Polar Moment of Inertia:	$J_{w} =$	0.77	in^4	Flange Thickness:	$t_{f} =$	0.45	in
Radius of Gyration, x:	$r_{xw} =$	8.06	in	Web Thickness:	$t_{w} =$	0.35	in
Radius of Gyration, y	ryw =	1.26	in	Area:	$A =$	13	in^2
Section Modulus:	$S_{x} =$	81.6	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	$Z =$	95.4	in^3	Distance flange/centro	$h_{0} =$	11.60	in
Plastic Section Modulus, y:	$Z_{y} =$	10.2	in	Warping Constant	$C_{w} =$	2110	in
T	$T =$	0	in	Section Modulus:	$S_{y} =$	6.37	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	Flange
Flexure	Compact	Compact
Compression	Mntx =	20

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	$Z_i =$	0.428	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	$P_{nt} =$	6.6	kips	
Ultimate Moment, NT, x	$M_{ntx} =$	20	kip.ft	
Ultimate Moment, NT, y	$M_{nty} =$	0	kip.ft	
Ultimate Axial Load, LT	$P_{lt} =$	2	kips	
Ultimate Moment, LT, y	$Ml_{tx} =$	15	kip.ft	
Ultimate Moment, LT, y	$Ml_{tx} =$	0	kip.ft	
Total V. load in story	$P_{story} =$	3643.2	kip	
	$P_{mf} =$	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	ΔH/L =	0.0007		

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n =$	12.8	ksi		
Flexure Capacity					
Along axis x:	Zone =	3			
	Cb =	1.34			
Flexure Capacity, x	$M_{cx} =$	358.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	$M_{cy} =$	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.97		A-8-8
	$P_{e\text{-}story}$ =	40486.8	kip	A-8-7
	B_{2x} =	1.10		A-8-6
2nd-Order Axial Strength	P_r =	8.8	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.97		A-8-8
	$P_{e\text{-}storyY}$ =	40486.8	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	8.6	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-20.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.686			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a

GIVEN:

Number of Floors: 2

Reference: Section Information | Excel Eq/Fig/Table/Notes

1. MEMBERS FOR ANALYSIS:

Member Ref.	Frame	Floor	Member	Section	Length	No translation Axial Load	Mom. (x)	Mom.(y)	Lateral Translation Axial Load	Mom. (x)	Mom.(y)	Lateral Deflection
(#)	(type)	Units		(Shape)	(ft)	$P_{nt\ (kip)}$	$M_{ntx\ (kip.ft)}$	$M_{nty\ (kip.ft)}$	$P_{lt\ (kip)}$	$Ml_{tx\ (kip.ft)}$	$Ml_{tx\ (kip.ft)}$	ΔH (in)
1	Braced	First	Column	W8X40	15	296	14	0	0	0	0	0
2	Braced	Roof	Column	W8X40	15							
3	Braced	First	Column	W8X40	15	296	14	0	0	0	0	0
4	Braced	Roof	Column	W8X40	15							
5	Braced	First	Column	W8X40	15							
6	Braced	Roof	Column	W8X40	15							
9	Braced	First	Beam	W21X55	36							
10	Braced	First	Interior Beam	W21X55	36							
11	Braced	First	Beam	W21X55	36							
12	Braced	Roof	Beam	W14X34	36							
13	Braced	Roof	Interior Beam	W14X34	36							
14	Braced	Roof	Beam	W14X34	36							
15	Braced	Roof	Braces	WT9X48.5	39							
16	Braced	Roof	Braces	WT9X48.5	39							
17	Braced	First	Braces	WT9X48.5	39							
18	Braced	First	Braces	WT9X48.5	39							
1	Moment	First	Column	W8X40	15							
2	Moment	Roof	Column	W8X40	15							
7	Moment	First	Column	W8X40	15							
8	Moment	Roof	Column	W8X40	15							

