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1.0 INTRODUCTION

Singly symmetric sections WT and double angle are widely used in steel structure as bracing members. STAAD Pro's code check ratio output of these singly symmetric members are useless due to the following limitations

1. Eccentric Moment Not Considered in STAAD for WT and Double Angle Code Check
   STAAD output code check ratio is based on pure compression/tension case, which is not correct as there is an eccentric moment applied due to brace connection. The correct code check shall consider the combination of axial tension /compression+ flexure, which actually yields a much lower capacity. From the following design charts we can see the brace actual tension/compression resistance under combined tension/compression + flexure can be only 35% of pure tension/compression case.
2. Complexities on Applying Eccentric Moment to Brace Member in STAAD for Code Check
   The eccentricity of brace force varies based on brace type (WT or double angle) and brace location (horizontal or vertical). The eccentric moment also varies in direction based on tension or compression force, which causes the tee stem in tension or compression and different flexural capacity. Considering the different load combinations causing different tension/compression forces in brace, it's impractical to apply eccentric moment manually in STAAD for code check.
3. Complexities on Defining Correct Brace Member Local Axis, In-Plane and Out-of-Plane Unsupported Length
   There are tremendous complexities to get a brace member correctly modeled/defined in STADD model for correct code checking. For example, the double angle X-bracing, engineers shall define correct in-plane and out-of- plane Ly, Lz value, check member's local axis to make sure it's correct in terms of ry, rz corresponding to previous defined Ly, Lz etc.
4. Code Checking on Class 4 Member Not Available in STAAD
   STAAD does not design Class 4 section. Class 4 section is valid for use in steel structure. The engineers have to work around this by defining lower yield strength in STAAD, section by section, to make it pass the code check. This is very time consuming process.

To simplify the code checking of WT and double angle bracing member, design charts and tables are created based on CSA S16-09 for quick member capacity lookup. These design charts and tables eliminate the complexities by sorting the charts and tables in different criteria. By looking up the charts and tables in the desired category, the engineers will get the correct brace design parameters (eccentricity and moment, in-plane and out-of-plane unsupported length, eccentric moment direction) automatically and find the brace tension/compression capacity in seconds.

- For X-bracing, correct in-plane and out-of-plane unsupported length is implemented in terms of brace location (horizontal or vertical) and brace type (WT or double angle)
- Correct eccentric moment is applied in terms of brace location (horizontal or vertical) and brace type (WT or double angle)
- The eccentric moment is applied at right direction in terms of brace location (horizontal or vertical) and brace type (WT or double angle). This will cause the WT or double angle's stem in the correct tension or compression case.
- Class 4 member capacity is available. Class 4 member is designed using the effective area per CSA S16-09 clause 13.3.5 (a)

2.0 DESIGN CHARTS

2.1 WT BRACE

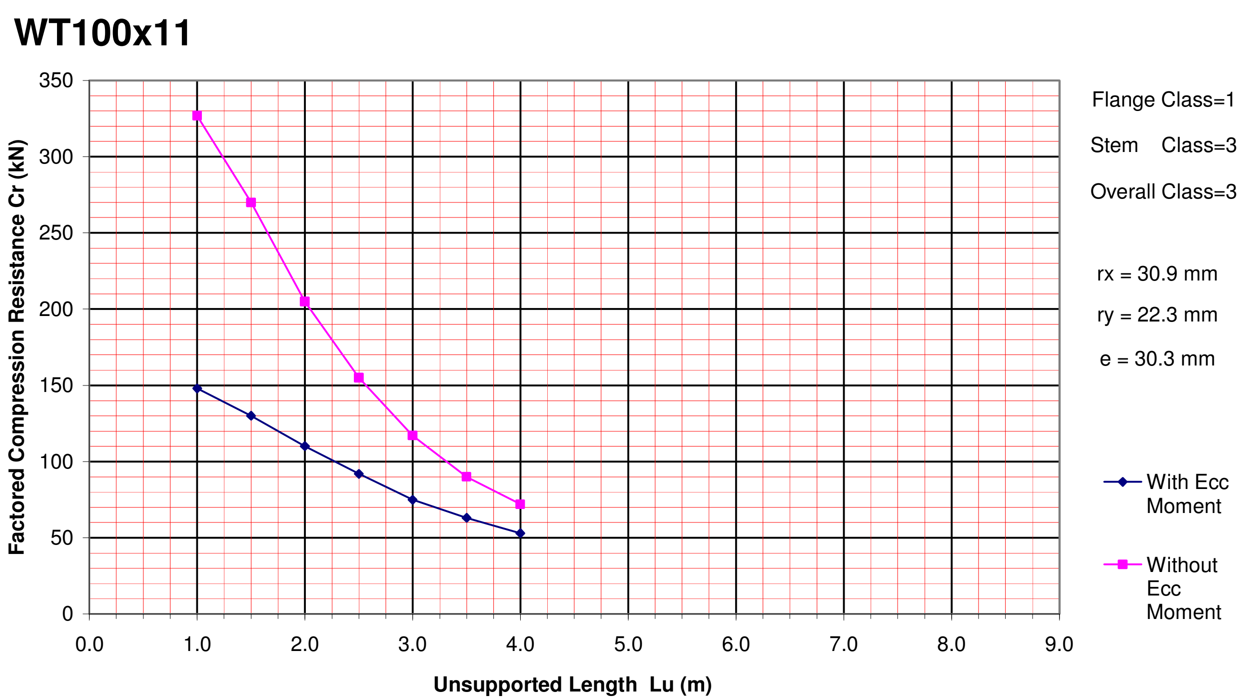
2.1.1 WT Brace Under Compression

2.1.1.1 WT Brace Compression – Non X-Brace Case

Design Basis & Assumption:

1. WT compression resistance capacity for two cases, with and without eccentric moment presence are presented.
2. The eccentric moment is calculated using gusset plate thickness =10mm. The eccentricity value is shown in each chart.
3. The capacity curve stops if next 0.5m increase of unsupported length causes KL/r ratio exceeding 200.
4. Assume WT unsupported length Lx = Ly =1.0L and Kx = Ky =1.0
5. WT brace design yield strength = 50 ksi =345 MPa
6. Section class is shown in each chart. For class 4 member, the effective area is used for calculation as per CSA S16-09 clause 13.3.5 (a)
7. WT flexural capacity is calculated based on AISC 360-05 section F9
8. Assume P-δ (small delta) is not performed in steel frame stability calculation. The U1 factor (CSA S16-09 clause 13.8.4) is calculated and applied for WT strength and stability check as per CSA S16-09 clause 13.8.3

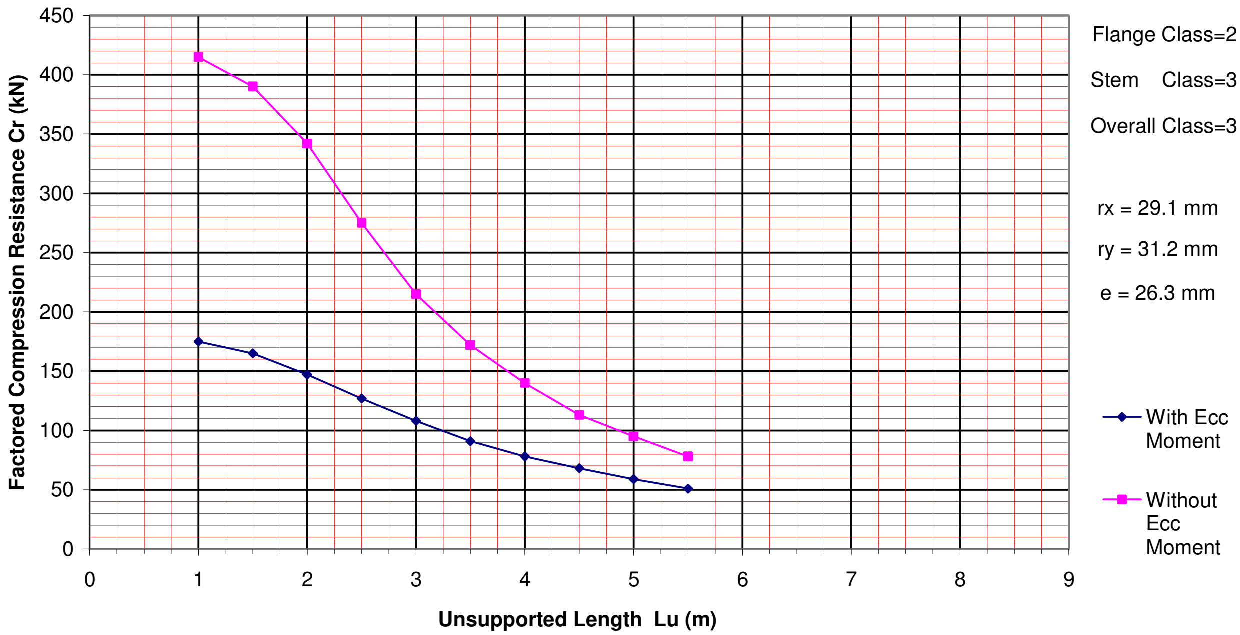
WT100x11



WT100x11

Lu	Cr - ECC	Cr - No ECC
m	kN	kN
1.0	148	327
1.5	130	270
2.0	110	205
2.5	92	155
3.0	75	117
3.5	63	90
4.0	53	72
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0
9.0	0	0

WT100x13.5



WT100x13.5

Lu	Cr - ECC	Cr - No ECC
m	kN	kN
1.0	175	415
1.5	165	390
2.0	147	342
2.5	127	275
3.0	108	215
3.5	91	172
4.0	78	140
4.5	68	113
5.0	59	95
5.5	51	78
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0
9.0	0	0

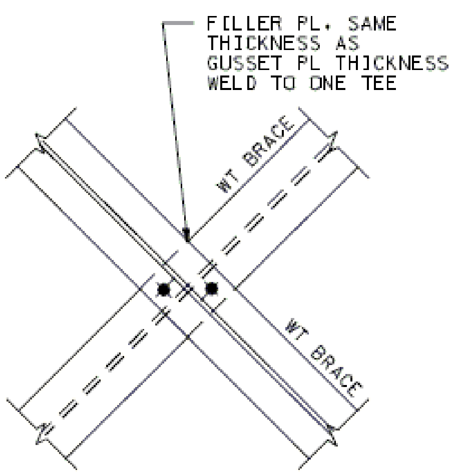
2.1.1.2 WT Brace Compression –X-Brace Case

Design Basis & Assumption

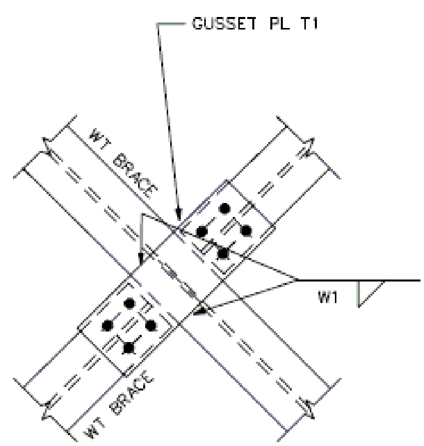
1. X-Bracing Out-of-Plane Unsupported Length
   In X-bracing design where two braces are connected at midpoint, some engineers may consider the brace in tension can be counted on to laterally brace the compression strut at midpoint against out-of-plane buckling. Reference is made to AISC Engineering Journal 4th Quarter 1997 "Practical Application of Energy Methods to Structural Stability Problems" by R. Shankar Nair. The out-of-plane stiffness of the intersection point of the braces must be calculated to determine if the tension member can be taken as a brace for the compression member.
   Since it is not practical to do such calculation for every X-bracing and the result may not be positive even the calculation is done, we assume in X-bracing design the out-of-plane unsupported length is full length and the in-plane unsupported length is half length.

