RISAConnection

Rapid Interactive Structural Analysis – Connection Design

Verification Problems

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Verification Overview

Verification Methods

We at RISA maintain a library of dozens of test problems used to validate the computational aspects of RISA programs. In this verification package we will compare RISAConnection to various design examples provided by the AISC. Note that all the examples were done using ASD design unless noted otherwise. Images reproduced in this document were taken from referenced example documents. These include:

- ***AISC Design Examples Version 14.1(February 2013 Revision)***
- ***AISC Design Guide 1, Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing)***
- ***AISC Design Guide 24, Hollow Structural Section Connections***
- ***AISC Design Guide 29, Vertical Brace Connections- Analysis and Design***
- ***AISC Seismic Design Manual (2010)***

The input for these test problems was formulated to test the performance of RISAConnection, not necessarily to show how certain structures should be modeled. The RISAConnection solutions for each of these problems are compared to these AISC examples.

The data for each of these verification problems is provided. The RISAConnection example file is called ***Verification Problems.rcn***. This file is located in the **C:\RISA User Data\%username%\Model Files\Examples** directory.

Verification Version

This document contains problems that have been verified in RISAConnection version 16.

Connections 1.1A & 1.1B- Slip Critical Bolts

Slip-Critical Connection Bolt Capacities

This problem was adapted from example J4.A and example J4.B in the *AISC Design Examples version 14.1 (February 2013 revision).* The slip critical bolt capacities in RISAConnection are compared to those from the published example.