21	Moment	First	Column	W8X40	15							
22	Moment	Roof	Column	W8X40	15							
23	Moment	First	Column	W8X40	15	57.2	32	0	29.4	159	0	0.003
24	Moment	Roof	Column	W8X40	15							
25	Moment	Roof	Beam	W12X16	24							
26	Moment	Roof	Interior Beam	W18X35	24	7.3	14.5	0	2.35	43.1	0	0.003
27	Moment	Roof	Beam	W12X16	24							
28	Moment	First	Beam	W14X22	24							
29	Moment	First	Interior Beam	W21X44	24	11.6	74.3	0	6	113	0	0.003
30	Moment	First	Beam	W14X22	24							

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

2. LOADS

Member Ref.	Frame	Floor	P_{story}	P_{mf}	Lateral Shear	α	Gravity Load	Notional Load
(#)	*(type)*	*Units*	*(kip)*	*(kip)*	*(kip)*	*(LRFD)*	*(kip)*	*(kip)*
1-B	Braced	First	3643.2	607.6	31	1	214	0.43
1-M	Moment	First	3643.2	607.6	31	1	214	0.43
2-B	Braced	Roof	622.1	607.6	31	1	470	0.94
2-M	Moment	Roof	622.1	607.6	31	1	470	0.94

3. DETERMINATION OF LATERAL-TORSIONAL BUCKLING FACTOR C_b

F Eq. F1-1

Member Ref.	Frame	Floor	Member	Section	Length	M_{max}	$M_{.25}$	$M_{.5}$	$M_{.75}$	C_b	K
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(kip.ft)*	*(kip.ft)*	*(kip.ft)*	*(kip.ft)*		
1	Braced	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
3	Braced	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
9	Braced	First	Beam	W21X55	36	500.00	270.13	440.24	285.65	1.34	
7	Moment	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
8	Moment	Roof	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
23	Moment	First	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
24	Moment	Roof	Column	W8X40	15	500.00	270.13	440.24	285.65	1.34	
26	Moment	Roof	Interior Beam	W18X35	24	500.00	270.13	440.24	285.65	1.34	
27	Moment	Roof	Beam	W12X16	24	500.00	270.13	440.24	285.65	1.34	
29	Moment	First	Interior Beam	W21X44	24	500.00	270.13	440.24	285.65	1.34	
30	Moment	First	Beam	W14X22	24	500.00	270.13	440.24	285.65	1.34	

4. PRE-DETERMINATION OF EFFECTIVE LENGTH FACTOR K

Appendix 7 Eq. C-A-7-1

Member Ref.	Frame	Floor	Member	Section	Length	Moment of Inertia	Modulus of Elasticity	Support end A	Support end B	Stiffness	Factor K
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(in^4)*	*(ksi)*	*(type)*	*(type)*	*(kip.ft)*	
1	Braced	First	Column	W8X40	15	146	29000	Pin	2/9	1960.2	0.8
2	Braced	Roof	Column	W8X40	15	146	29000	1*9	12	1960.2	
3	Braced	First	Column	W8X40	15	146	29000	Pin	4/9-10	1960.2	0.745
4	Braced	Roof	Column	W8X40	15	146	29000	3/9-10	12/13	1960.2	
9	Braced	First	Beam	W21X55	36	1140	29000	1*2/9	3-4*9-10	9566.0	0.66
10	Braced	First	Interior Beam	W21X55	36	1140	29000	Pin	Pin	9566.0	
12	Braced	Roof	Beam	W14X34	36	340	29000	Pin	Pin	2853.0	
13	Braced	Roof	Interior Beam	W14X34	36	340	29000	Pin	Pin	2853.0	
7	Moment	First	Column	W8X40	15	146	29000	Pin	8/30	1960.2	2.5
8	Moment	Roof	Column	W8X40	15	146	29000	7/30	27	1960.2	2.18
21	Moment	First	Column	W8X40	15	146	29000	Pin	21-22/28-29	1960.2	
22	Moment	Roof	Column	W8X40	15	146	29000	22-21/28-29	Pin	1960.2	
23	Moment	First	Column	W8X40	15	146	29000	Pin	24/29-30	1960.2	1.88
24	Moment	Roof	Column	W8X40	15	146	29000	23/29-30	26-27	1960.2	1.2
25	Moment	Roof	Beam	W12X16	24	103	29001	23/29-31	26-28	432.2	
26	Moment	Roof	Interior Beam	W18X35	24	510	29000	2*25	22/25-26	2139.8	1.25
27	Moment	Roof	Beam	W12X16	24	103	29000	24*26-27	8*27	432.1	1.6
28	Moment	First	Beam	W14X22	24	199	29001	1-2*28	21-22/28-29	835.0	
29	Moment	First	Interior Beam	W21X44	24	843	29000	Moment	Moment	3536.9	1.28
30	Moment	First	Beam	W14X22	24	199	29000	23-24/29-30	7*8/30	834.9	1.65

