**COLUMN-CAPACITY** 

Member Ref:1Frame:BracedFloor:FirstMember:ColumnRef. 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: 15 ft Global or Local System? L<sub>bx</sub> = Unbraced Length, y: L<sub>by =</sub> 15 ft Unbraced Length, z: 15 ft  $L_{bz} =$ Eff. Length Factor, x: 1 K<sub>x</sub> = 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:	VV	Compression					
Wichiber is iii.		Compression					
Moment of Inertia, x:	I <sub>xw =</sub>	146	in <sup>4</sup>	Depth:	d =	8.25	in
Moment of Inertia, y	I <sub>yw =</sub>	49.1	in <sup>4</sup>	Width:	b <sub>f=</sub>	8.07	in
Polar Moment of Inertia:	J <sub>w=</sub>	1.12	in <sup>4</sup>	Flange Thickness:	t <sub>f=</sub>	0.56	in
Radius of Gyration, x:	r <sub>xw =</sub>	3.53	in	Web Thickness:	t <sub>w =</sub>	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in <sup>2</sup>
Section Modulus:	S <sub>x =</sub>	35.5	in <sup>3</sup>	$r_{ts}$	r <sub>ts =</sub>	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in <sup>3</sup>	Distance flange/centro	h <sub>0 =</sub>	11.60	in
Т	T =	0	in	Warping Constant	C <sub>w =</sub>	726	in
2. DDELIBAINIA DV ANALYCIC				Fo. F.C. 20/h			
3. PRELIMINARY ANALYSIS				Eq. E 6-2a/b			
Slenderness Ratios:	(KL/r) <sub>x =</sub>	51.0					
	(KL/r) <sub>y =</sub>	88.2		AISC	٦	Гable	3-2
	(KL) <sub>z =</sub>	180.0		AISC	٦	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	$\lambda_{r}$	$\lambda_{r}$
Case	[case 5]	[case 1]
	35.9	35.9
Check	Nonslender	Nonslender
•		

				Reference: Section	AISC 14th Eq/Fig/Table/Notes
5. BUCKLING ANALYSIS:				E	
Euler Buckling Stress:	F <sub>e3 =</sub>	36.8	ksi		Eq. E3-4
Torsional Buckling Stress:	F <sub>e4 =</sub>	97.2	ksi		Eq. E4-4
Controling Euler Stress:	Fe3 =	36.8	ksi		
Critical Buckling Stress:	F <sub>cr =</sub>	28.3	ksi		Eq. E3-2
6. COLUMN CAPACITY:				Eq. E3-1	

14.551 Advanced Steel Design Homework 4 **Problem # 4.3 M**ember-Capacity and Beam-Column Analysis

Ana Gouveia 12/8/2014

Compressive Strength:

Column Capacity:

Factor:

P<sub>n =</sub> 331.1 ksi

 $\Phi_{=}$  0.9  $\Phi.P_{n=}$  298.0 ksi

Eq. E3-1

#### **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: Excel

Section *Eq/Fig/Table/Notes* 

3. SECTION PROPERTIES Information

Section:	W	W8X40					
Member is in:		Compression					
Moment of Inertia, x:	I <sub>xw =</sub>	146	in <sup>4</sup>	Depth:	d =	8.25	ir
Moment of Inertia, y	I <sub>yw =</sub>	49.1	in <sup>4</sup>	Width:	b <sub>f=</sub>	8.07	ii
Polar Moment of Inertia:	J <sub>w=</sub>	1.12	in <sup>4</sup>	Flange Thickness:	t <sub>f=</sub>	0.56	ii
Radius of Gyration, x:	r <sub>xw =</sub>	3.53	in	Web Thickness:	t <sub>w =</sub>	0.36	ii
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	ir
Section Modulus:	S <sub>x =</sub>	35.5	in <sup>3</sup>	r <sub>ts</sub>	r <sub>ts =</sub>	2.81	ir
Plastic Section Modulus, x:	Z =	39.8	in <sup>3</sup>	Distance flange/centro	h <sub>0 =</sub>	11.60	ir
Plastic Section Modulus, y:	$Z_{y} =$	18.5	in	Warping Constant	C <sub>w =</sub>	726	ir
Т	T =	0	in	Section Modulus:	S <sub>y =</sub>	12.2	ir

Along axis x:

# 3. SLENDERNESS CHARACTERISTICS:

Table B4.1a

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

4. CONSIDERATION OF IMP	ERFECTIONS -	NOTIONAL I	LOADS:	C2.2(b)		
Notional Load:	<b>Z</b> <sub>i =</sub>	0.428	kip		Eq.	C2-1
Second/First order drift ratio		2	in		-4.	02 1
		_				
Is it applied at all levels in al	I combinations	?	YES	Ref. to C.2.3(3)		
				Reference:	GTS	
				Section	Eq/Fig/	Table/Notes
5. FIRST ORDER ANALYSIS F	ORCES:			GTS		
Ultimate Avial Load NT	D	130	king			
Ultimate Axial Load, NT	P <sub>nt</sub> =		kips			
Ultimate Moment, NT, x	M <sub>ntx</sub> =	14	kip.ft			
Ultimate Moment, NT, y	$M_{nty} =$	0	kip.ft			
Ultimate Axial Load, LT	P <sub>lt</sub> =	0	kips			
Ultimate Moment, LT, y	MI <sub>tx</sub> =	0	kip.ft			
Ultimate Moment, LT, y	MI <sub>tx</sub> =	0	kip.ft			
Total V. load in story	P <sub>story</sub> =	3643.2	kip			
	P <sub>mf</sub> =	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	φ.P <sub>n</sub> =	298.0	ksi	_		
ar capacity	λ 11 =	250.0	101			
Flexure Capacity						
Along axis x:	Zone =	2				
	Cb =	1.34				
Flexure Capacity, x	M <sub>cx</sub> =	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 14t	
				Section	Eq/Fig/	Table/Notes
7. APPROXIMATE SECOND	ORDER ANALY	SIS:		С		

DAM:

Use reduced stiffness per C2.3

14.551 Advanced Steel Design Homework 4		<b>M</b> ember-Cap	Problem # 4. pacity and Beam	. <b>3</b> n-Column Analysis		Ana Gouveia 12/8/2014
	$\tau_{b}$ =	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:		OK		Check		
Along axis y:						
	$\tau_{b}$ =	1		Apply to all	С	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M <sub>1 =</sub>	-1				
Larger 1st-O End Mom:	M <sub>2</sub> =	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	$P_{r} =$	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
	$P_{e-storyY} =$	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 INTERA	CTION EQU	ATION:		GTS		
Check Pr/Pc	$P_r/P_{c}$	0.436				
Pr/Pc ≥ 0,2	,, 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:3Frame:BracedFloor:FirstMember:ColumnRef. 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: 15 ft Global or Local System? L<sub>bx</sub> = Unbraced Length, y: L<sub>by =</sub> 15 ft Unbraced Length, z: 15 ft  $L_{bz} =$ Eff. Length Factor, x: 1 K<sub>x</sub> = Eff. Length Factor, y:  $K_{y} =$ 1 Eff. Length Factor, z: 1  $K_{z} =$ 

Reference: Excel

				Section		Eq/Fig/Tai	ble/Notes
3. SECTION PROPERTIES				Information			
<u>Section:</u>	W	W8X40					
Member is in:		Compression					
			. 4				
Moment of Inertia, x:	I <sub>xw</sub> =	146	in <sup>4</sup>	Depth:	d =	8.25	in
Moment of Inertia, y	I <sub>yw =</sub>	49.1	in <sup>4</sup>	Width:	b <sub>f=</sub>	8.07	in
Polar Moment of Inertia:	J <sub>w =</sub>	1.12	in <sup>4</sup>	Flange Thickness:	t <sub>f=</sub>	0.56	in
Radius of Gyration, x:	r <sub>xw =</sub>	3.53	in	Web Thickness:	t <sub>w =</sub>	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in <sup>2</sup>
Section Modulus:	S <sub>x =</sub>	35.5	in <sup>3</sup>	$r_{ts}$	r <sub>ts =</sub>	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in <sup>3</sup>	Distance flange/centro	h <sub>0 =</sub>	11.60	in
Т	T =	0	in	Warping Constant	C <sub>w =</sub>	726	in
3. PRELIMINARY ANALYSIS				Eq. E 6-2a/b			
Slenderness Ratios:	$(KL/r)_{x=}$	51.0					
	$(KL/r)_{y=}$	88.2		AISC	7	Table	3-2
	(KL) <sub>z =</sub>	180.0		AISC	7	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	$\lambda_{r}$	$\lambda_{r}$
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 <sub>e3 =</sub>	36.8	ksi		Eq. E3-4
Torsional Buckling Stress:	F <sub>e4 =</sub>	97.2	ksi		Eq. E4-4
Controling Euler Stress:	Fe3 =	36.8	ksi		
Critical Buckling Stress:	F <sub>cr =</sub>	28.3	ksi		Eq. E3-2
6. COLUMN CAPACITY:				Eq. E3-1	