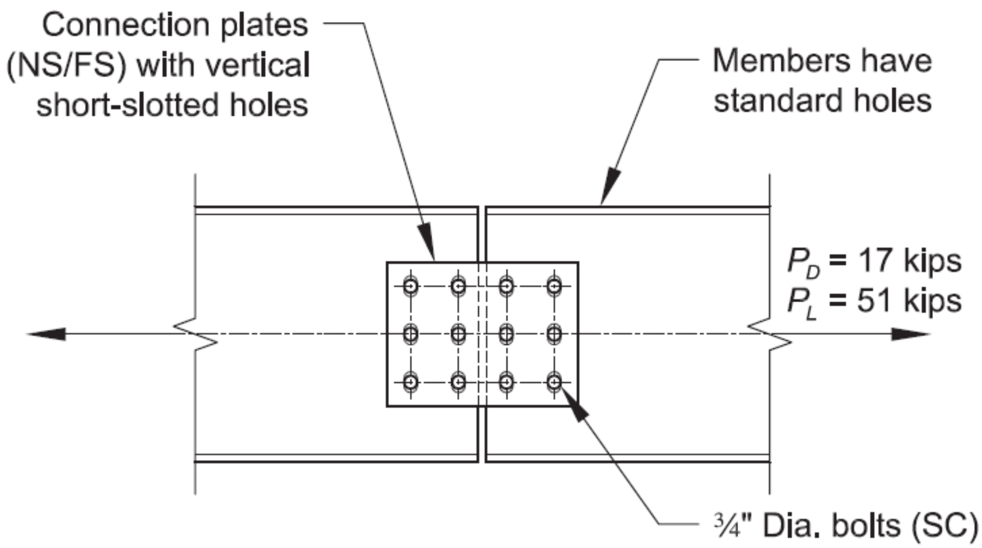


Figure 1.1A – AISC Design Example J.4A Information

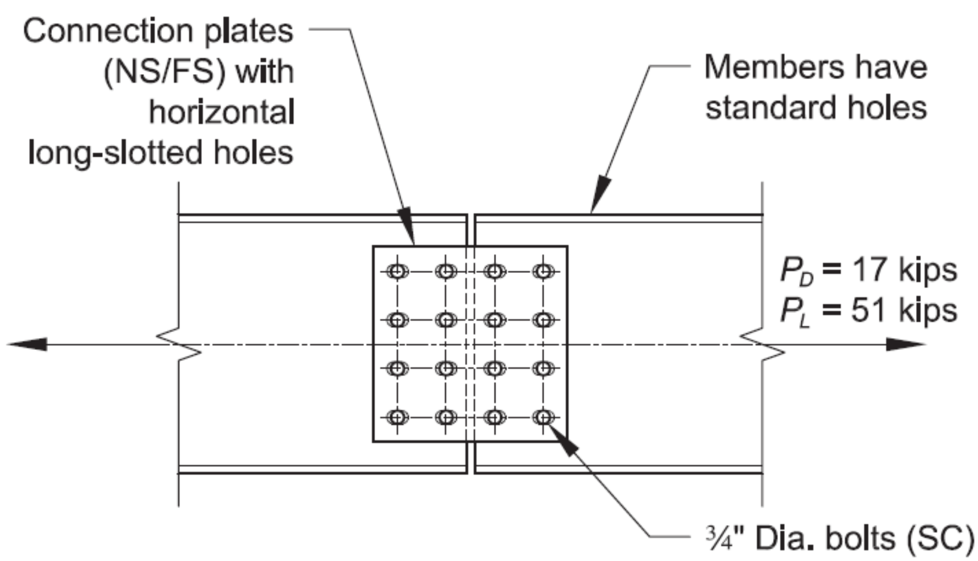


Figure 1.1B - AISC Design Example J.4B Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Bolt Slip Critical – J.4A	74.53/6 = 12.42 kips per bolt[1,2]	12.7 kips per bolt	2.2
Bolt Slip Critical – J.4B	69.66/8 = 8.71 kips per bolt[1,2]	8.88 kips per bolt	1.91

Table 1.1 – Capacity Comparison

[1] In the AISC example, the values are given on a per bolt basis.

[2]RISAConnection applies the bolt group eccentricity to the slip critical capacity. This value is 0.98.

Conclusion

In this example it is shown that the RISAConnection calculations nearly match the design examples. The only minor differences are due to bolt group eccentricity application.

Connection 1.2- Bolted Double Angle Shear

Beam/Column Bolted Double-Angle Connection

This problem was adapted from example II.A-1 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity of an all bolted double angle connection in RISAConnection is compared to that from the published example.

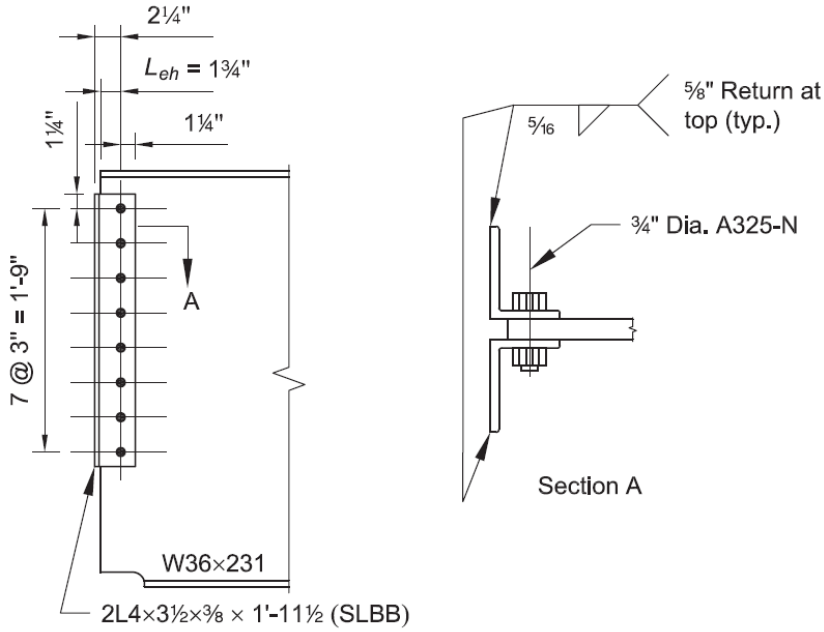


Figure 1.2- AISC Design Example II.A-1 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Min Value[1]	164.91	165	0.05
Bolt Bearing on Beam	190.85[2]	356	N/A
Bolt Bearing on Column	190.85[2]	665	N/A

Table 1.2a – Capacity Comparison

[1]Note that the **Min Value** here is the minimum limit state of clip angle bearing, clip angle shear yielding, clip angle shear rupture and bolt shear. See the Table 1.2b below for this value.

[2]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Shear at Beam	190.85
Clip Angle Shear Yield	211.5
Clip Angle Shear Rupture at Beam	179.44
Clip Angle Shear Rupture at Column	179.44
Clip Angle Block Shear at Beam	164.91
Clip Angle Block Shear at Column	168.31
Bolt Bearing on Clip Angle at Beam	185.35[1]
Bolt Bearing on Clip Angle at Column	185.35[1]
Min Value	**164.91**

Table 1.2b – Capacities from RISAConnection

[1]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Conclusion

In this example many of the calculations are very similar between RISAConnection and the AISC example. There is one major difference due to the AISC 360-10 Section J3.10 user note.

Connection 1.3- Bolt/Weld Double Angle Shear

Beam/Column Bolted/Welded Double-Angle Connection

This problem was adapted from example II.A-2 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity of a double angle, bolted at the beam web and welded to the supporting column, in RISAConnection is compared to that from the published example.

