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:	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:	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} =	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:	$\Phi =$	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	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	$\phi.P_n =$	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