WT Brace Intersect Point Connection

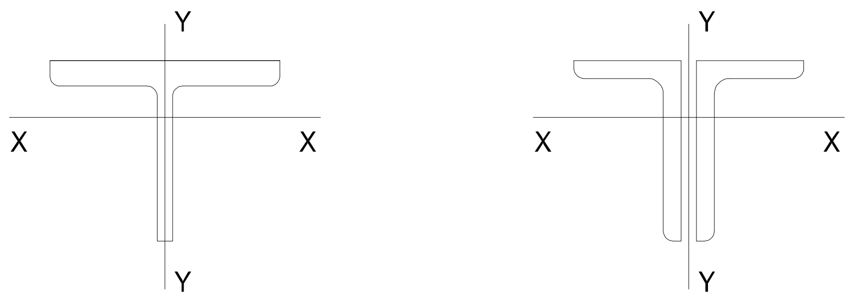
WT Vertical Brace



WT Horizontal Brace



WT and Double Angle Brace Local Axis



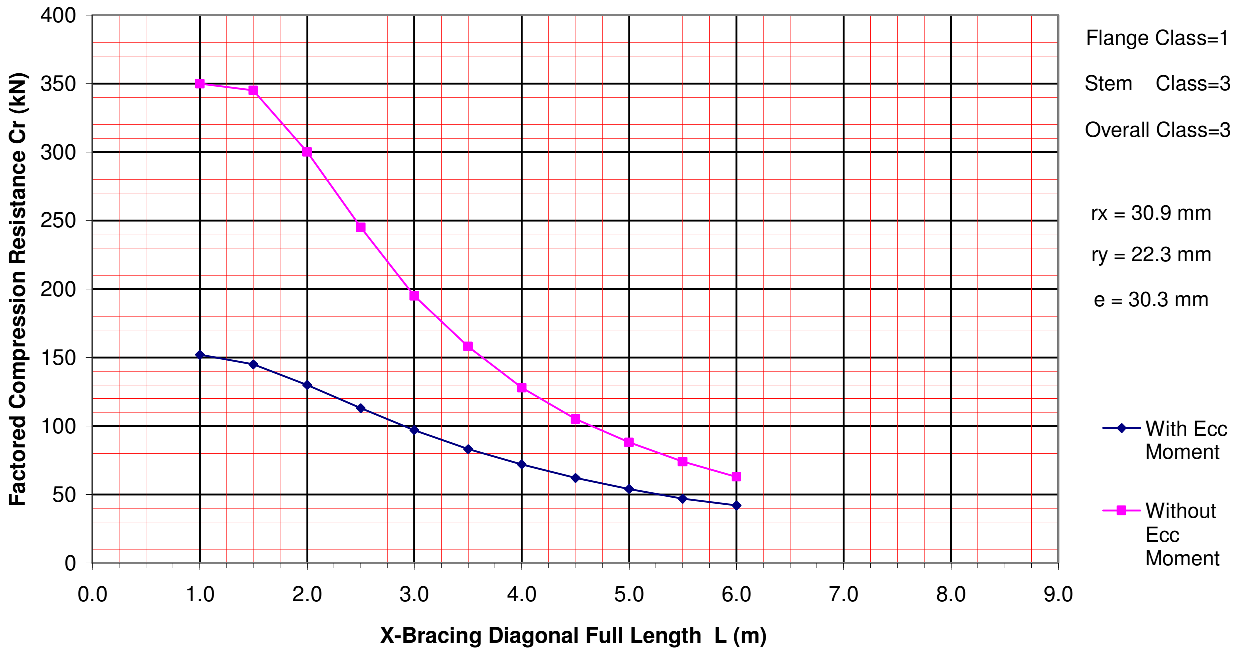
For both horizontal and vertical case, used Lx = 1.0L , Ly =0.5L for all X-bracing compression capacity calculation.

2. For all design charts in this section we assume the X-bracing member can take compression force. For Tension Only X-bracing member design please refer to tension capacity charts.
3. WT compression resistance capacity for two cases, with and without eccentric moment presence are presented.
4. The eccentric moment is calculated using gusset plate thickness =10mm. The eccentricity value is shown in each chart.
5. The capacity curve stops if next 0.5m increase of unsupported length causes KL/r ratio exceeding 200.
6. Assume WT unsupported length Lx = 1.0L , Ly =0.5L and Kx = Ky =1.0
7. WT brace design yield strength = 50 ksi =345 MPa
8. Section class is shown in each chart. For class 4 member, the effective area is used for calculation as per CSA S16-09 clause 13.3.5 (a)
9. WT flexural capacity is calculated based on AISC 360-05 section F9
10. Assume P-δ (small delta) is not performed in steel frame stability calculation. The U1 factor (CSA S16-09 clause 13.8.4) is calculated and applied for WT strength and stability check as per CSA S16-09 clause 13.8.3
11. For all WT sections, only sections with rx > ry get higher capacity when Lx=1.0L and Ly=0.5L These sections are

WT100x11			
WT125x12.5	WT125x16.5	WT125x19.5	WT125x22.5
WT155x19.5	WT155x22.5	WT155x26	

These sections are marked with “*” in the chart

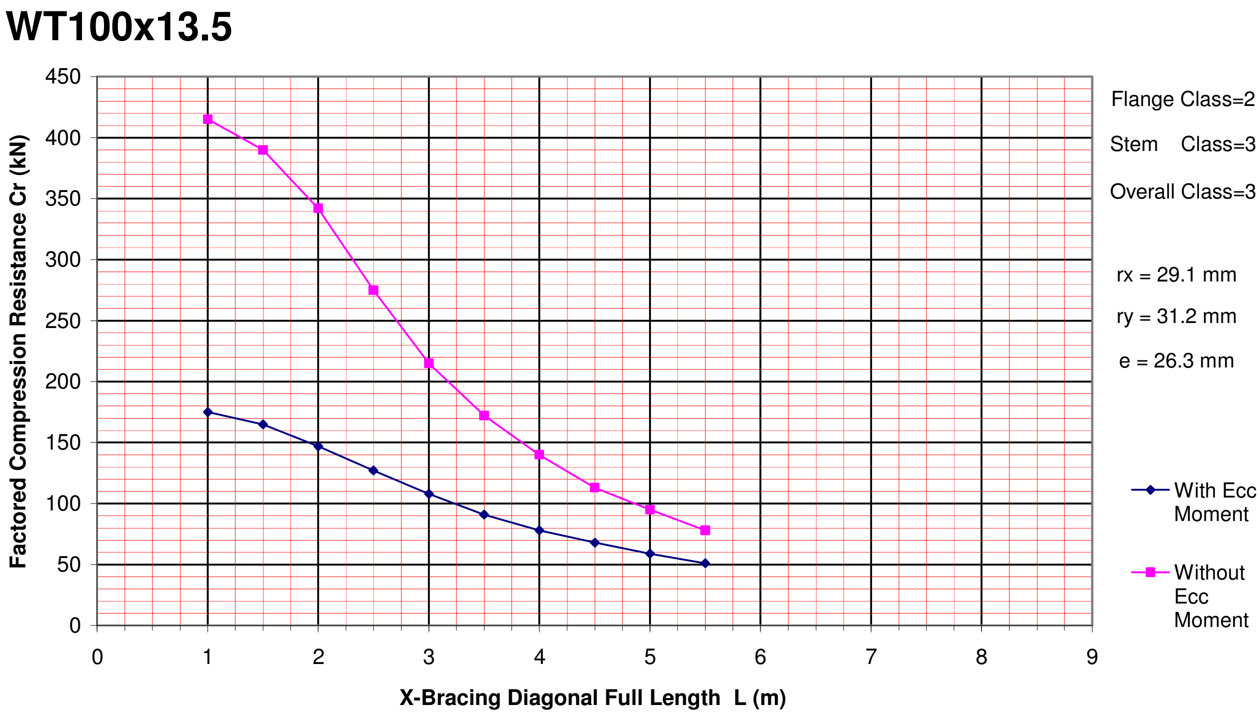
*WT100x11



*WT100x11

X-Brace Diagonal Full Length L (m)	Cr - ECC	Cr - No ECC
	kN	kN
1.0	152	350
1.5	145	345
2.0	130	300
2.5	113	245
3.0	97	195
3.5	83	158
4.0	72	128
4.5	62	105
5.0	54	88
5.5	47	74
6.0	42	63
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0

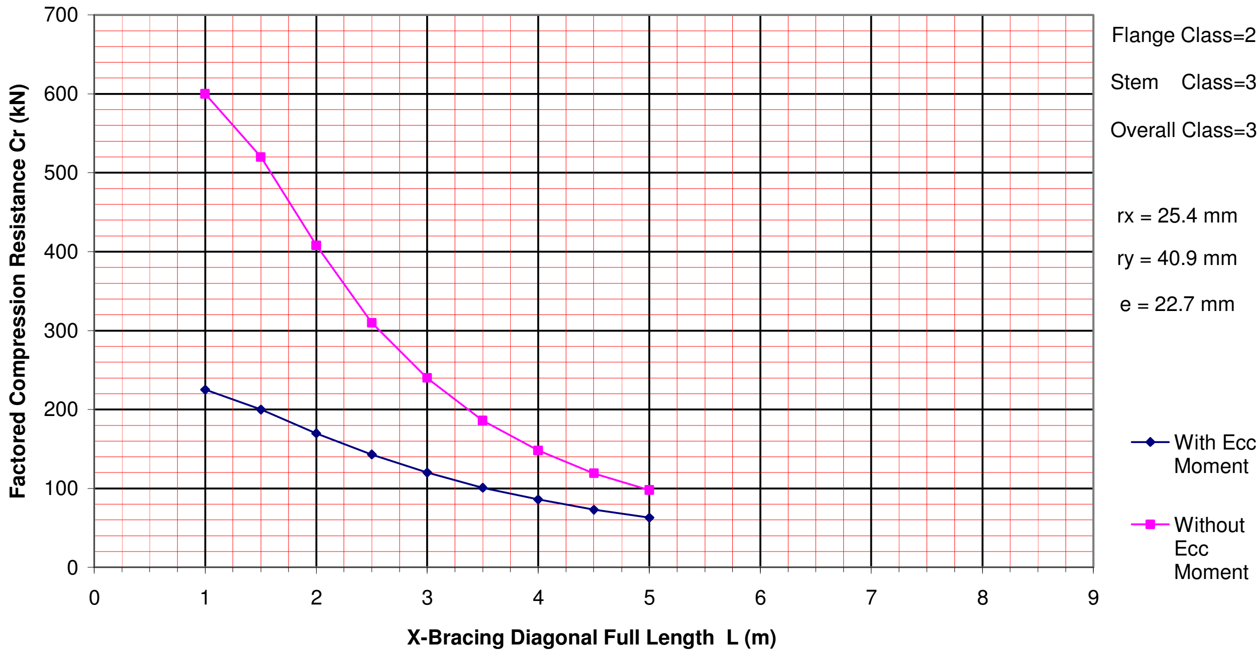
WT100x13.5



WT100x13.5

X-Brace Diagonal Full Length L (m)	Cr - ECC kN	Cr - No ECC kN
1.0	175	415
1.5	165	390
2.0	147	342
2.5	127	275
3.0	108	215
3.5	91	172
4.0	78	140
4.5	68	113
5.0	59	95
5.5	51	78
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0

WT100x18



WT100x18

X-Brace Diagonal Full Length L (m)	Cr - ECC	Cr - No ECC
	kN	kN
1.0	225	600
1.5	200	520
2.0	170	408
2.5	143	310
3.0	120	240
3.5	101	186
4.0	86	148
4.5	73	119
5.0	63	98
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0

2.1.2 WT Brace Under Tension

Design Basis & Assumption:

1. WT member capacity under tension + eccentric moment governs the design. Brace connection capacity, including the effective net area caused by shear lag (CSA S16-09 clause 12.3.3.2), does not govern the design.
2. The eccentric moment is calculated using gusset plate thickness =10mm. The eccentricity value is shown in each chart.
3. WT maximum unsupported length is calculated as L_{max} = min(rx , ry) x 300 as per CSA S16-09 clause 10.4.2.2
4. WT brace design yield strength = 50 ksi =345 MPa
5. WT flexural capacity is calculated based on AISC 360-05 section F9, tee stem in compression case.