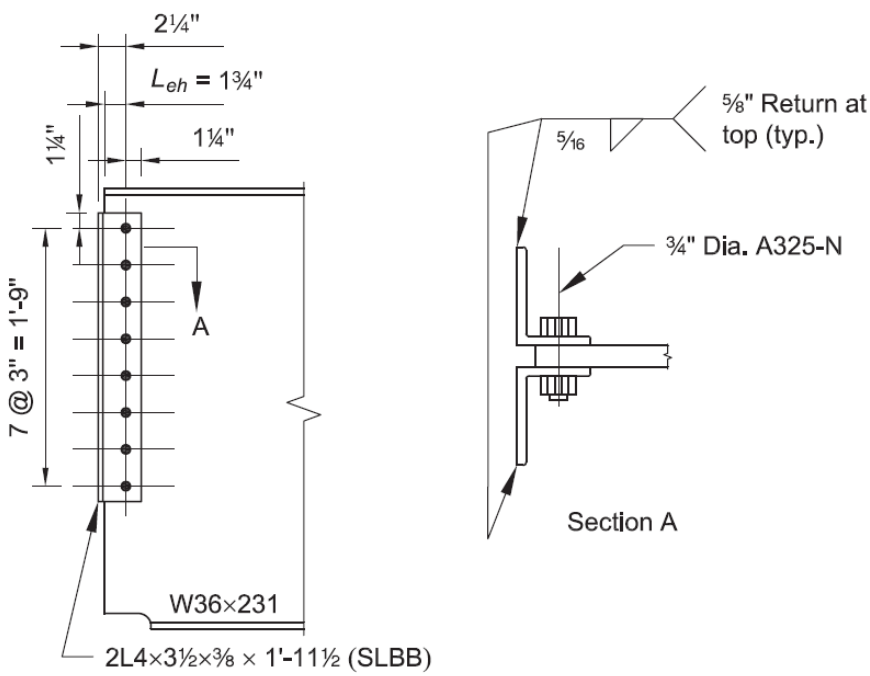


Figure 1.3 - AISC Design Example II.A-2 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Column Weld Strength	181.09[1]	186	2.68
Min Value	189.02[2]	191	1.04
Bolt Bearing on Beam	190.85[3]	356	N/A

Table 1.3a – Capacity Comparison

[1]The difference here is based on the eccentricity "e". The AISC uses the width of the leg of the connection angle. This is found on P10-11 of the AISC 14th edition manual. RISA is conservatively using the width of the leg of connection angle + beam web width/2.

[2]Note that the **Min Value** here is the minimum limit state of clip angle bearing, clip angle shear yielding, clip angle shear rupture and bolt shear. See the Table 1.3b below for this value and reasoning for the differences.

[3]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Shear at Beam	190.85
Clip Angle Shear Yield	253.80
Clip Angle Shear Rupture at Beam	215.32
Clip Angle Shear Rupture at Column	306.67
Clip Angle Block Shear at Beam	197.90
Bolt Bearing on Clip Angle at Beam	189.02[1]
Min Value	**189.02**

Table 1.3b – Capacities from RISAConnection

[1]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Conclusion

In this example many of the calculations are very similar between RISAConnection and the AISC example. There is one major difference due to the AISC 360-10 Section J3.10 user note.

Connection 1.4-Welded Double Angle Shear

Beam/Column Welded Double-Angle Connection

This problem was adapted from example II.A-3 in the *AISC Design Examples version 14.1 (February 2013 revision).* The welded shear capacity in RISAConnection is compared to that from the published example. The beam weld results are also compared to the instantaneous center of rotation method and the column weld capacity doesn't change between IC and the Elastic method.