14.551 Advanced Steel Design Homework 4

Factor:

**Problem # 4.3 M**ember-Capacity and Beam-Column Analysis

Ana Gouveia 12/8/2014

Compressive Strength:  $P_{n} = 3$ 

 $P_{n=}$  331.1 ksi  $\Phi_{=}$  0.9

Column Capacity:  $\Phi.P_{n=}$  298.0 ksi

Eq. E3-1

### **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

 $\phi_b M_{px} = 149$  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 <sub>xw =</sub>	146	in <sup>4</sup>	Depth:	d =	8.25	in
Moment of Inertia, y	I <sub>yw =</sub>	49.1	in <sup>4</sup>	Width:	b <sub>f=</sub>	8.07	in
Polar Moment of Inertia:	J <sub>w =</sub>	1.12	in <sup>4</sup>	Flange Thickness:	$t_{f=}$	0.56	in
Radius of Gyration, x:	r <sub>xw =</sub>	3.53	in	Web Thickness:	t <sub>w =</sub>	0.36	in
Radius of Gyration, y	ryw =	2.04	in	Area:	A =	11.7	in <sup>2</sup>
Section Modulus:	S <sub>x =</sub>	35.5	in <sup>3</sup>	$r_ts$	r <sub>ts =</sub>	2.81	in
Plastic Section Modulus, x:	Z =	39.8	in <sup>3</sup>	Distance flange/centro	h <sub>0 =</sub>	11.60	in
Plastic Section Modulus, y:	Z <sub>y =</sub>	18.5	in	Warping Constant	C <sub>w =</sub>	726	in
Т	T =	0	in	Section Modulus:	S <sub>y=</sub>	12.2	in <sup>3</sup>

Flexure

Along axis x:

### Member-Capacity and Beam-Column Analysis

### 3. SLENDERNESS CHARACTERISTICS:

Web

Compact

Flange

Compact

Table B4.1a

riexure	Compact	Compact				
Compression	Mntx =	14				
4. CONSIDERATION OF IMP	PERFECTIONS -	NOTIONAL L	OADS:	C2.2(b)		
Notional Load:	Z <sub>i =</sub>	0.428	kip		Eq.	C2-1
Second/First order drift rati	io:	2	in			
Is it applied at all levels in a	II combinations	?	YES	Ref. to C.2.3(3)		
				Reference: Section	GTS Fa/Fia/	Table/Notes
5. FIRST ORDER ANALYSIS I	FORCES:			GTS	<u> </u>	rabicytvotes
Ultimate Axial Load, NT	P <sub>nt</sub> =	130	kips			
Ultimate Moment, NT, x	M <sub>ntx</sub> =	14	kip.ft			
Ultimate Moment, NT, y	M <sub>nty</sub> =	0	kip.ft			
Ultimate Axial Load, LT	P <sub>lt</sub> =	0	kips			
Ultimate Moment, LT, y	MI <sub>tx</sub> =	0	kip.ft			
Ultimate Moment, LT, y	MI <sub>tx</sub> =	0	kip.ft			
Total V. load in story	P <sub>story</sub> =	3643.2	kip			
	P <sub>mf</sub> =	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	φ.P <sub>n =</sub>	298.0	ksi			
Flexure Capacity	_	_				
Along axis x:	Zone =	2				
Floyura Canacity	Cb =	1.34 149.0	kin ft			
Flexure Capacity, x	M <sub>cx</sub> =	925	kip.ft		Ea	E <i>E</i> 1
Along axis y:	Fy.Zy = 1.6Fy.Sy =	925 976			Eq. Eq.	F6-1 F6-1
Flexure Capacity, y	M <sub>cy =</sub>	832.5	kip.ft		<b>-</b> 4٠	10-1
rickare capacity, y	ivicy =	002.0	мрис	Reference:	AISC 14t	h
				Section		Table/Notes
7. APPROXIMATE SECOND	ORDER ANALYS	SIS:		C	-4/119/	. 3.2.2/110103
		•				

DAM:

Use reduced stiffness per C2.3

14.551 Advanced Steel Design Homework 4		Problem # 4.3  Member-Capacity and Beam-Column Analysis				Ana Gouveia 12/8/2014
	$\tau_{b}$ =	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:		OK		Check		
Along axis y:						
	$\tau_{b}$ =	1		Apply to all	С	2.3(2)
Type of Curvature:		Single				
Smaller 1st-O End Mom:	M <sub>1 =</sub>	-1				
Larger 1st-O End Mom:	M <sub>2</sub> =	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	$P_{r} =$	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
	$P_{e-storyY} =$	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 INTERA		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