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

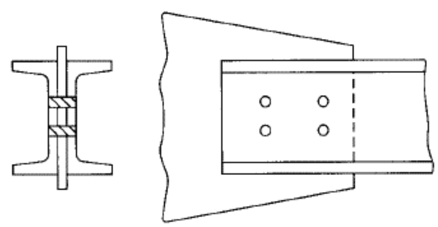


FIGURE P4-18

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 ʼFig/Table/No

LRFD				ASD		
1) Demand:				**Demand:**		
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	

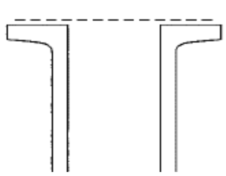
LRFD: 2 - C10X30	**ASD: 2 -** C10X30	**ANSWER**

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.



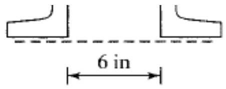


FIGURE P6-22

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	
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
	ϕF_{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	ϕF_{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)

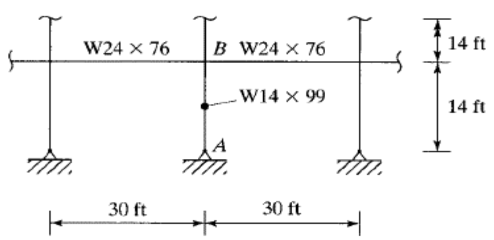


FIGURE P7-5

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:**			
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 $\phi_t Pn$=	922.00	ft	Pn/Ω=	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)

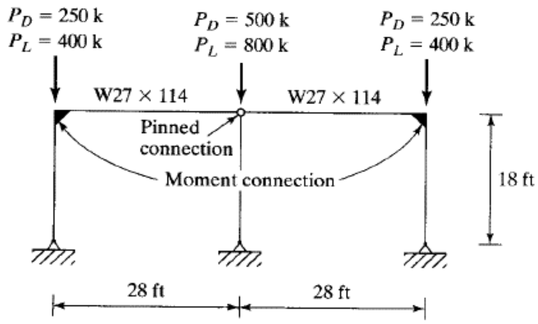


FIGURE P7-9

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
	B	Normal	1	2	0	3	4.98	1

App. 7 Fig. C-A-7.2
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 Loac	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**