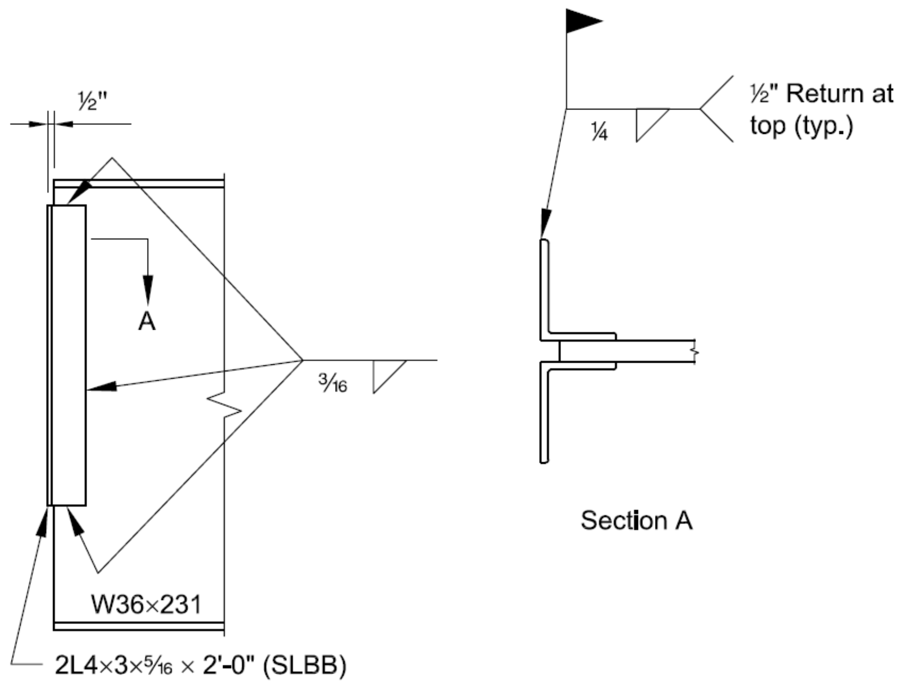


Figure 1.4 - AISC Design Example II.A-3 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Beam Weld Strength	166.59[1]	171	2.6
Column Weld Strength	148.91[1]	153	2.6
Clip Angle Shear Yield	216.00	216	0.00

Table 1.4 – Capacity Comparison

[1]The difference here is based on the eccentricity "e". The AISC uses the width of the leg of the connection angle. However, RISA is conservatively using the width of the leg of connection angle + beam web width/2.

Conclusion

In this example it is shown that the RISAConnection calculations for the most part match the design examples. The differences are due to small differences in the assumptions made.

Connection 1.5- Bolted Coped Beam Shear

Beam/Girder Bolted Double Angle Connection on Coped Beam

This problem was adapted from example II.A-4 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity in RISAConnection is compared to that from the published example.

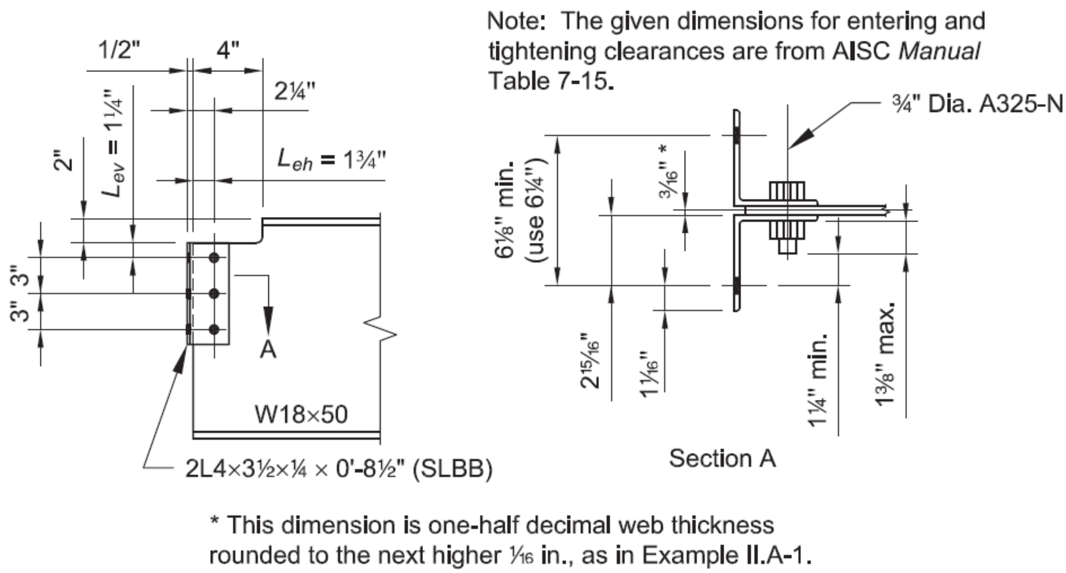


Figure 1.5 – AISC Design Example II.A-4 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Min Value[1]	51.11	50.9	0.41
Min (Bolt Bearing on Beam, Beam Block Shear)	(53.22, 50.19[2]) = 50.19	47.2	6.34
Bolt Bearing on Girder	71.57[3]	140	N/A
Coped Beam Local Web Buckling	155.61	156	0.25
Beam Shear Yield	113.60	113	0.50
Beam Shear Rupture	92.59	92.5	0.10

Table 1.5a – Capacity Comparison

[1]Note that the **Min Value** here is the minimum limit state of clip angle bearing, clip angle shear yielding, clip angle shear rupture, block shear rupture on the angles, and bolt shear. See Table 1.5b below for this value and reasoning for the differences.

[2]AISC Table 10-1 automatically subtracts ¼" from the L_{eh} value when calculating block shear capacity, per a discussion on P10-9. If the L_{eh} value is set to 1.5" instead of 1.75" then this value matches identically.