5. DETERMINATION OF EFFECTIVE LENGTH FACTOR K

Appendix 7 Eq. C-A-7-2

Member Ref.	End	Support	Column 1	Column 2	Beam 1	Beam 2	Beam 3	Beam 4	Rotational Stiffness (G)	K
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(in^4)*	*(ksi)*	*(kip.ft)*	*(kip.ft)*	

1	A	Pin					10.00	0.80
	B	2/9	2	1	9		0.41	
3		Pin					10.00	0.745
		4/9-10	4	3	9	10	0.20	
9		1*2/9	1	2	9		0.41	0.66
		3-4*9-10	3	4	9		0.41	
7		Pin					10.00	2.5
		8/30	8	7	30		4.70	
8		7/30	7	8	30		4.70	2.18
		27	8		27		4.54	
23		Pin					10.00	1.88
		24/29-30	23	24	29	30	0.90	
24		23/29-30	23	24	29	30	0.90	1.2
		26-27	24		26	27	0.76	
26		2*25	24		26	27	0.45	1.25
		22/25-26	22		25	26	0.76	
27		24*26-27	24		26	27	0.76	1.6
		8*27	8		27		4.54	
29		Moment	22	21	28	29	0.90	1.28
		Moment	23	24	29	30	0.90	
30		23-24/29-30	24	23	30	29	0.90	1.65
		7*8/30	7	8	30		4.70	