T_r – WT factored tension resistance capacity considering eccentric moment, CSA S16-09 clause 13.9

$T_{1.0}$ – 100% of (ϕ A_g F_y) CSA S16-09 clause 13.2 (a) (i)

$T_{0.5}$ – 50% of (ϕ A_g F_y) , this is normally the value used for connection design for connections without specified design force

L_{max} – Maximum allowed brace unsupported length, CSA S16-09 clause 10.4.2.2

WT100x11

WT Tension Capacity

T_r = **126** kN L_{max} = **6690** mm $T_{1.0}$ = 444 kN $T_{0.5}$ = 222 kN

WT Section Properties

r_x = 30.9 mm r_y = 22.3 mm   e = 30.3 mm

WT Section Class

FLG= 1   Stem= 3   Overall = 3

WT100x13.5

WT Tension Capacity

T_r = **147** kN L_{max} = **8730** mm $T_{1.0}$ = 528 kN $T_{0.5}$ = 264 kN

WT Section Properties

r_x = 29.1 mm r_y = 31.2 mm   e = 26.3 mm

WT Section Class

FLG= 2   Stem= 3   Overall = 3

WT100x15.5

WT Tension Capacity

T_r = **167** kN L_{max} = **8550** mm $T_{1.0}$ = 621 kN $T_{0.5}$ = 311 kN

WT Section Properties

r_x = 28.5 mm r_y = 32.0 mm   e = 26.1 mm

WT Section Class

FLG= 1   Stem= 3   Overall = 3

T_r – WT factored tension resistance capacity considering eccentric moment, CSA S16-09 clause 13.9

$T_{1.0}$ – 100% of (ϕ A_g F_y) CSA S16-09 clause 13.2 (a) (i)

$T_{0.5}$ – 50% of (ϕ A_g F_y) , this is normally the value used for connection design for connections without specified design force

L_{max} – Maximum allowed brace unsupported length, CSA S16-09 clause 10.4.2.2

WT100x18

WT Tension Capacity

T_r = **180** kN L_{max} = **7620** mm $T_{1.0}$ = 711 kN $T_{0.5}$ = 356 kN

WT Section Properties

r_x = 25.4 mm r_y = 40.9 mm e = 22.7 mm

WT Section Class

FLG= 2 Stem= 3 Overall = 3

WT100x21

WT Tension Capacity

T_r = **205** kN L_{max} = **7770** mm $T_{1.0}$ = 826 kN $T_{0.5}$ = 413 kN

WT Section Properties

r_x = 25.9 mm r_y = 41.2 mm e = 23.8 mm

WT Section Class

FLG= 1 Stem= 3 Overall = 3

WT100x23

WT Tension Capacity

T_r = **223** kN L_{max} = **7410** mm $T_{1.0}$ = 910 kN $T_{0.5}$ = 455 kN

WT Section Properties

r_x = 24.7 mm r_y = 51.2 mm e = 22.0 mm

WT Section Class

FLG= 3 Stem= 3 Overall = 3

2.2 DOUBLE ANGLE BRACE

2.2.1 Double Angle Brace Under Compression

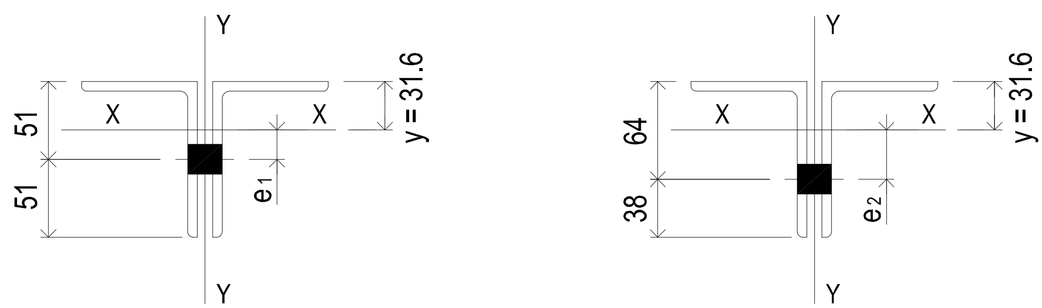
2.2.1.1 Vertical Double Angle Brace

Design Basis & Assumption

1. Assume gusset plate thickness = 10mm → section radius of gyration ry is calculated based on double angle back-to-back with 10mm gap
2. Assume the double angle brace is bolt connected and this will create eccentricity between bolt center line (force center line) and section's X-X axis line. The eccentricity is calculated as the maximum of
   - e_1 calculated assuming bolt center line is at center line of angle leg
   - e_2 calculated assuming bolt center line is at AISC recommended gauge line of angle leg

   In the following example the eccentricity is calculated as $e = \max(e_1 , e_2) = 32.4$mm

2L102x76x6.4 LLBB BOLTED VERTICAL BRACE ECCENTRICITY



BOLT AT LEG CENTER LINE

$e_1 = 51 - 31.6 = 19.4$ mm

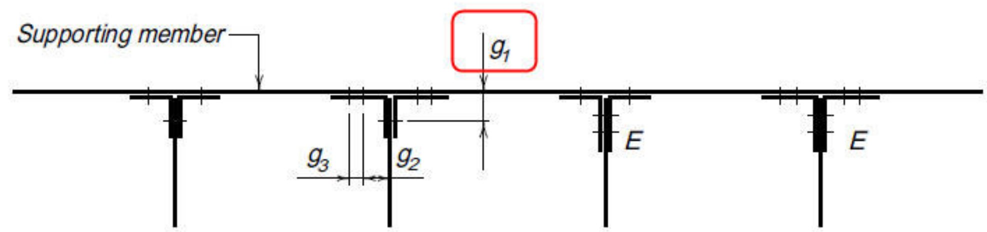
BOLT AT GAUGE LINE

$e_2 = 64 - 31.6 = 32.4$ mm

3. The bolt gauge used for double angle connection is based on AISC Manual of Steel Construction 2nd Edition Figure 9-5 on Page 9-13

All-Bolted Double-Angle Connections

Tables 9-2 are design aids for all-bolted double-angle connections. Design strengths are tabulated for supported and supporting member material, as well as angle material with



E indicates that eccentricity must be considered in this leg. Gages g_1, g_2, g_3 are usual gages as shown below

Usual gages* in angle legs, in.														
Leg	**8**	**7**	**6**	**5**	**4**	**3½**	**3**	**2½**	**2**	**1¾**	**1½**	**1⅜**	**1¼**	**1**
g_1	4½	4	3½	3	2½	2	1¾	1⅜	1⅛	1	⅞	⅞	¾	⅝
g_2	3	2½	2¼	2										
g_3	3	3	2½	1¾										
*Other gages are permitted to suit specific requirements subject to clearances and edge distance limitations.														

Figure 9-5. Eccentricity in double-angle connections.

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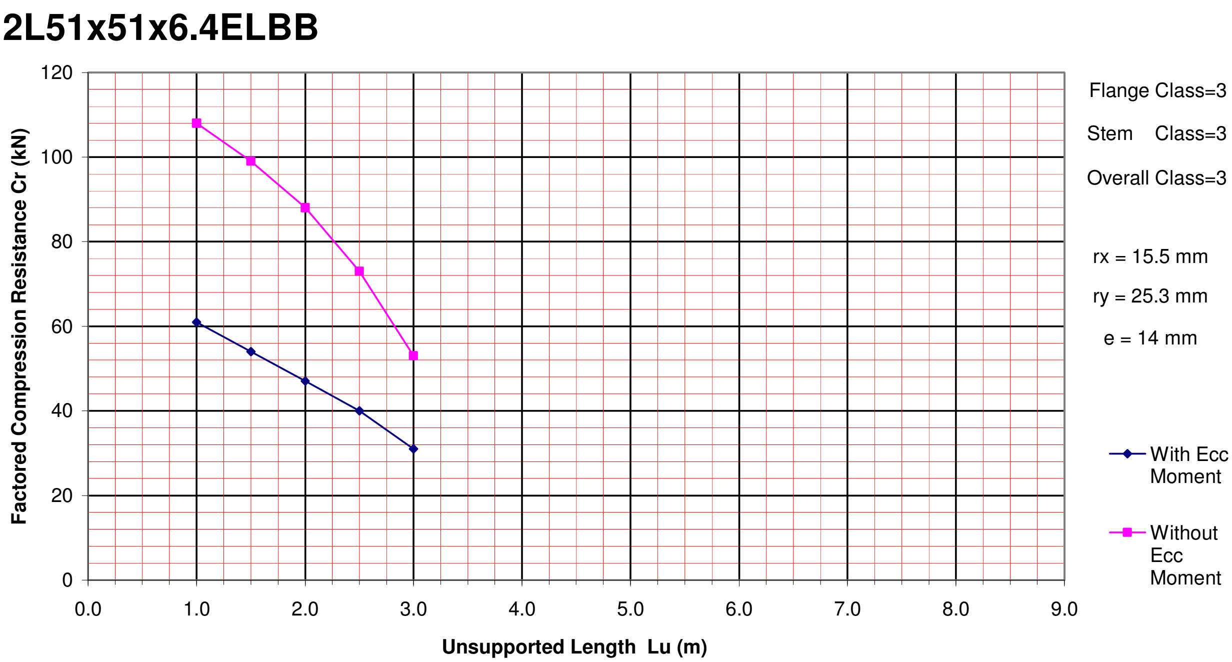
4. Double angle compression resistance capacity for two cases, with and without eccentric moment presence are presented.
5. The capacity curve stops if next 0.5m increase of unsupported length causes KL/r ratio, <u>including modified slenderness ratio considering interconnection bolt at spacing = 1200 mm</u>, exceeding 200.
6. Double angle brace design yield strength =300 MPa
7. Section class is shown in each chart. For class 4 member, the effective area is used for calculation as per CSA S16-09 clause 13.3.5 (a)
8. Double angle flexural capacity is calculated based on AISC 360-05 section F9, stem in compression case.
9. Assume double angle brace has interconnecting batten plate and 3/4" dia. bolt at spacing = 1200 mm
   Modified slenderness ratio is used as per AISC 360-05 E6-1 to account for interconnection bolt at spacing = 1200 mm
10. Assume P-δ (small delta) is not performed in steel frame stability calculation. The U1 factor (CSA S16-09 clause 13.8.4) is calculated and applied for WT strength and stability check as per CSA S16-09 clause 13.8.3

2.2.1.1.1 Double Angle Brace Compression – Non X-Brace Case

Design Basis & Assumption

1. All Design Basis & Assumption on *Section 2.2.1.1 Vertical Double Angle Brace* apply to this section
2. Assume double angle unsupported length Lx = Ly =1.0L and Kx = Ky =1.0