[3]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Bearing on Clip Angle at Beam	62.39[1]
Bolt Bearing on Clip Angle at Girder	62.39[1]
Clip Angle Shear Yield	61.20
Clip Angle Shear Rupture at Beam	51.11
Clip Angle Shear Rupture at Girder	51.11
Clip Angle Block Shear at Beam	58.18
Clip Angle Block Shear at Girder	58.18
Bolt Shear at Beam	71.57
Bolt Shear at Girder	71.57
Min Value	**51.11**

Table 1.5b – Capacities from RISAConnection

[1]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Conclusion

In this example many of the calculations are very similar between RISAConnection and the AISC example. There is one major difference due to the AISC 360-10 Section J3.10 user note.

Connection 1.6- Welded Coped Beam Shear

Beam/Girder Welded/Bolted Double-Angle Connection on a Coped Beam

This problem was adapted from example II.A-5 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity of bolts and welds in RISAConnection are compared to those from the published example.

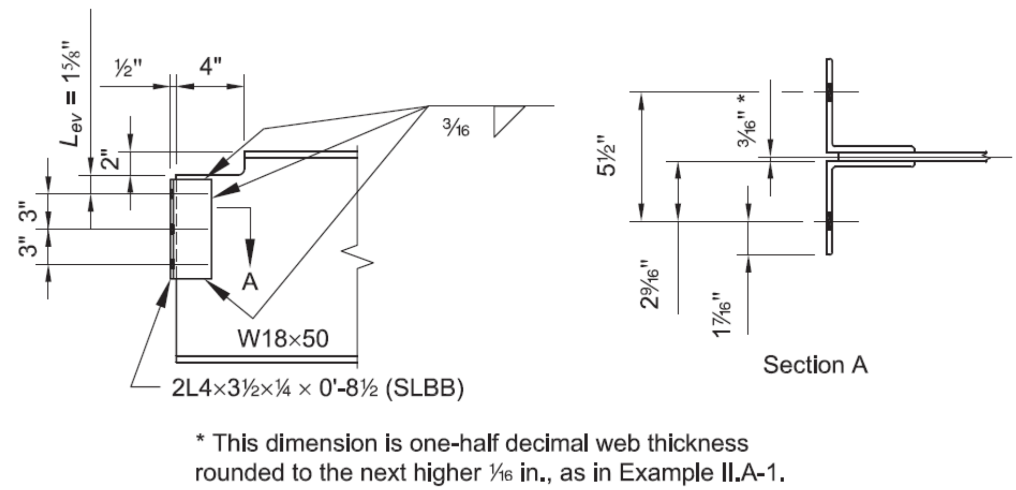


Figure 1.6 – AISC Design Example II.A-5 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Beam Weld Strength	75.77	73.5	3.09
Bolt Bearing on Girder	71.57[2]	140	N/A
Min Value[1]	51.11	50.9	0.41
Beam Shear Yield	113.60	113	0.53
Beam Shear Rupture	110.76	111	0.22

Table 1.6a – Capacity Comparison

[1]Note that the **Min Value** here is the minimum limit state of bearing, shear and block shear for bolts and angles. See Table 1.6b below for these values.

[2]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Bearing on Clip Angle at Girder	62.39[1]
Clip Angle Block Shear at Girder	58.18
Clip Angle Shear Rupture at Beam	73.95
Clip Angle Shear Rupture at Girder	51.11
Clip Angle Shear Yield	61.20
Bolt Shear at Girder	71.57
Min Value	**51.11**

Table 1.6b – Capacities from RISAConnection

[1]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Conclusion

In this example many of the calculations are very similar between RISAConnection and the AISC example. There is one major difference due to the AISC 360-10 Section J3.10 user note.

Connection 1.7- Coped Top Flange Shear

Beam/girder Beam End Coped at the Top Flange Only

This problem was adapted from example II.A-6 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity in RISAConnection is compared to that from parts A and B of the published example.