6. RESULTS

F

Member Ref.	Frame	Floor	Member	Pre-Section	Length	New-Section	Unit Weight	Spacing or a	Beams/Bay	Qty of Members	Amount of Steel
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(Shape)*	*(plf)*	*(ft)*	*(Units)*	*(Units)*	*(kips)*
1	Braced	First	Column	W8X40	15	**W12X50**	50	12.0	0.0	1.20	1.50
2	Braced	Roof	Column	W8X40	15	W12X50	50	1.7	0.0	1.20	1.50
3	Braced	First	Column	W8X40	15	**W12X50**	50	5.0	0.0	1.20	1.50
4	Braced	Roof	Column	W8X40	15	**W12X40**	40	0.0	0.0	1.20	1.20
5	Braced	First	Column	W8X40	15	W12X50	50	0.0	0.0	1.20	1.50
6	Braced	Roof	Column	W8X40	15	W12X50	50	0.0	0.0	1.20	1.50
9	Braced	First	Beam	W21X55	36	**W10X68**	68	0.0	0.0	2.16	2.16
10	Braced	First	Interior Beam	W21X55	36	W14X30	30	0.0	0.0	2.16	2.16
11	Braced	First	Beam	W21X55	36	W14X30	30	0.0	0.0	2.16	2.16
12	Braced	Roof	Beam	W14X34	36	W10X19	19	0.0	0.0	1.37	1.37
13	Braced	Roof	Interior Beam	W14X34	36	W10X19	19	0.0	0.0	1.37	1.37
14	Braced	Roof	Beam	W14X34	36	W10X19	19	0.0	0.0	1.37	1.37
15	Braced	Roof	Braces	WT9X48.5	39	W10X33	33	0.0	0.0	3.78	2.57
16	Braced	Roof	Braces	WT9X48.5	39	W10X33	33	0.0	0.0	3.78	2.57
17	Braced	First	Braces	WT9X48.5	39	**W10X33**	33	0.0	0.0	3.78	2.57
18	Braced	First	Braces	WT9X48.5	39	**W10X33**	33	0.0	0.0	3.78	2.57
1	Moment	First	Column	W8X40	15	**W10X33**	33	12.0	0.0	1.20	1.50
2	Moment	Roof	Column	W8X40	15	**W10X33**	33	1.7	0.0	1.20	1.50
7	Moment	First	Column	W8X40	15	W12X50	50	0.0	0.0	1.20	1.50
8	Moment	Roof	Column	W8X40	15	W12X50	50	0.0	0.0	1.20	1.50
21	Moment	First	Column	W8X40	15	W10X68	68	0.0	0.0	1.17	2.04
22	Moment	Roof	Column	W8X40	15	W10X68	68	0.0	0.0	1.17	2.04
23	Moment	First	Column	W8X40	15	W10X68	68	0.0	0.0	1.17	2.04
24	Moment	Roof	Column	W8X40	15	W10X68	68	5.0	0.0	1.17	2.04
25	Moment	Roof	Beam	W12X16	24	**W10X68**	68	8.0	0.0	1.44	1.44
26	Moment	Roof	Interior Beam	W18X35	24	W10X39	39	0.0	0.0	1.92	1.87
27	Moment	Roof	Beam	W12X16	24	W14X30	30	0.0	0.0	1.44	1.44
28	Moment	First	Beam	W14X22	24	**W10X39**	39	0.0	0.0	1.92	1.92
29	Moment	First	Interior Beam	W21X44	24	W12X53	53	0.0	0.0	1.92	2.54
30	Moment	First	Beam	W14X22	24	W8X40	40	0.0	0.0	1.92	1.92

7. CONNECTIONS

J

Member Ref.	Frame	Floor	Member	Pre-Section	Length	New-Section	Unit Weight	Spacing or a	Beams/Bay	Qty of Members	Amount of Steel
(#)	*(type)*	*Units*		*(Shape)*	*(ft)*	*(Shape)*	*(plf)*	*(ft)*	*(Units)*	*(Units)*	*(kips)*
0	0	0	0	0	0	**#N/A**	#N/A	#N/A	#N/A	#N/A	#N/A
0	0	0	0	0	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	0	0	0	0	0	**W12X50**	50	#N/A	#N/A	#N/A	#N/A

COLUMN-CAPACITY

Member Ref:	23
Frame:	Moment
Floor:	First
Member:	Column
Ref. 2:	1-M

ASSUMPTIONS:

No transverse loads are applied to the member (Per section 7)

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	$E =$	29000	ksi
	$G =$	11200	ksi
Yield Strength:	$F_{y} =$	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	$L =$	15	ft	15 *Project Information*

Column Slenderness Parameters:

Unbraced Length, x:	$L_{bx} =$	15	ft	Global or Local System?
Unbraced Length, y:	$L_{by} =$	15	ft	
Unbraced Length, z:	$L_{bz} =$	15	ft	
Eff. Length Factor, x:	$K_{x} =$	1		
Eff. Length Factor, y:	$K_{y} =$	1		
Eff. Length Factor, z:	$K_{z} =$	1		