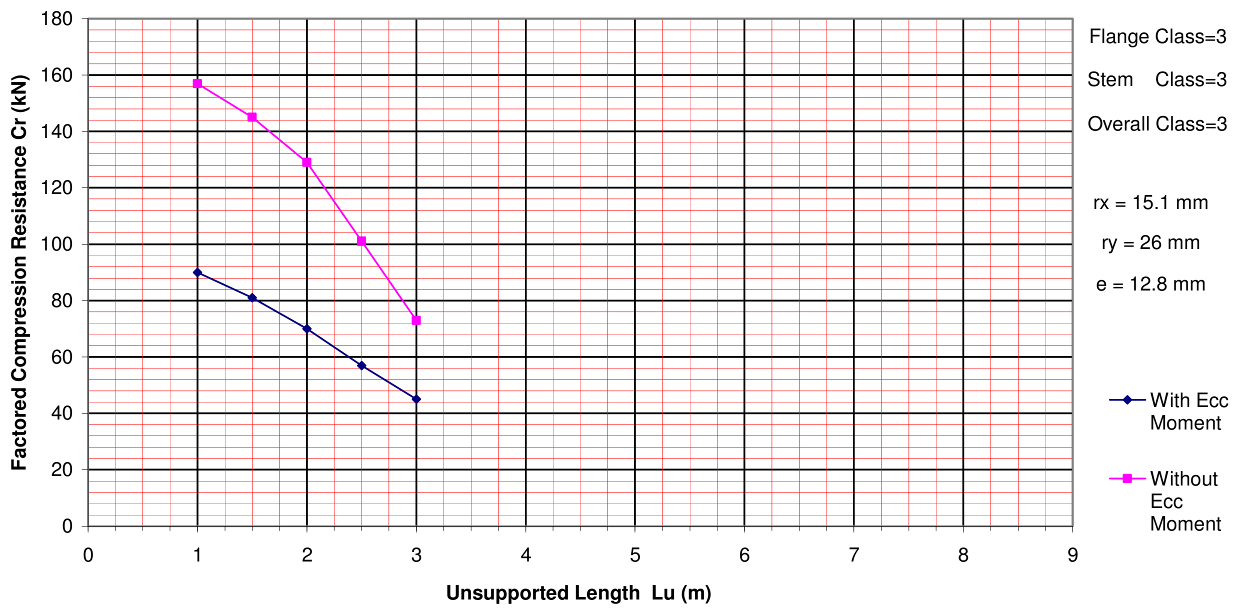
2.2.1.1.1.1 Equal Angle Back-to-Back



2L51x51x6.4ELBB

Lu	Cr - ECC	Cr - No ECC
m	kN	kN
1.0	61	108
1.5	54	99
2.0	47	88
2.5	40	73
3.0	31	53
3.5	0	0
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0
9.0	0	0

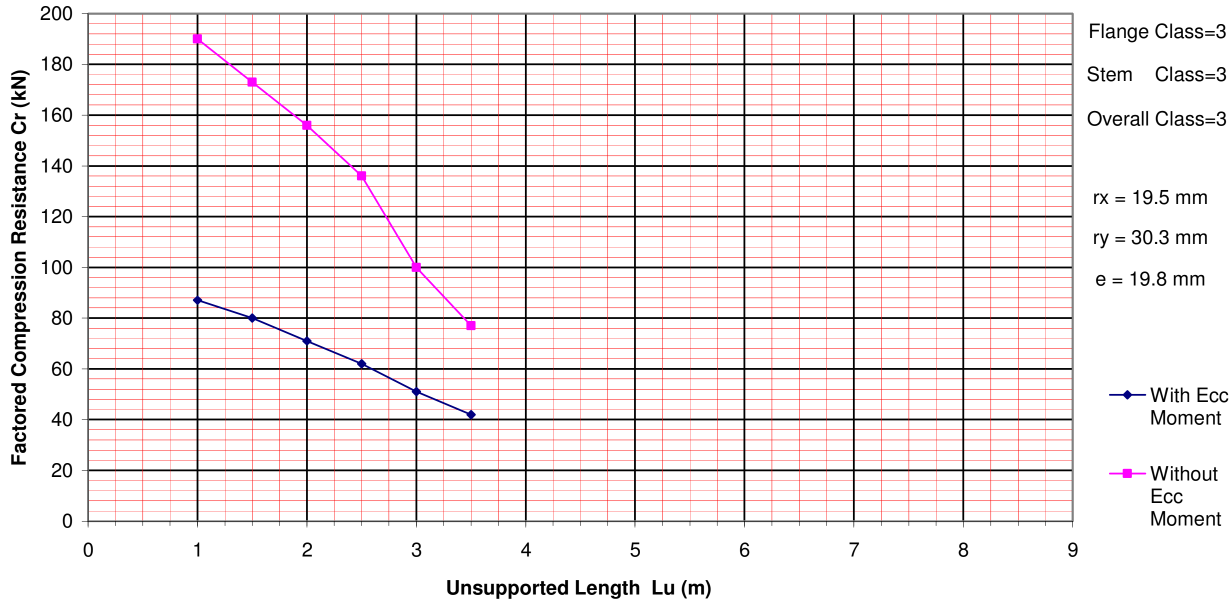
2L51x51x9.5ELBB



2L51x51x9.5ELBB

Lu	Cr - ECC	Cr - No ECC
m	kN	kN
1.0	90	157
1.5	81	145
2.0	70	129
2.5	57	101
3.0	45	73
3.5	0	0
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0
9.0	0	0

2L64x64x6.4ELBB



2L64x64x6.4ELBB

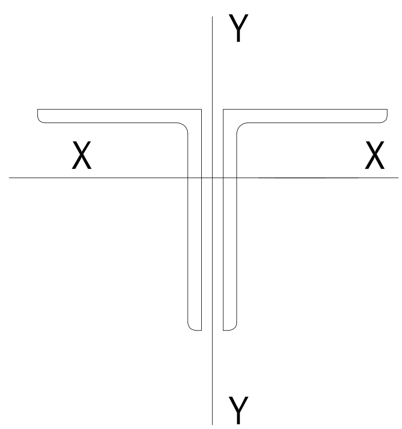
Lu	Cr - ECC	Cr - No ECC
m	kN	kN
1.0	87	190
1.5	80	173
2.0	71	156
2.5	62	136
3.0	51	100
3.5	42	77
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0
9.0	0	0

2.2.1.1.2 Double Angle Brace Compression –X-Brace Case

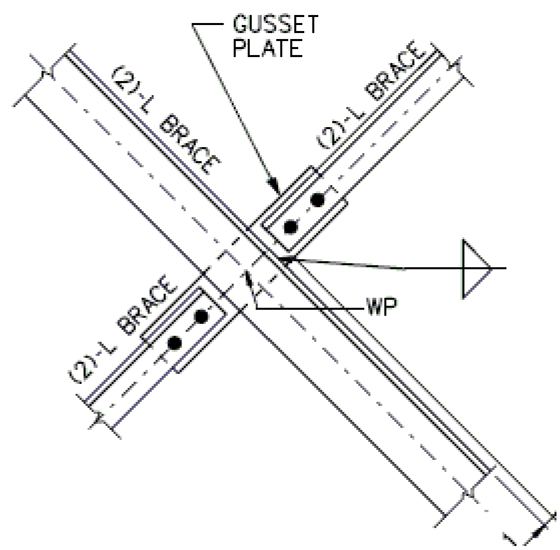
Design Basis & Assumption

1. Design charts in this section apply to vertical double angle X-bracing
2. All Design Basis & Assumption on *Section 2.2.1.1 Vertical Double Angle Brace* apply to this section
3. For vertical double angle X-bracing, use unsupported length Lx = 0.5L and Ly =1.0L , Kx = Ky =1.0

Double Angle Section Local Axis

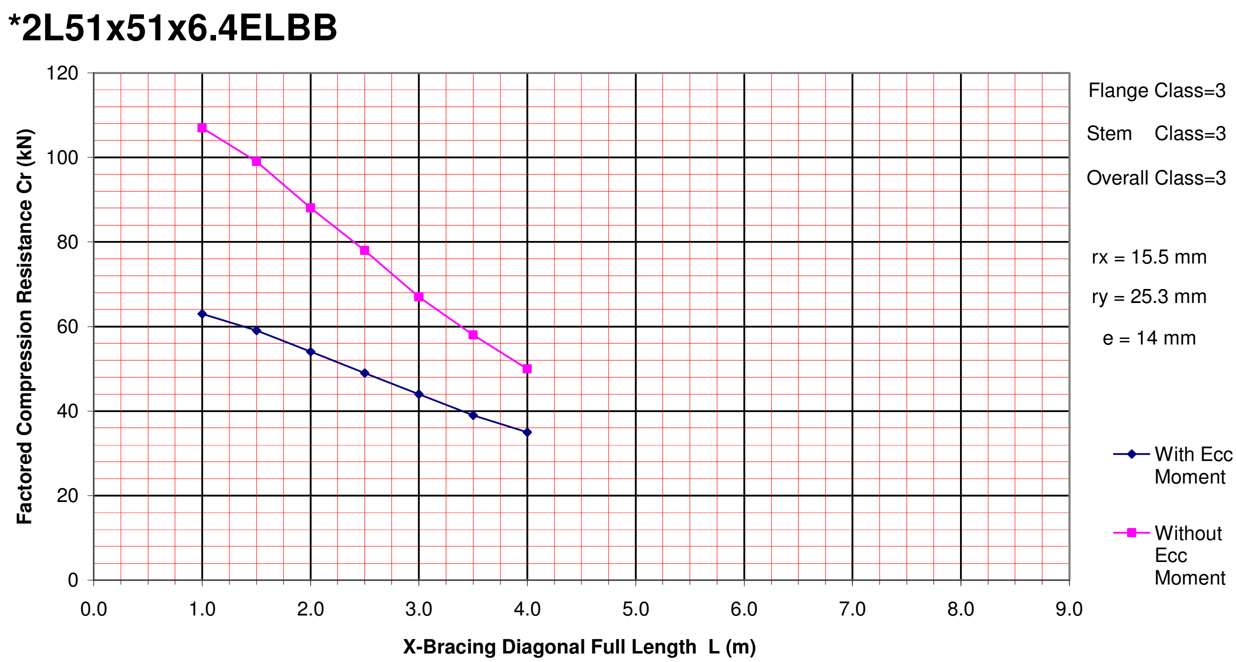


Vertical Double Angle X-Bracing
Intersect Point Connection



4. For all equal leg double angle sections, rx < ry, all sections get higher capacity due to X-bracing's unsupported length Lx = 0.5L and Ly =1.0L
5. All sections getting higher capacity due to rx < ry are marked with "*" in the chart