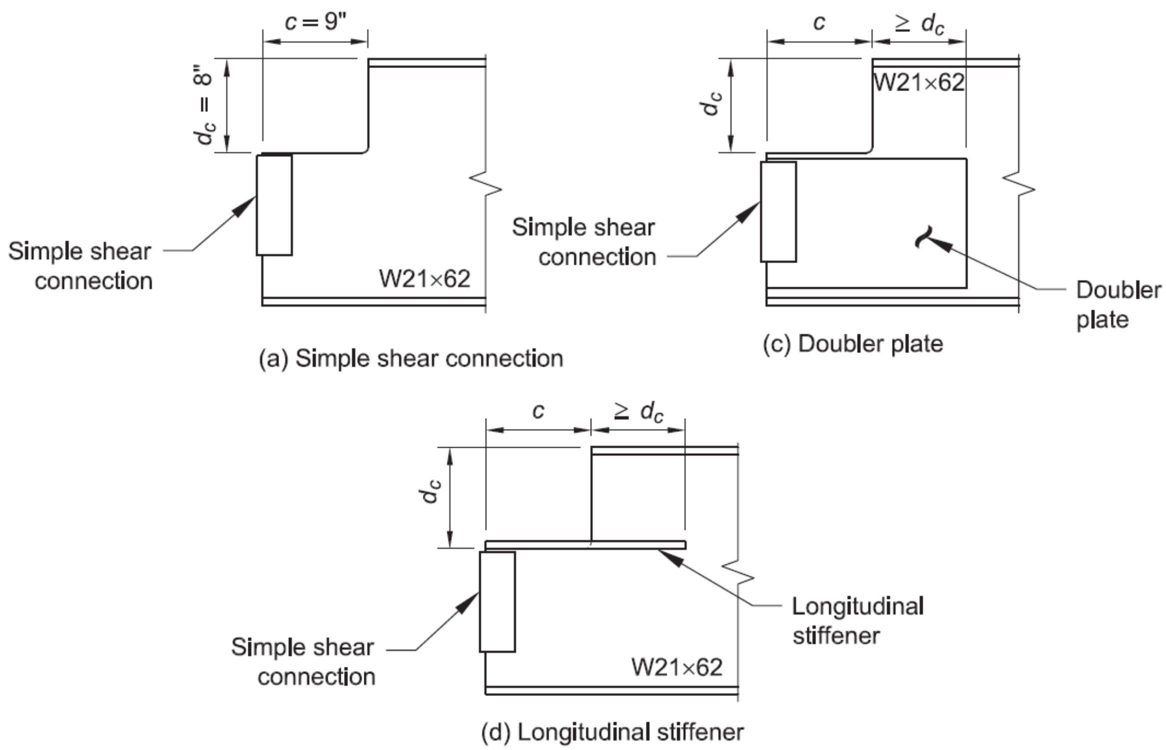


Figure 1.7 – AISC Design Example II.A-6 Information

Calculation and Comparison

Part A. Capacity Comparison of **W21x62** (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Coped Beam Local Web Buckling	56.29	56.1	0.34
Beam Shear Yield	104.00	104	0.00
Beam Shear Rupture	101.40	102	0.59

Table 1.7a – Capacity Comparison

Part B. Calculation Comparison of **W21x73** (All Results Shown in in^3)			
Limit State	RISAConnection	AISC Example	% Difference
S_{net}, elastic modulus of net section	20.91[1]	21	0.43

Table 1.7b – Calculation Comparison

[1]You must change the section from a W21x62 to a W21x73 and look at the **Coped Beam Local Web Buckling** section to confirm the S_{net} value.

Conclusion

In this example it is shown that the RISAConnection calculations match the design examples. The only minor differences are due to rounding.

Connection 1.8- Coped Lat. Torsional Buckling

Beam/Girder Beam End Coped at the Top and Bottom Flanges

This problem was adapted from example II.A-7 in the *AISC Design Examples version 14.1 (February 2013 revision).* The Lateral Torsional Buckling limit state capacity in RISAConnection is compared to that from of the published example.

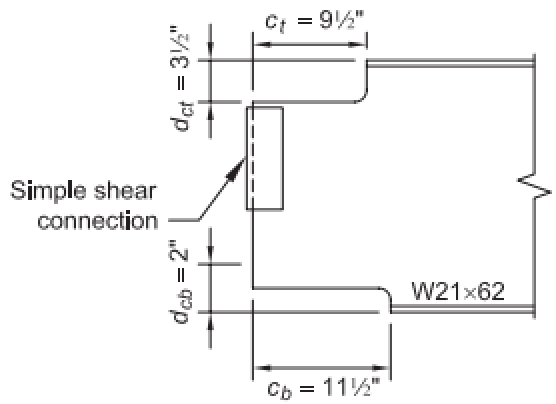


Figure 1.8 – AISC Design Example II.A-7 Information

Calculation and Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Coped Beam Lateral Torsional Buckling	12.52	15.5[1]	19.22

Table 1.8 – Capacity Comparison

[1]The AISC example properly assumes the top of the coped beam is in compression and the bottom in tension. Thus, they provide a local buckling check for the top of the coped beam and a flexural yielding check for the bottom of the coped beam. In RISAConnection we do not know the direction of the shear force and do not know whether the extreme fibers are in compression or tension. Because of this we use the worst case cope (11 ½" in this case) and provide both a buckling and a yielding check for the connection. Because the worst case in RISAConnection was for local buckling of the bottom of the coped beam the program is being conservative compared to the design example.

Conclusion

In this example it is shown that the RISAConnection calculations are conservative compared to the design examples.

Connection 1.9- Bolted Shear End Plate Shear

Beam/Girder Bolted End-Plate Connection

This problem was adapted from example II.A-11 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity of a beam to girder end plate shear connection in RISAConnection is compared to that from of the published example.

