COLUMN-CAPACITY

Member Ref:23Frame:MomentFloor:FirstMember:ColumnRef. 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: 15 ft Global or Local System? L_{bx} = Unbraced Length, y: L_{by =} 15 ft Unbraced Length, z: 15 ft $L_{bz} =$ Eff. Length Factor, x: K_x = 1 Eff. Length Factor, y: $K_{y} =$ 1 Eff. Length Factor, z: 1 $K_{z} =$

Reference: Excel

				Section		Eq/Fig/Tal	ble/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 _{vw} =	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
Т	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	7	Гable	3-2
	(KL) _{z =}	180.0		AISC	7	Гable	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: Section	AISC 14th <i>Eq/Fig/Table/Notes</i>
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	

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ksi

Member-Capacity and Beam-Column Analysis

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Compressive Strength:

_{1 =} 331.1 ksi

Eq. E3-1

Factor: Column Capacity: $\Phi_{=}$ 0.9 $\Phi.P_{n=}$ 298.0

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: $\begin{array}{ccc} L_{r} = & 29.9 & \text{ft} \\ & \phi_b BF = & 2.46 & \text{kips} \end{array}$

 $\phi_b M_{px} = 149$ kip.ft

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	ir
Polar Moment of Inertia:	J _{w=}	1.12	in⁴	Flange Thickness:	t _{f=}	0.56	ir
Radius of Gyration, x:	r _{xw =}	3.53	in	Web Thickness:	t _{w =}	0.36	ir
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	ir
Section Modulus:	S _{x =}	35.5	in ³	r_{ts}	r _{ts =}	2.81	ir
Plastic Section Modulus, x:	Z =	39.8	in ³	Distance flange/centro	h _{0 =}	11.60	ir
Plastic Section Modulus, y:	$Z_{y} =$	18.5	in	Warping Constant	C _{w =}	726	ir
Т	T =	0	in	Section Modulus:	$S_{y=}$	12.2	ir

Flexure

Along axis x:

Member-Capacity and Beam-Column Analysis

3. SLENDERNESS CHARACTERISTICS:

Web

Compact

Flange

Compact

Table B4.1a

Compression	Mntx =	6				
4. CONSIDERATION OF IMP	ERFECTIONS - I	NOTIONAL L	.OADS:	C2.2(b)		
Notional Load:	Z _{i =}	0.428	kip		Eq.	C2-1
Second/First order drift ratio	o:	2	in			
Is it applied at all levels in al	l combinations	?	YES	Ref. to C.2.3(3)		
				Reference: Section	GTS Eq/Fia/	Table/Notes
5. FIRST ORDER ANALYSIS F	ORCES:			GTS	, 3,	,
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	MI _{tx} =	71	kip.ft			
Ultimate Moment, LT, y		0	kip.ft			
Offilinate Moment, LT, y	MI _{tx} =	U	KIP.IT			
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	∆H/L =	0.0012				
6. MEMBER CAPACITY:				Eq. E3-1		
Axial Capacity	φ.P _{n =}	298.0	ksi			
Flexure Capacity	_	_				
Along axis x:	Zone = Cb =	2				
Flexure Capacity, x	$M_{cx} =$	1.34 149.0	kip.ft			
Along axis y:	Fy.Zy =	925	κιμ.ιι		Eq.	F6-1
AIDIIS ANIS Y.	1.6Fy.Sy =	925 976			Eq.	F6-1 F6-1
Flexure Capacity, y	M _{cy =}	832.5	kip.ft		-y·	
· p · · · · · · · · · · · · · · · ·	cy –		i	Reference:	AISC 14t	:h
				Section		Table/Notes
7. APPROXIMATE SECOND	ORDER ANALYS	SIS:		С	<u>,, 3, </u>	

DAM:

Use reduced stiffness per C2.3

14.551 Advanced Steel Design Homework 5	١	Member-Cap	Problem # 5 pacity and Beam	. 2 n-Column Analysis		Ana Gouveia 12/8/2014
	τ_{b} =	1.00		Apply to all	С	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:		ОК		Check		
Along axis y:						
	τ_b =	1		Apply to all	С	2.3(2)
Type of Curvature:	2.4	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-∆ Amplification Fa	actor:					
Along axis x:						
	$R_{m} =$	0.97				A-8-8
	$P_{e-story} =$	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
	$P_{e-storyY} =$	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 INTERA	CTION EQU	ATION:		GTS		
Check Pr/Pc	$P_r/P_{c} =$	0.105				
Pr/Pc ≥ 0,2	i, C -	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:26Frame:MomentFloor:RoofMember:Interior BeamRef. 2:2-M

ASSUMPTIONS:

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

1. MATERIAL PROPERTIES:

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

y- ----

2. MEMBER GEOMETRIC INFORMATION:

Beam Length L = 24 ft 24 Project Information

Column Slenderness Parameters:

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

Reference: Excel

				Section		Eq/Fig/Tal	ble/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
Т	T =	0	in	Warping Constant	C _{w =}	1140	in
3. PRELIMINARY ANALYSIS				Eq. E 6-2a/b			
	(m. 1.)						
Slenderness Ratios:	$(KL/r)_{x=}$						
	$(KL/r)_{y=}$	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	$\lambda_{\rm r}$	λ_{r}
Case	[case 5]	[case 1]
	35.9	35.9
Check	N.G	Nonslender

				Reference: Section	AISC 14th 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	

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Factor:

Problem # 5.2

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Eq. E3-1

Member-Capacity and Beam-Column Analysis

Compressive Strength: $P_{n=}$

 $P_{n=}$ 8.8 ksi $\Phi_{=}$ 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

 ϕ_b BF = 12.3 kips ϕ_b M_{px =} 249 kip.ft

Reference: Excel

Section *Eq/Fig/Table/Notes*

3. SECTION PROPERTIES Information

Section:	W	W18X35			
Member is in:		Compression			
			1		
Moment of Inertia, x:	I _{xw =}	510	in ⁴	Depth:	
Moment of Inertia, y	I _{yw} =	15.3	in ⁴	Width:	
Polar Moment of Inertia:	$J_{w} =$	0.506	in ⁴	Flange Thick	kness:
Radius of Gyration, x:	r _{xw =}	7.04	in	Web Thickne	ess:
Radius of Gyration, y	ryw =	1.22	in	Area:	
Section Modulus:	S _{x =}	57.6	in ³	r_{ts}	
Plastic Section Modulus, x:	Z =	66.5	in ³	Distance flange	/centro
Plastic Section Modulus, y:	$Z_{y} =$	8.06	in	Warping Consta	nt
Т	T =	0	in	Section Modulus:	

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3. SLENDERNESS CHARACTERISTICS:

Table B4.1a

	Web	Flange
Flexure	Compact	Compact
Compression	Mntx =	3.5

4. CONSIDERATION OF IMP	ERFECTIONS - I	NOTIONAL I	OADS:	C2.2(b)		
Notional Load:	Z _{i =}	0.94	kip		Eq.	C2-1
Second/First order drift rati	o:	2	in			
Is it applied at all levels in al	I combinations	?	YES	Ref. to C.2.3(3)		
				Reference:	GTS	
5. FIRST ORDER ANALYSIS F	ORCES:			Section GTS	Eq/Fig/	Table/Notes
5. TIKST ORDER ARAETSIST	ORCLS.			<u> </u>		
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 _{It} =	1	kips			
Ultimate Moment, LT, y	MI _{tx} =	10.4	kip.ft			
Ultimate Moment, LT, y	MI _{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			
Lateral Ballanda	α =	1		LRFD		
Lateral Deflection Fact. Story Drift Limit	ΔH = ΔH/L =	0.215 0.0007	in			
race. Story Diffe Limit	ΔII/ L -	0.0007				
6. MEMBER CAPACITY:				Eq. E3-1		
Axial Capacity	φ.P _n =	7.9	ksi			
Flexure Capacity						
Along axis x:	Zone =	3				
Florence Compositor v	Cb =	1.34	Lin Ex			
Flexure Capacity, x	M _{cx} =	249.0 925	kip.ft		Ea	F6-1
Along axis y:	Fy.Zy = 1.6Fy.Sy =	925 976			Eq. Eq.	F6-1 F6-1
Flexure Capacity, y	M _{cy =}	832.5	kip.ft		-4.	
1 - 11 1	cy -	-	•	Reference:	AISC 14t	h
				Section		Table/Notes
7. APPROXIMATE SECOND	ORDER ANALYS	SIS:		С		

Along axis x:

DAM:

Use reduced stiffness per C2.3

14.551 Advanced Steel Design Homework 5	١	Member-Cap	Problem # 5.2 pacity and Beam-			Ana Gouveia 12/8/2014
	τ_{b} =	1.00		Apply to all	С	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:		ОК		Check		
Along axis y:						
	τ_b =	1		Apply to all	С	2.3(2)
Type of Curvature:	2.4	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-∆ Amplification Fa	actor:					
Along axis x:						
	$R_{m} =$	0.85				A-8-8
	$P_{e-story} =$	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
	$P_{e-storyY} =$	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 INTERA		GTS				
Check Pr/Pc	$P_r/P_{c} =$	0.802				
Pr/Pc ≥ 0,2	i, C -	1.294	ОК		Eq.	H.1-1a
Pr/Pc < 0,2		0.000	ОК		Eq.	H.1-1b
Design Check			OK		Eq.	H.1-1a