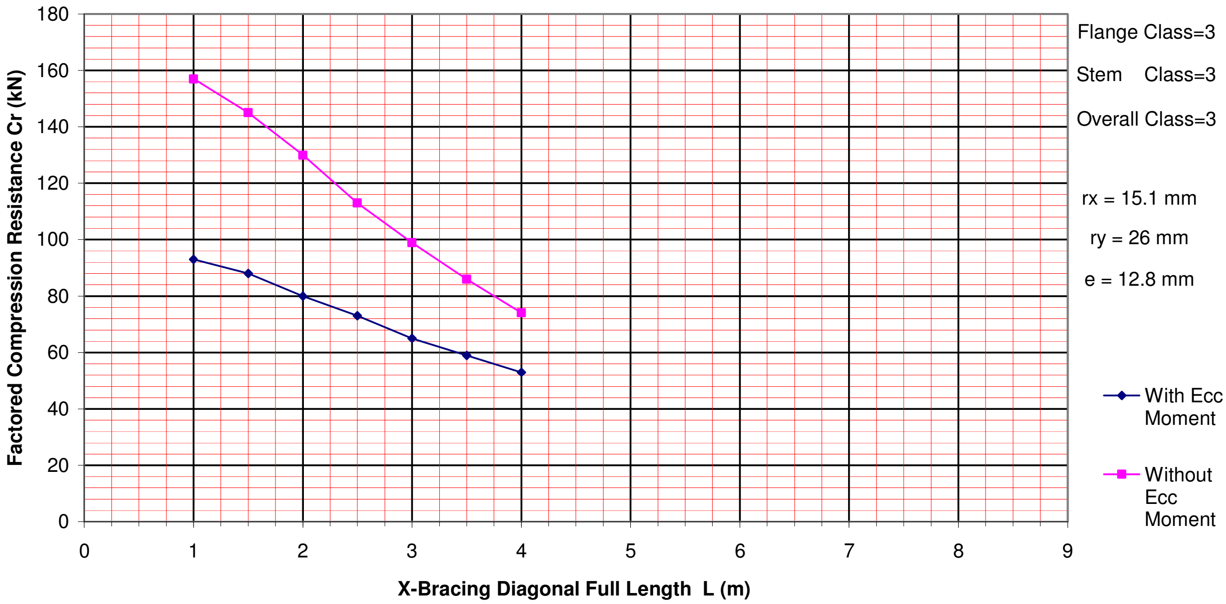
2.2.1.1.2.1 Equal Angle Back-to-Back



*2L51x51x6.4ELBB

X-Brace Diagonal Full L (m)	Cr - ECC	Cr - No ECC
	kN	kN
1.0	63	107
1.5	59	99
2.0	54	88
2.5	49	78
3.0	44	67
3.5	39	58
4.0	35	50
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0
9.0	0	0

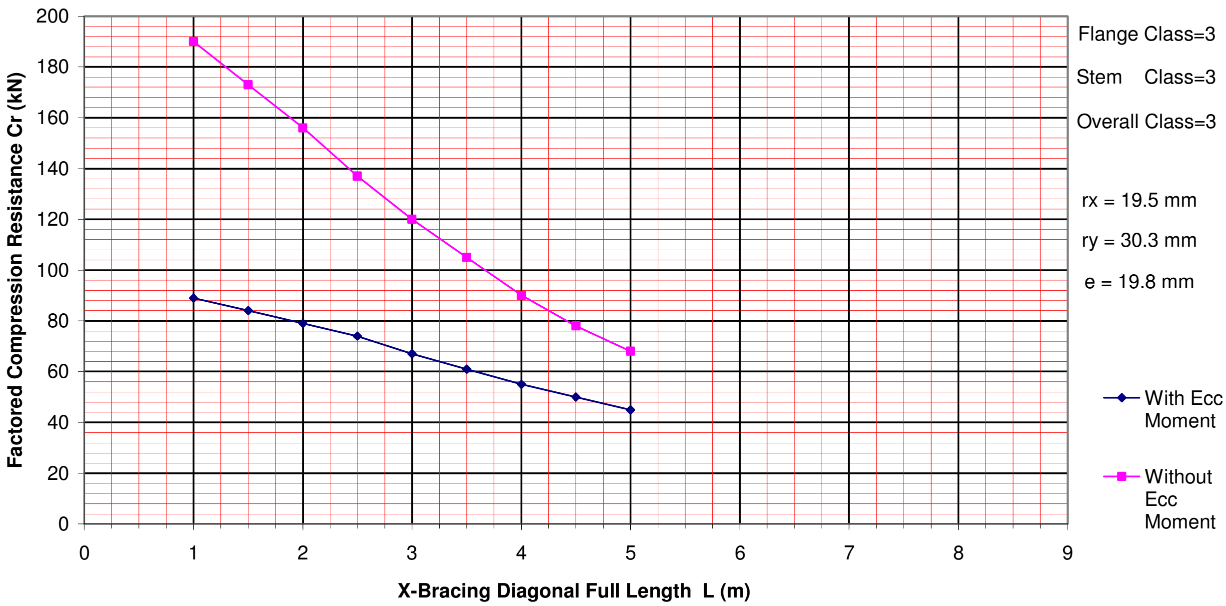
*2L51x51x9.5ELBB



*2L51x51x9.5ELBB

X-Brace Diagonal Full L (m)	Cr - ECC kN	Cr - No ECC kN
1.0	93	157
1.5	88	145
2.0	80	130
2.5	73	113
3.0	65	99
3.5	59	86
4.0	53	74
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0
9.0	0	0

*2L64x64x6.4ELBB

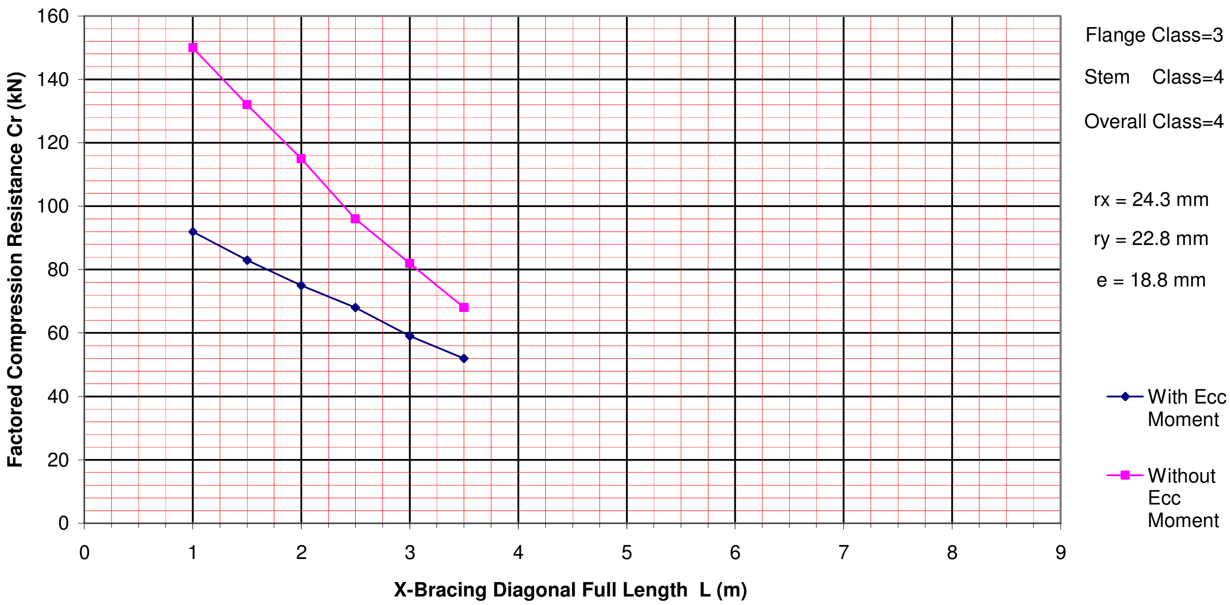


*2L64x64x6.4ELBB

X-Brace Diagonal Full L (m)	Cr - ECC	Cr - No ECC
	kN	kN
1.0	89	190
1.5	84	173
2.0	79	156
2.5	74	137
3.0	67	120
3.5	61	105
4.0	55	90
4.5	50	78
5.0	45	68
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0
9.0	0	0

2.2.1.1.2.2 Unequal Angle Long Leg Back-to-Back

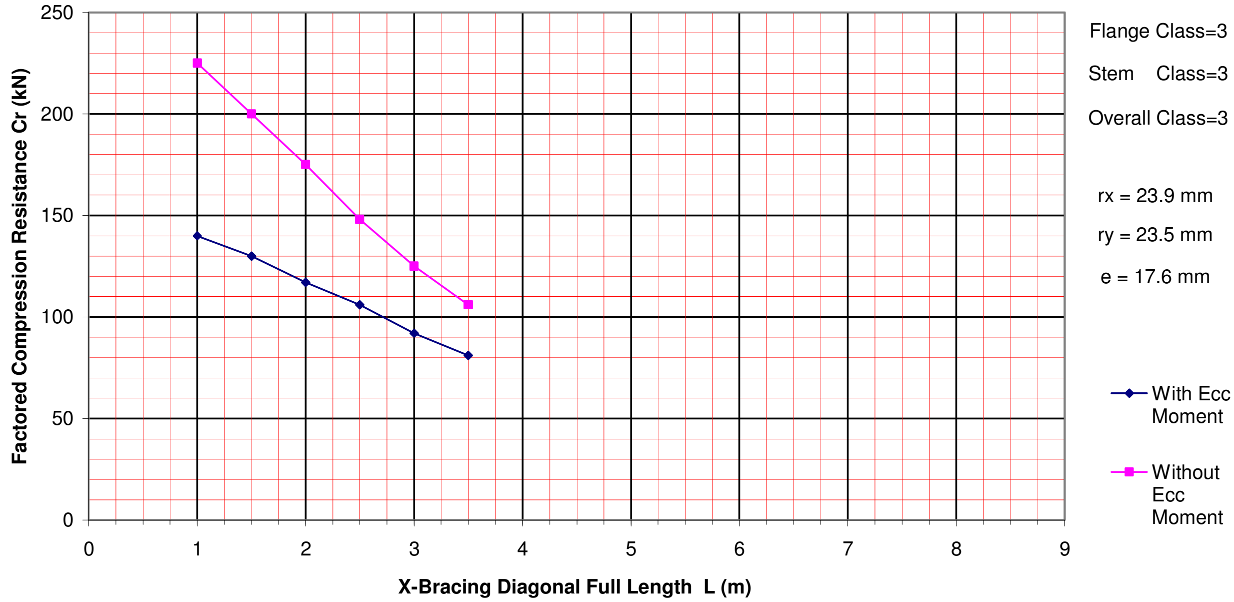
2L76x51x6.4LLBB



2L76x51x6.4LLBB

X-Brace Diagonal Full L (m)	Cr - ECC	Cr - No ECC
	kN	kN
1.0	92	150
1.5	83	132
2.0	75	115
2.5	68	96
3.0	59	82
3.5	52	68
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0

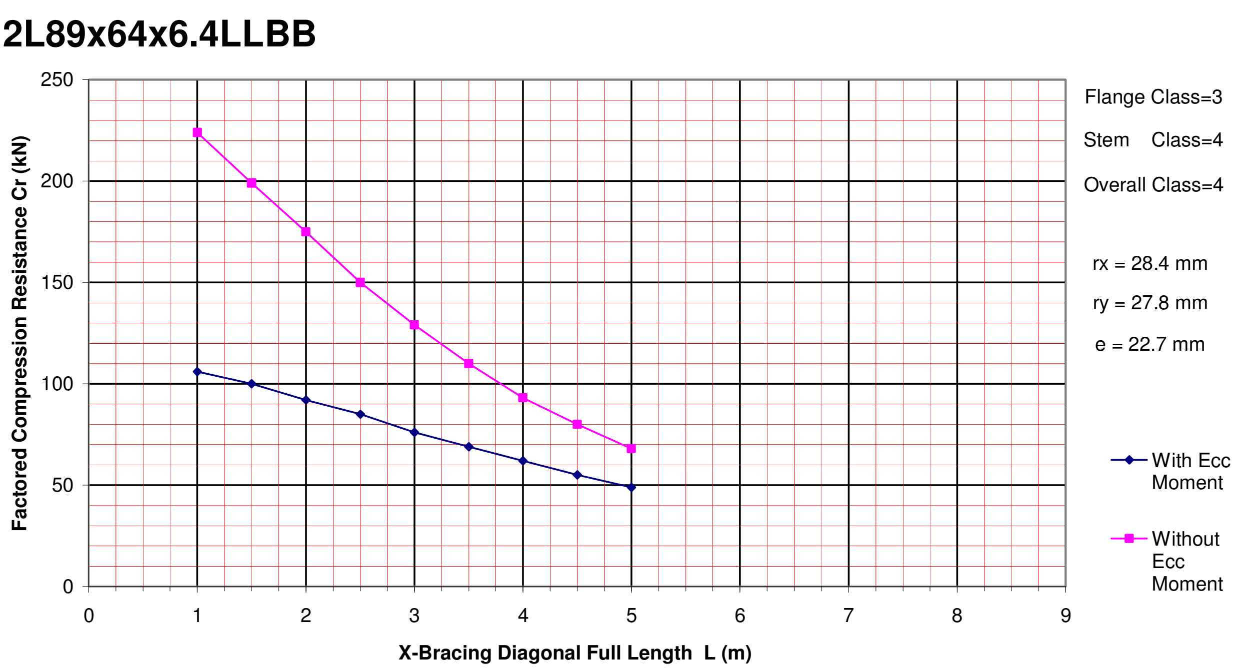
2L76x51x9.5LLBB



2L76x51x9.5LLBB

X-Brace Diagonal Full L (m)	Cr - ECC	Cr - No ECC
	kN	kN
1.0	140	225
1.5	130	200
2.0	117	175
2.5	106	148
3.0	92	125
3.5	81	106
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0