Reference: Excel

Section | *Eq/Fig/Table/Notes*
Information

3. SECTION PROPERTIES

Section:	W	**W8X40**						
Member is in:		Compression						
Moment of Inertia, x:	$I_{xw} =$	146	in^4	Depth:	$d =$	8.25	in
Moment of Inertia, y	$I_{yw} =$	49.1	in^4	Width:	$b_f =$	8.07	in
Polar Moment of Inertia:	$J_w =$	1.12	in^4	Flange Thickness:	$t_f =$	0.56	in
Radius of Gyration, x:	$r_{xw} =$	3.53	in	Web Thickness:	$t_w =$	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	$A =$	11.7	in^2
Section Modulus:	$S_x =$	35.5	in^3	r_{ts}	$r_{ts} =$	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	$h_0 =$	11.60	in
T	T =	0	in	Warping Constant	$C_w =$	726	in

3. PRELIMINARY ANALYSIS

Eq. E 6-2a/b

Slenderness Ratios:	$(KL/r)_x =$	51.0			
	$(KL/r)_y =$	88.2	AISC	Table	3-2
	$(KL)_z =$	180.0	AISC	Table	3-2
Largest Possible Ratio:		88.2			
Compressive Control:		113.43	E		
Critical Stress, Fcr equation:		**USE E3-2**			

4. LOCAL SLENDERNESS CHECK:

Table B4.1a

	Web	**Flange**
Member	**h/tw**	**bf/2t**
	17.6	7.21
Critical	λ_r	λ_r
Case	[case 5]	[case 1]
	35.9	*35.9*
Check	Nonslender	Nonslender

Reference: AISC 14th
Section | *Eq/Fig/Table/Notes*

5. BUCKLING ANALYSIS:

E

Euler Buckling Stress:	$F_{e3} =$	36.8	ksi	*Eq. E3-4*
Torsional Buckling Stress:	$F_{e4} =$	97.2	ksi	*Eq. E4-4*
Controling Euler Stress:	Fe3 =	36.8	ksi	
Critical Buckling Stress:	$F_{cr} =$	28.3	ksi	*Eq. E3-2*

6. COLUMN CAPACITY:

Eq. E3-1

Compressive Strength:	P_{n} =	331.1	ksi	*Eq. E3-1*
Factor:	Φ =	0.9		
Column Capacity:	$\Phi.P_{n}$ =	298.0	ksi	

BEAM-COLUMN ANALYSIS

1. MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
	G =	11200	ksi
Yield Strength:	F_{y} =	50	ksi

2. MEMBER GEOMETRIC INFORMATION:

Beam Length	L =	15	ft	*Project Information*
Column Slenderness Parameters:				
Unbraced Length, x:	L_{bx} =	15	ft	*Global or Local System?*
Unbraced Length, y:	L_{by} =	15	ft	
Unbraced Length, z:	L_{bz} =	15	ft	
Eff. Length Factor Check:		**K>1**		*Check for values below*
Eff. Length Factor, x:	K_{x} =	1		
Eff. Length Factor, y:	K_{y} =	1		
Eff. Length Factor, z:	K_{z} =	1		
Plastic Zones Lengths and Info:				
Full plastic yield Length:	L_{p} =	7.2	ft	
LTB Length:	L_{r} =	29.9	ft	
	$\phi_b BF$ =	2.46	kips	
	$\phi_b M_{px}$ =	149	kip.ft	

Reference: Section Information | Excel *Eq/Fig/Table/Notes*

3. SECTION PROPERTIES

Section:	W	**W8X40**					
Member is in:		Compression					
Moment of Inertia, x:	I_{xw} =	146	in^4	Depth:	d =	8.25	in
Moment of Inertia, y	I_{yw} =	49.1	in^4	Width:	b_f =	8.07	in
Polar Moment of Inertia:	J_w =	1.12	in^4	Flange Thickness:	t_f =	0.56	in
Radius of Gyration, x:	r_{xw} =	3.53	in	Web Thickness:	t_w =	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in^2
Section Modulus:	S_x =	35.5	in^3	r_{ts}	r_{ts} =	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in^3	Distance flange/centro	h_0 =	11.60	in
Plastic Section Modulus, y:	Z_y =	18.5	in	Warping Constant	C_w =	726	in
T	T =	0	in	Section Modulus:	S_y =	12.2	in^3