COLUMN-CAPACITY

Member Ref:29Frame:MomentFloor:FirstMember:Interior BeamRef. 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 Information 3. SECTION PROPERTIES W W21X44 Section: Compression Member is in: in^4 843 d = 20.7 Moment of Inertia, x: Depth: in in^4 20.7 Width: $b_{f\,=}$ 6.5 Moment of Inertia, y I_{yw} = in in^4 Polar Moment of Inertia: 0.77 J_{w=} Flange Thickness: $t_{f=}$ 0.45 in Radius of Gyration, x: 8.06 Web Thickness: 0.35 $r_{xw} =$ in in in² Radius of Gyration, y 1.26 in Area: A = 13 ryw = in^3 Section Modulus: $S_{x} =$ 81.6 \mathbf{r}_{ts} $r_{ts} =$ 2.81 in in^3 Plastic Section Modulus, x: Z = 95.4 Distance flange/centro $h_{0} =$ 11.60 in Т 0 T = Warping Constant $C_{w} =$ 2110 in 3. PRELIMINARY ANALYSIS Eq. E 6-2a/b $(KL/r)_{x=}$ 35.7 **Slenderness Ratios:** $(KL/r)_{v=}$ 228.6 AISC Table 3-2 $(KL)_{z}$ 288.0 **AISC** Table 3-2 Largest Possible Ratio: 228.6 **Compressive Control:** 113.43 Ε

USE E3-3

4. LOCAL SLENDERNESS CHECK:

Critical Stress, Fcr equation:

Table B4.1a

	Web	Flange
Member	h/tw	bf/2t
	53.6	7.22
Critical	λ_{r}	$\lambda_{\rm r}$
Case	[case 5]	[case 1]
	35.9	35.9
Check	N.G	Nonslender

				Reference: Section	AISC 14th <i>Eq/Fig/Table/Notes</i>
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	

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Problem # 5.2

Member-Capacity and Beam-Column Analysis

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12/8/2014

Compressive Strength: Eq. E3-1 14.3 ksi

Φ = 0.9 Factor:

 $\Phi.P_{\text{n=}}$ Column Capacity: 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: $L_{hy} = 24$ ft

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

 $\phi_b BF = 16.8 \qquad \text{kips}$ $\phi_b M_{px} = 358 \qquad \text{kip.ft}$

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	\mathbf{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
Т	T =	0	in	Section Modulus:	S _{y =}	6.37	in ³

Flexure

Along axis x:

Member-Capacity and Beam-Column Analysis

3. SLENDERNESS CHARACTERISTICS:

Web

Compact

Flange

Compact

Table B4.1a

7. APPROXIMATE SECOND	ORDER ANALYS	SIS:		С		
				Section		Table/Notes
Flexure Capacity, y	M_{cy} =	832.5	kip.ft	Reference:	AISC 14t	h
Floyuro Canacitu u	1.6Fy.Sy =	976	kin ft		Eq.	F6-1
Along axis y:	Fy.Zy =	925			Eq.	F6-1
Flexure Capacity, x	M _{cx} =	358.0	kip.ft		_	
	Cb =	1.34				
Along axis x:	Zone =	3				
Flexure Capacity						
Axial Capacity	φ.P _n =	12.8	ksi			
6. MEMBER CAPACITY:				Eq. E3-1		
Fact. Story Drift Limit	Δ H/L =	0.0007				
Lateral Deflection	ΔH =	0.215	in			
City Official in Direction of	α =	1	МР	LRFD		
Story Shear in Direction of	rmf = H =	31	kip			
otal V. load in story	P _{story} = P _{mf} =	3643.2 607.6	kip kip			
Jltimate Moment, LT, y	MI _{tx} =	0	kip.ft			
Jltimate Moment, LT, y	MI _{tx} =	15	kip.ft			
Jitimate Axial Load, LT	P _{lt} =	2	kips			
Jitimate Moment, NT, y	M _{nty} =	0	kip.ft			
Jitimate Moment, NT, x	M _{ntx} =	20	kip.ft			
Jitimate Axial Load, NT	P _{nt} =	6.6	kips			
5. FIRST ORDER ANALYSIS I	FORCES:			GTS	-4/3/	
				Reference: Section	GTS Fa/Fia/	Table/Notes
s it applied at all levels in a	ll combinations	?	YES	Ref. to C.2.3(3)		
Second/First order drift rati	0:	2	in			
Notional Load:	Z _i =	0.428	kip		Eq.	C2-1
. CONSIDERATION OF IMP	PERFECTIONS - I	NOTIONAL L	OADS:	C2.2(b)		
Compression	Mntx =	20				

DAM:

Use reduced stiffness per C2.3

14.551 Advanced Steel Design Homework 5		Member-Cap	Problem # 5.2 pacity and Beam-	2 -Column Analysis		Ana Gouveia 12/8/2014
	τ_{b} =	1.00		Apply to all	С	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:		ОК		Check		
Along axis y:						
	τ_b =	1		Apply to all	С	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-∆ Amplification Fa	actor:					
Along axis x:						
	$R_{m} =$	0.97				A-8-8
	$P_{e-story} =$	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
	$P_{e-storyY} =$	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 INTERA		GTS				
Check Pr/Pc	$P_r/P_{c} =$	0.686				
Pr/Pc ≥ 0,2	1, C -	1.294	ОК		Eq.	H.1-1a
Pr/Pc < 0,2		0.000	ОК		Eq.	H.1-1b
Design Check			OK		Eq.	H.1-1a