2L89x64x6.4LLBB



2L89x64x6.4LLBB

X-Brace Diagonal Full L (m)	Cr - ECC kN	Cr - No ECC kN
1.0	106	224
1.5	100	199
2.0	92	175
2.5	85	150
3.0	76	129
3.5	69	110
4.0	62	93
4.5	55	80
5.0	49	68
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0

2.2.1.2 Horizontal Double Angle Brace

Design Basis & Assumption

1. Double angle section radius of gyration ry is calculated based on double angle back-to-back with 0mm gap
2. The eccentric moment is calculated using gusset plate thickness =10mm. The eccentricity value is shown in each chart.
3. Double angle compression resistance capacity for two cases, with and without eccentric moment presence are presented.
4. The capacity curve stops if next 0.5m increase of unsupported length causes KL/r ratio, including modified slenderness ratio considering interconnection bolt at spacing = 1200 mm, exceeding 200.
5. Double angle brace design yield strength =300 MPa
6. Section class is shown in each chart. For class 4 member, the effective area is used for calculation as per CSA S16-09 clause 13.3.5 (a)
7. Double angle flexural capacity is calculated based on AISC 360-05 section F9, stem in tension case.
8. Assume double angle brace has interconnecting 3/4" dia. bolt at spacing = 1200 mm . Modified slenderness ratio as per AISC 360-05 E6-1 is used to account for interconnection bolt at spacing = 1200 mm .
9. Assume P-δ (small delta) is not performed in steel frame stability calculation. The U1 factor (CSA S16-09 clause 13.8.4) is calculated and applied for WT strength and stability check as per CSA S16-09 clause 13.8.3

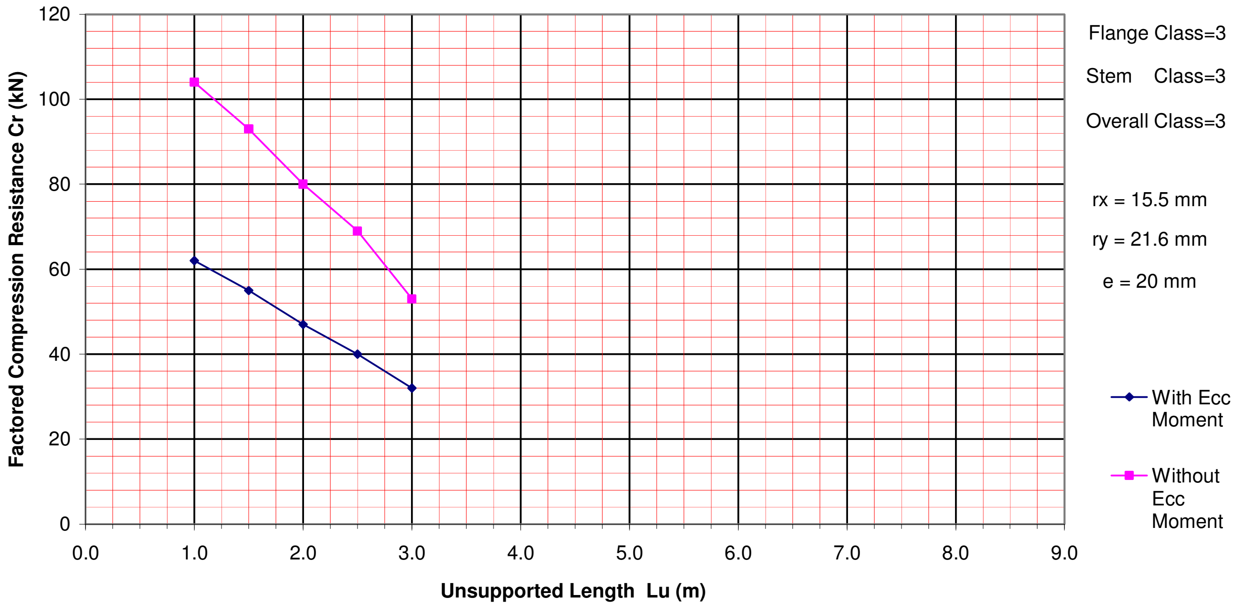
2.2.1.2.1 Double Angle Brace Compression – Non X-Brace Case

Design Basis & Assumption

1. All Design Basis & Assumption on *Section 2.2.1.2 Horizontal Double Angle Brace* apply to this section
2. Assume double angle unsupported length Lx = Ly =1.0L and Kx = Ky =1.0

2.2.1.2.1.1 Equal Angle Back-to-Back

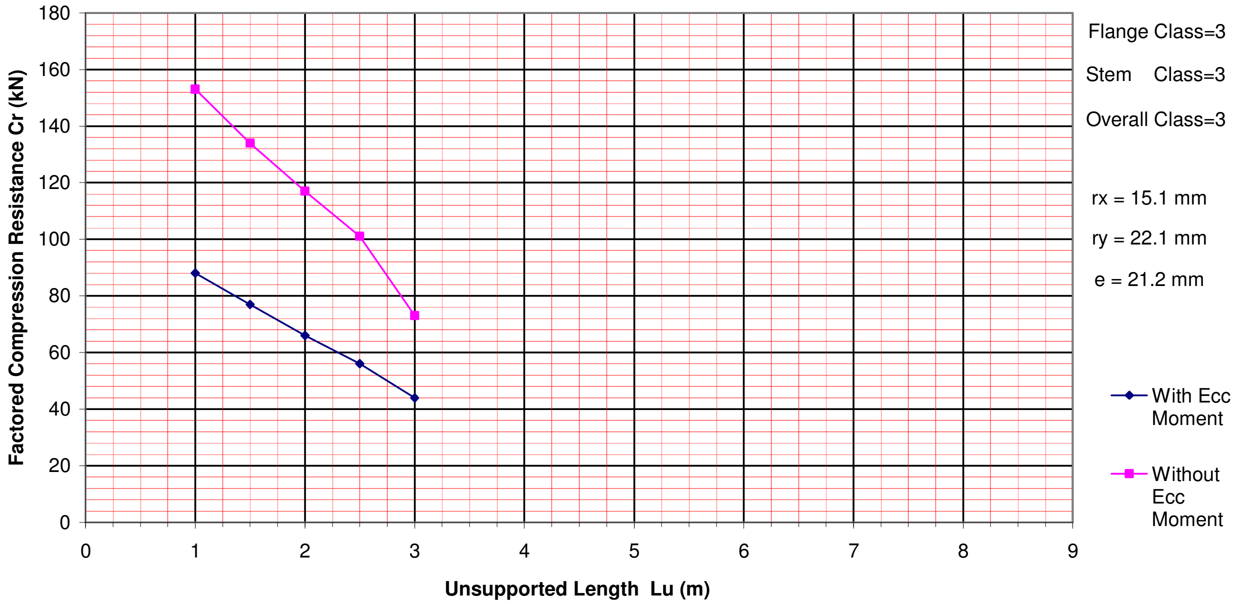
2L51x51x6.4ELBB



2L51x51x6.4ELBB

Lu	Cr - ECC	Cr - No ECC
m	kN	kN
1.0	62	104
1.5	55	93
2.0	47	80
2.5	40	69
3.0	32	53
3.5	0	0
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0

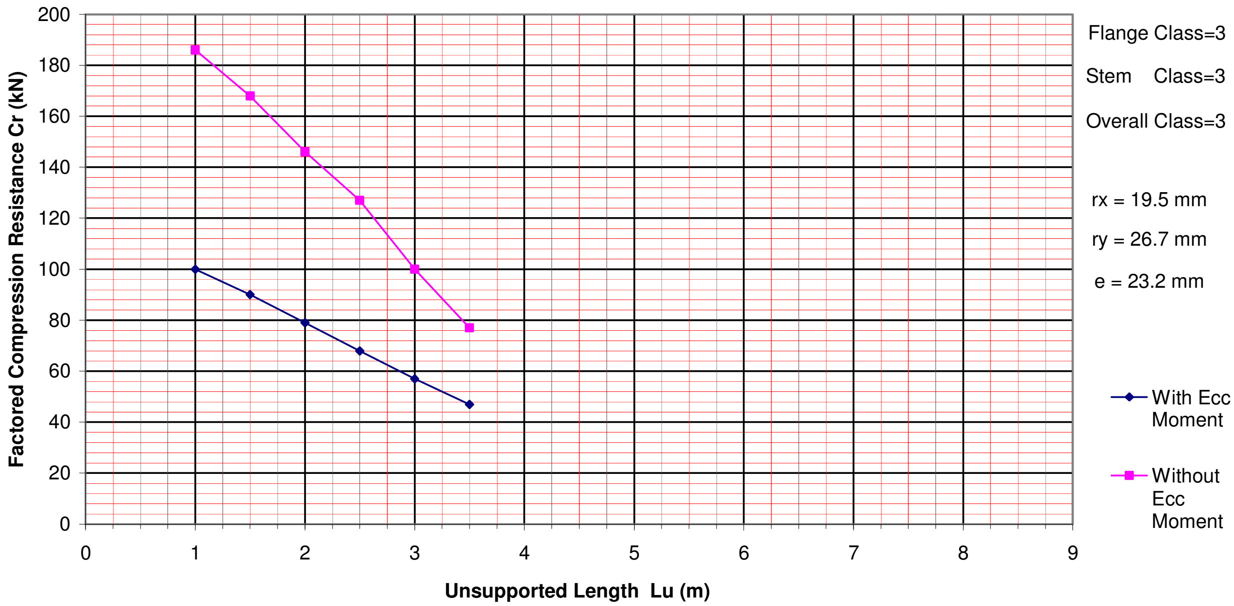
2L51x51x9.5ELBB



2L51x51x9.5ELBB

Lu	Cr - ECC	Cr - No ECC
m	kN	kN
1.0	88	153
1.5	77	134
2.0	66	117
2.5	56	101
3.0	44	73
3.5	0	0
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0

2L64x64x6.4ELBB



2L64x64x6.4ELBB

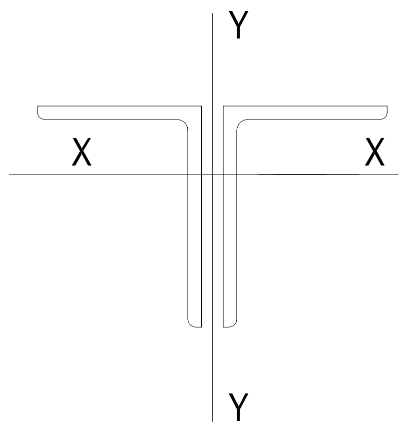
Lu	Cr - ECC	Cr - No ECC
m	kN	kN
1.0	100	186
1.5	90	168
2.0	79	146
2.5	68	127
3.0	57	100
3.5	47	77
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0