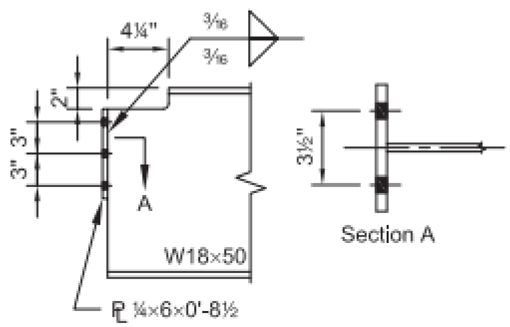


Figure 1.9 – AISC Design Example II.A-11 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Min Value-1[1]	50.93	50.9	0.06
Min Value-2[2]	45.24	45.2	0.09
Bolt Bearing on Girder	71.57[3]	140	N/A

Table 1.9a – Capacity Comparison

[1]Note that the **Min Value-1** here is the minimum limit state of bolt shear, bolt bearing, shear yielding, shear rupture, and block shear rupture of end-plate. See Table 1.9b below for these values.

[2]Note that the **Min Value-2** here is the minimum limit state of weld shear and beam web shear rupture. See Table 1.9c below for these values.

[3]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Shear at Girder	71.57
Bolt Bearing on Girder	71.57[3]
Bolt Bearing on Plate at Girder	62.39[3]
Beam Shear Yield	113.60
Plate Shear Yield	61.20
Beam Shear Rupture	110.76
Plate Shear Rupture at Girder	51.11
Plate Block Shear at Girder	50.93
Min Value	**50.93**

Table 1.9b – Capacities from RISAConnection

[3]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Beam Weld Strength	45.24
Beam Shear Rupture	110.76
Min Value	**45.24**

Table 1.9c – Capacities from RISAConnection

Conclusion

In this example many of the calculations are very similar between RISAConnection and the AISC example. There is one major difference due to the AISC 360-10 Section J3.10 user note.

Connection 1.10- Bolted Shear Tab Shear

Beam/Column Bolted Shear Tab Connection

This problem was adapted from example II.A-17 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity in RISAConnection is compared to that from of the published example.

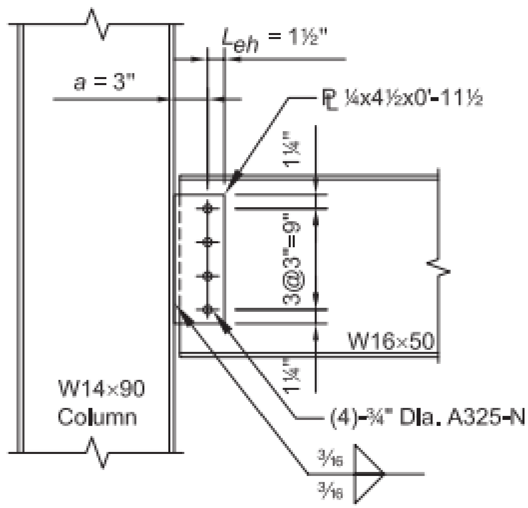


Figure 1.10 – AISC Design Example II.A-17 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Min Value[1]	34.8	34.8	0.00
Bolt Bearing on Beam	47.71[2]	88.9	N/A

Table 1.10a – Capacity Comparison

[1]Note that the **Min Value** here is the minimum limit state of bolt shear, weld shear, bolt bearing, shear yielding, shear rupture, and block shear rupture of the plate. See Table 1.10b below for these values.

[2]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Shear at Beam	42.42[1]
Column Weld Strength	56.28
Bolt Bearing on Plate at Beam	43.13[2]
Beam Shear Yield	123.88
Plate Shear Yield	41.40
Beam Shear Rupture	94.85
Plate Shear Rupture at Beam	34.80
Plate Block Shear at Beam	35.38
Min Value	**34.80**

Table 1.10b – Capacities from RISAConnection

[1]The program is using a Bolt Group Eccentricity that is conservative to what the manual gives. The manual allows you to ignore the eccentricity for certain bolt configurations. In the program you will see that the bolt group eccentricity coefficient is listed as 0.89. If you take 42.42/0.89 = 47.66 kips.

[2]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Conclusion

In this example many of the calculations are very similar between RISAConnection and the AISC example. There is one major difference due to the AISC 360-10 Section J3.10 user note.

Connection 1.11- Beam to Girder Shear Tab

Beam/Girder Bolted Shear Plate Connection

This problem was adapted from example II.A-18 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity in RISAConnection is compared to that from of the published example.