3. SLENDERNESS CHARACTERISTICS: Table B4.1a

	Web	**Flange**
Flexure	Compact	Compact
Compression	Mntx =	32

4. CONSIDERATION OF IMPERFECTIONS - NOTIONAL LOADS: C2.2(b)

Notional Load:	Z_i =	0.428	kip	Eq.	C2-1
Second/First order drift ratio:		2	in		

Is it applied at all levels in all combinations? **YES** Ref. to C.2.3(3)

Reference: GTS
Section *Eq/Fig/Table/Notes*

5. FIRST ORDER ANALYSIS FORCES: GTS

Ultimate Axial Load, NT	P_{nt} =	57.2	kips	
Ultimate Moment, NT, x	M_{ntx} =	32	kip.ft	
Ultimate Moment, NT, y	M_{nty} =	0	kip.ft	
Ultimate Axial Load, LT	P_{lt} =	29.4	kips	
Ultimate Moment, LT, y	Ml_{tx} =	159	kip.ft	
Ultimate Moment, LT, y	Ml_{tx} =	0	kip.ft	
Total V. load in story	P_{story} =	3643.2	kip	
	P_{mf} =	607.6	kip	
Story Shear in Direction of	H =	31	kip	
	α =	1		*LRFD*
Lateral Deflection	ΔH =	0.215	in	
Fact. Story Drift Limit	$\Delta H/L$ =	0.0012		

6. MEMBER CAPACITY: Eq. E3-1

Axial Capacity	$\phi.P_n$ =	298.0	ksi		
Flexure Capacity					
Along axis x:	Zone =	2			
	Cb =	1.34			
Flexure Capacity, x	M_{cx} =	149.0	kip.ft		
Along axis y:	Fy.Zy =	925		Eq.	F6-1
	1.6Fy.Sy =	976		Eq.	F6-1
Flexure Capacity, y	M_{cy} =	832.5	kip.ft		

Reference: AISC 14th
Section *Eq/Fig/Table/Notes*

7. APPROXIMATE SECOND ORDER ANALYSIS: C

Along axis x: DAM: Use reduced stiffness per C2.3

	τ_b =	1.00		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, x:	C_{mx} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength, x	P_{ex} =	1612	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1x} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Along axis y:

	τ_b =	1		*Apply to all*	C	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M_1 =	-1				
Larger 1st-O End Mom:	M_2 =	1				
Modif. Coefficient, y	C_{my} =	1		App. 8	Eq.	A-8-4
Elastic Buckling Strength	P_{ey} =	542	kip	App. 8	Eq.	A-8-5
Amplification Factor	B_{1y} =	1.0		App. 8	Eq.	A-8-3
Factor Check:		OK		*Check*		

Calculate P-Δ Amplification Factor:

Along axis x:

	R_m =	0.97		A-8-8
	$P_{e\text{-}story}$ =	25304.2	kip	A-8-7
	B_{2x} =	1.17		A-8-6
2nd-Order Axial Strength	P_r =	91.5	kip	A-8-2
2nd-Order Mom. Strength	M_{rx} =	0.0	kip.ft	A-8-1

Along axis y:

	R_{my} =	0.97		A-8-8
	$P_{e\text{-}storyY}$ =	25304.2	kip	A-8-7
	B_{2y} =	1.00		A-8-6
2nd-Order Axial Strength	P_{ry} =	86.6	kip	A-8-2
2nd-Order Mom. Strength	M_{ry} =	-32.0	kip.ft	A-8-1

8. COMBINED FORCES INTERACTION EQUATION: GTS

Check Pr/Pc	P_r/P_c =	0.307			
Pr/Pc ≥ 0,2		1.294	OK	Eq.	H.1-1a
Pr/Pc < 0,2		0.000	OK	Eq.	H.1-1b
Design Check			OK	Eq.	H.1-1a