2.2.1.2.2 Double Angle Brace Compression – X-Brace Case

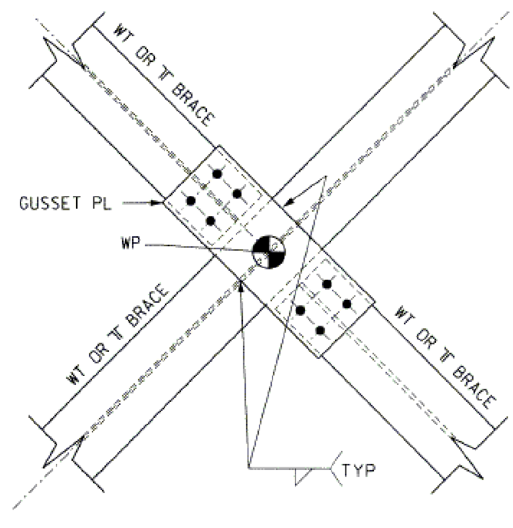
Design Basis & Assumption

1. Design charts in this section apply to horizontal double angle X-bracing
2. All Design Basis & Assumption on *Section 2.2.1.2 Horizontal Double Angle Brace* apply to this section
3. For horizontal double angle X-bracing, use unsupported length Lx = 1.0L and Ly =0.5L , Kx = Ky =1.0

Double Angle Section Local Axis

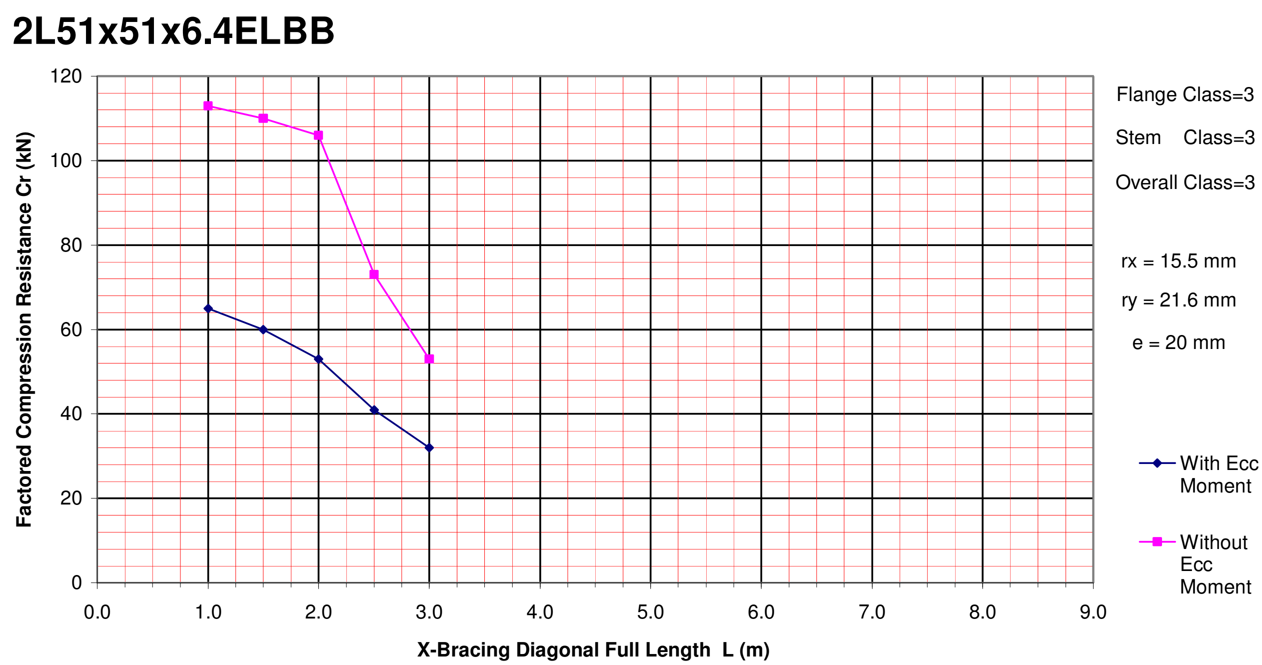


Horizontal Double Angle X-Bracing
Intersect Point Connection



4. For all equal leg double angle sections, rx < ry, none of the equal leg angle section gets higher capacity due to X-bracing's unsupported length Lx = 1.0L and Ly =0.5L

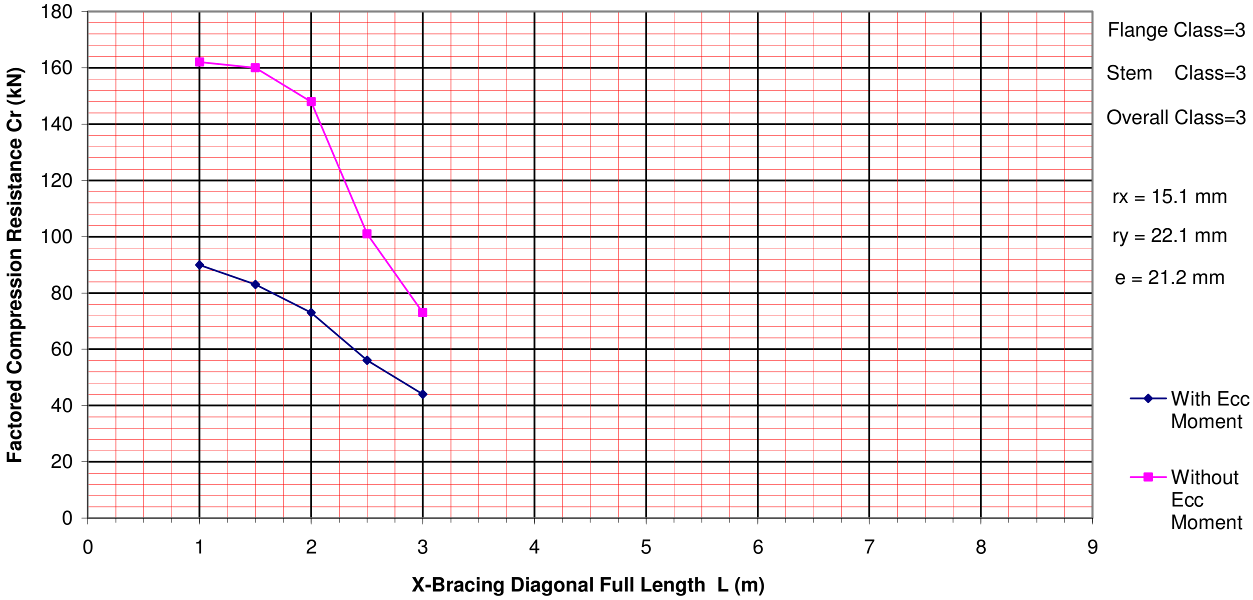
2.2.1.2.2.1 Equal Angle Back-to-Back



2L51x51x6.4ELBB

X-Bracing Diagonal Full Length L (m)	Cr - ECC	Cr - No ECC
	kN	kN
1.0	65	113
1.5	60	110
2.0	53	106
2.5	41	73
3.0	32	53
3.5	0	0
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0

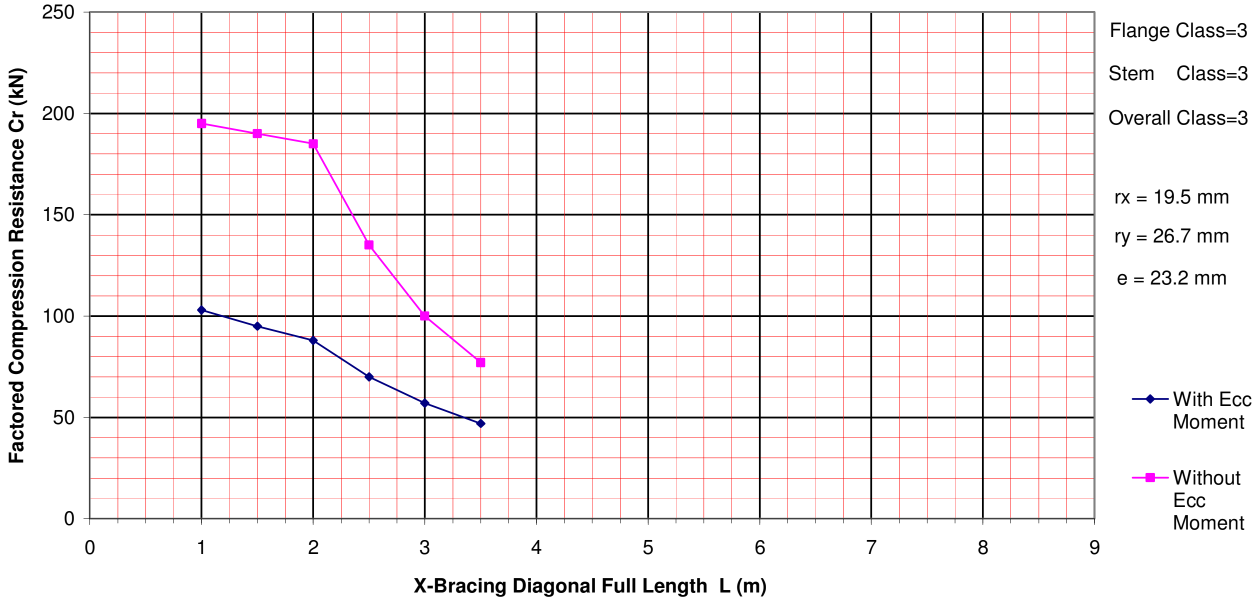
2L51x51x9.5ELBB



2L51x51x9.5ELBB

X-Bracing Diagonal Full Length L (m)	Cr - ECC	Cr - No ECC
	kN	kN
1.0	90	162
1.5	83	160
2.0	73	148
2.5	56	101
3.0	44	73
3.5	0	0
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0

2L64x64x6.4ELBB



2L64x64x6.4ELBB

X-Bracing Diagonal Full Length L (m)	Cr - ECC	Cr - No ECC
	kN	kN
1.0	103	195
1.5	95	190
2.0	88	185
2.5	70	135
3.0	57	100
3.5	47	77
4.0	0	0
4.5	0	0
5.0	0	0
5.5	0	0
6.0	0	0
6.5	0	0
7.0	0	0
7.5	0	0
8.0	0	0
8.5	0	0

2.2.2 Double Angle Brace Under Tension

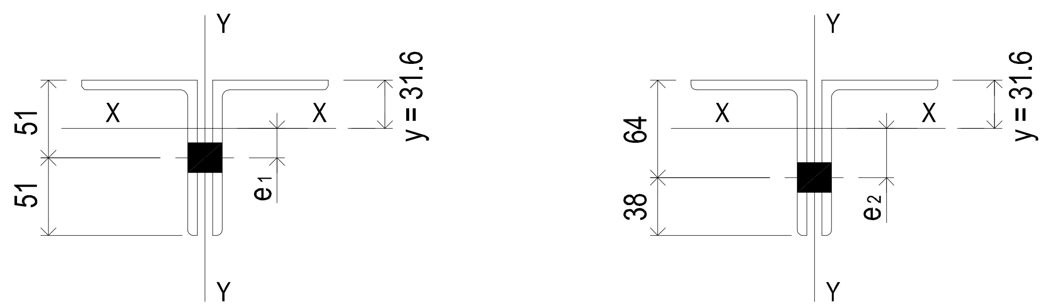
2.2.2.1 Vertical Double Angle Brace

Design Basis & Assumption

1. Assume gusset plate thickness = 10mm → section radius of gyration ry is calculated based on double angle back-to-back with 10mm gap
2. Assume the double angle brace is bolt connected and this will create eccentricity between bolt center line (force center line) and section's X-X axis line. The eccentricity is calculated as the maximum of
   - e_1 calculated assuming bolt center line is at center line of angle leg
   - e_2 calculated assuming bolt center line is at AISC recommended gauge line of angle leg

   In the following example the eccentricity is calculated as $e = \max(e_1 , e_2) = 32.4$mm

2L102x76x6.4 LLBB BOLTED VERTICAL BRACE ECCENTRICITY



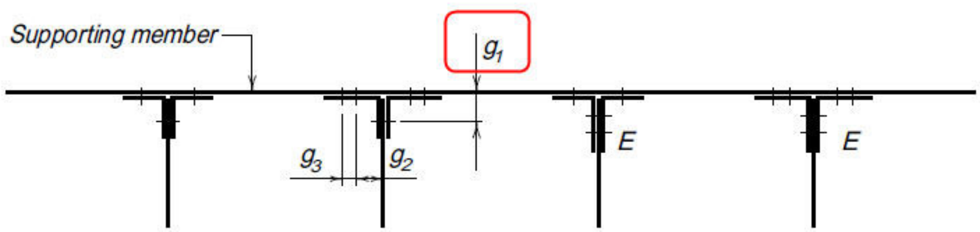
BOLT AT LEG CENTER LINE
$e_1 = 51 - 31.6 = 19.4$ mm

BOLT AT GAUGE LINE
$e_2 = 64 - 31.6 = 32.4$ mm

3. The bolt gauge used for double angle connection is based on AISC Manual of Steel Construction 2nd Edition Figure 9-5 on Page 9-13

All-Bolted Double-Angle Connections

Tables 9-2 are design aids for all-bolted double-angle connections. Design strengths are tabulated for supported and supporting member material, as well as angle material with



E indicates that eccentricity must be considered in this leg. Gages g_1, g_2, g_3 are usual gages as shown below

Usual gages* in angle legs, in.														
Leg	**8**	**7**	**6**	**5**	**4**	**3½**	**3**	**2½**	**2**	**1¾**	**1½**	**1⅜**	**1¼**	**1**
g_1	4½	4	3½	3	2½	2	1¾	1⅜	1⅛	1	⅞	⅞	¾	⅝
g_2	3	2½	2¼	2										
g_3	3	3	2½	1¾										

*Other gages are permitted to suit specific requirements subject to clearances and edge distance limitations.

Figure 9-5. Eccentricity in double-angle connections.

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4. Double angle brace design yield strength =300 MPa
5. Section class is shown in each chart. For class 4 member, the effective area is used for calculation as per CSA S16-09 clause 13.3.5 (a)
6. Double angle flexural capacity is calculated based on AISC 360-05 section F9, stem in tension case.
6. Double angle member capacity under tension + eccentric moment governs the design. Brace connection capacity, including the effective net area caused by shear lag (CSA S16-09 clause 12.3.3.2), does not govern the design.
7. Double angle maximum unsupported length is calculated as L_{max} = min(rx , ry) x 300 as per CSA S16-09 clause 10.4.2.2

2.2.2.1.1 Equal Angle Back-to-Back

T_r – Double angle factored tension resistance capacity considering eccentric moment, CSA S16-09 clause 13.9

$T_{1.0}$ – 100% of (ϕ A_g F_y) CSA S16-09 clause 13.2 (a) (i)

$T_{0.5}$ – 50% of (ϕ A_g F_y) , this is normally the value used for connection design for connections without specified design force

L_{max} – Maximum allowed brace unsupported length, CSA S16-09 clause 10.4.2.2

2L51x51x6.4ELBB

Double Angle Tension Capacity

T_r = **140** kN L_{max} = **4650** mm $T_{1.0}$ = 327 kN $T_{0.5}$ = 163 kN

Double Angle Section Properties

r_x = 15.5 mm r_y = 25.3 mm e = 14.0 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

2L51x51x9.5ELBB

Double Angle Tension Capacity

T_r = **210** kN L_{max} = **4530** mm $T_{1.0}$ = 473 kN $T_{0.5}$ = 236 kN

Double Angle Section Properties

r_x = 15.1 mm r_y = 26.0 mm e = 12.8 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

T_r – Double angle factored tension resistance capacity considering eccentric moment, CSA S16-09 clause 13.9

$T_{1.0}$ – 100% of (ϕ A_g F_y) CSA S16-09 clause 13.2 (a) (i)

$T_{0.5}$ – 50% of (ϕ A_g F_y) , this is normally the value used for connection design for connections without specified design force

L_{max} – Maximum allowed brace unsupported length, CSA S16-09 clause 10.4.2.2

2L64x64x6.4ELBB

Double Angle Tension Capacity

T_r = **165** kN L_{max} = **5850** mm $T_{1.0}$ = 413 kN $T_{0.5}$ = 207 kN

Double Angle Section Properties

r_x = 19.5 mm r_y = 30.3 mm e = 19.8 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

2L64x64x7.9ELBB

Double Angle Tension Capacity

T_r = **208** kN L_{max} = **5790** mm $T_{1.0}$ = 510 kN $T_{0.5}$ = 255 kN

Double Angle Section Properties

r_x = 19.3 mm r_y = 30.7 mm e = 19.2 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

2L64x64x9.5ELBB

Double Angle Tension Capacity

T_r = **250** kN L_{max} = **5730** mm $T_{1.0}$ = 605 kN $T_{0.5}$ = 302 kN

Double Angle Section Properties

r_x = 19.1 mm r_y = 31.0 mm e = 18.6 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

2.2.2.1.2 Unequal Angle Long Leg Back-to-Back

T_r – Double angle factored tension resistance capacity considering eccentric moment, CSA S16-09 clause 13.9

$T_{1.0}$ – 100% of (ϕ A_g F_y) CSA S16-09 clause 13.2 (a) (i)

$T_{0.5}$ – 50% of (ϕ A_g F_y) , this is normally the value used for connection design for connections without specified design force

L_{max} – Maximum allowed brace unsupported length, CSA S16-09 clause 10.4.2.2

2L76x51x6.4LLBB

Double Angle Tension Capacity

T_r = **196** kN L_{max} = **6840** mm $T_{1.0}$ = 413 kN $T_{0.5}$ = 207 kN

Double Angle Section Properties

r_x = 24.3 mm r_y = 22.8 mm e = 18.8 mm

Double Angle Section Class

FLG= 3 Stem= 4 Overall = 4

2L76x51x9.5LLBB

Double Angle Tension Capacity

T_r = **306** kN L_{max} = **7050** mm $T_{1.0}$ = 605 kN $T_{0.5}$ = 302 kN

Double Angle Section Properties

r_x = 23.9 mm r_y = 23.5 mm e = 17.6 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

2L89x64x6.4LLBB

Double Angle Tension Capacity

T_r = **195** kN L_{max} = **8340** mm $T_{1.0}$ = 500 kN $T_{0.5}$ = 250 kN

Double Angle Section Properties

r_x = 28.4 mm r_y = 27.8 mm e = 22.7 mm

Double Angle Section Class

FLG= 3 Stem= 4 Overall = 4

T_r – Double angle factored tension resistance capacity considering eccentric moment, CSA S16-09 clause 13.9
$T_{1.0}$ – 100% of (ϕ A_g F_y) CSA S16-09 clause 13.2 (a) (i)
$T_{0.5}$ – 50% of (ϕ A_g F_y) , this is normally the value used for connection design for connections without specified design force
L_{max} – Maximum allowed brace unsupported length, CSA S16-09 clause 10.4.2.2

2L89x64x9.5LLBB

Double Angle Tension Capacity

T_r = **360** kN L_{max} = **8400** mm $T_{1.0}$ = 734 kN $T_{0.5}$ = 367 kN

Double Angle Section Properties

r_x = 28.0 mm r_y = 28.4 mm e = 21.5 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

2L102x76x6.4LLBB

Double Angle Tension Capacity

T_r = **165** kN L_{max} = **9810** mm $T_{1.0}$ = 589 kN $T_{0.5}$ = 294 kN

Double Angle Section Properties

r_x = 32.7 mm r_y = 32.8 mm e = 32.4 mm

Double Angle Section Class

FLG= 4 Stem= 4 Overall = 4

2L102x76x7.9LLBB

Double Angle Tension Capacity

T_r = **275** kN L_{max} = **9720** mm $T_{1.0}$ = 729 kN $T_{0.5}$ = 365 kN

Double Angle Section Properties

r_x = 32.4 mm r_y = 33.1 mm e = 31.9 mm

Double Angle Section Class

FLG= 3 Stem= 4 Overall = 4

2.2.2.2 Horizontal Double Angle Brace

Design Basis & Assumption

1. Double angle member capacity under tension + eccentric moment governs the design. Brace connection capacity, including the effective net area caused by shear lag (CSA S16-09 clause 12.3.3.2), does not govern the design.
2. The eccentric moment is calculated using gusset plate thickness =10mm. The eccentricity value is shown in each chart.
3. Double angle maximum unsupported length is calculated as L_{max} = min(rx , ry) x 300 as per CSA S16-09 clause 10.4.2.2
4. Double angle brace design yield strength = 300 MPa
5. Double angle flexural capacity is calculated based on AISC 360-05 section F9, tee stem in compression case.

2.2.2.2.1 Equal Angle Back-to-Back

T_r – Double angle factored tension resistance capacity considering eccentric moment, CSA S16-09 clause 13.9

$T_{1.0}$ – 100% of (ϕ A_g F_y) CSA S16-09 clause 13.2 (a) (i)

$T_{0.5}$ – 50% of (ϕ A_g F_y) , this is normally the value used for connection design for connections without specified design force

L_{max} – Maximum allowed brace unsupported length, CSA S16-09 clause 10.4.2.2

2L51x51x6.4ELBB

Double Angle Tension Capacity

T_r = **81** kN L_{max} = **4650** mm $T_{1.0}$ = 327 kN $T_{0.5}$ = 163 kN

Double Angle Section Properties

r_x = 15.5 mm r_y = 21.6 mm e = 20.0 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

2L51x51x9.5ELBB

Double Angle Tension Capacity

T_r = **111** kN L_{max} = **4530** mm $T_{1.0}$ = 473 kN $T_{0.5}$ = 236 kN

Double Angle Section Properties

r_x = 15.1 mm r_y = 22.1 mm e = 21.2 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

T_r – Double angle factored tension resistance capacity considering eccentric moment, CSA S16-09 clause 13.9
$T_{1.0}$ – 100% of (ϕ A_g F_y) CSA S16-09 clause 13.2 (a) (i)
$T_{0.5}$ – 50% of (ϕ A_g F_y) , this is normally the value used for connection design for connections without specified design force
L_{max} – Maximum allowed brace unsupported length, CSA S16-09 clause 10.4.2.2

2L64x64x6.4ELBB

Double Angle Tension Capacity

T_r = **110** kN L_{max} = **5850** mm $T_{1.0}$ = 413 kN $T_{0.5}$ = 207 kN

Double Angle Section Properties

r_x = 19.5 mm r_y = 26.7 mm e = 23.2 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

2L64x64x7.9ELBB

Double Angle Tension Capacity

T_r = **131** kN L_{max} = **5790** mm $T_{1.0}$ = 510 kN $T_{0.5}$ = 255 kN

Double Angle Section Properties

r_x = 19.3 mm r_y = 27.0 mm e = 23.8 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3

2L64x64x9.5ELBB

Double Angle Tension Capacity

T_r = **150** kN L_{max} = **5730** mm $T_{1.0}$ = 605 kN $T_{0.5}$ = 302 kN

Double Angle Section Properties

r_x = 19.1 mm r_y = 27.2 mm e = 24.4 mm

Double Angle Section Class

FLG= 3 Stem= 3 Overall = 3