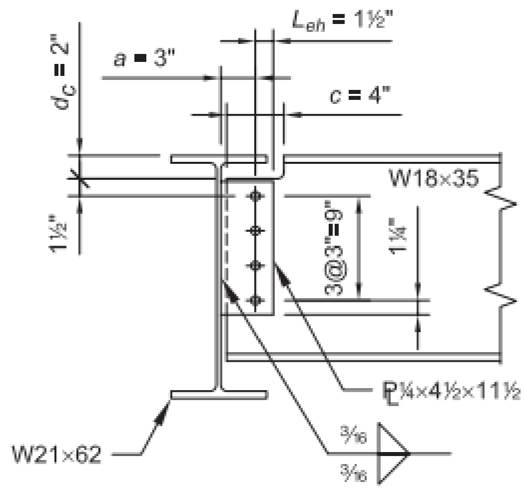


Figure 1.11 – AISC Design Example II.A-18 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Min Value-1[1]	34.8	34.8	0.00
Min Value-2[2]	47.71	54.0	N/A

Table 1.11a – Capacity Comparison

[1]Note that the **Min Value-1** here is the minimum limit state of bolt shear, weld shear, bolt bearing, shear yielding, shear rupture, and block shear rupture of the plate. See Table 1.11b below for these values.

[2]Note that the **Min Value-2** here is the minimum limit state of bolt bearing and block shear rupture for beam web. See Table 1.11c below for these values.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Shear at Beam	42.42[1]
Girder Weld Strength	56.28[2]
Bolt Bearing on Beam	47.71[3]
Bolt Bearing on Plate at Beam	43.13[3]
Beam Shear Yield	94.20
Plate Shear Yield	41.40
Beam Shear Rupture	71.37
Plate Shear Rupture at Beam	34.80
Plate Block Shear at Beam	35.38
Min Value	**34.8**

Table 1.11b – Capacities from RISAConnection

[1]The program is using a Bolt Group Eccentricity that is conservative to what the manual gives. The manual allows you to ignore the eccentricity for certain bolt configurations. In the program you will see that the bolt group eccentricity coefficient is listed as 0.89. If you take 42.42/0.89 = 47.66 kips.

[2]The AISC 14th edition Table 10-4 assumes an effective weld length equal to the end plate minus twice the weld size. RISAConnection uses the full length of the plate. This value does not control here but if you subtract 2*(3/16”) = 3/8” from the weld length then it would match AISC assumptions.

[3]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Bearing on Beam	47.71[1]
Beam Block Shear	63.62[2]
Min Value	**47.71**

Table 1.11c – Capacities from RISAConnection

[1]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

[2]For any L_{eh} greater than 1.75, the AISC conservatively uses the value for 1.75 (including a ¼” decrease due to possible beam over-run). In this problem the L_{eh} = 2.5”, so our calculations are based on this 2.5”, thus our Beam Block Shear capacity is much higher.

Conclusion

In this example many of the calculations are very similar between RISAConnection and the AISC example. There is one major difference due to the AISC 360-10 Section J3.10 user note.

Connection 1.12- Extended Shear Tab

Beam/Column Web Extended Shear Tab Connection

This problem was adapted from example II.A-19 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity in RISAConnection is compared to that from of the published example.

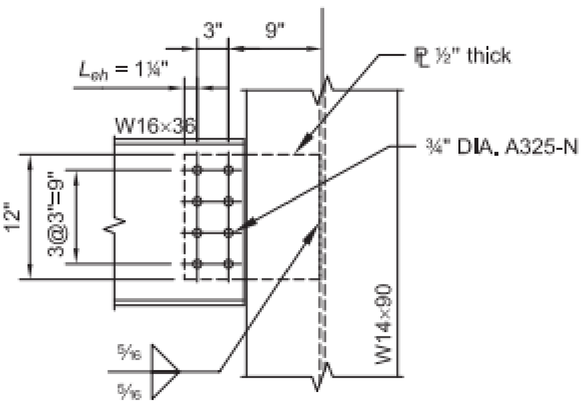


Figure 1.12 – AISC Design Example II.A-19 Information

Comparison

Capacity Comparison (All Results Shown in kips unless noted otherwise)			
Limit State	RISAConnection	AISC Example	% Difference
Bolt Bearing on Beam (single bolt)	95.43[2]/8 =11.93	17.3	N/A
Bolt Shear at Beam (Strength of the Bolt Group)	27.64	27.7	0.22
Bolt Bearing on Plate at Beam (single bolt)	38.06/2 = 19.03[1]	19.0	0.16
Plate Shear Yield	86.40	86.4	0.00
Plate Shear Rupture at Beam	73.95	74.0	0.07
Plate Block Shear	78.00	77.8	0.26
Plate Flexural Yielding	388.02 k-in	388 k-in	0.01
Plate Flexural Rupture	369.75 k-in	371 k-in	0.34

Table 1.12 – Capacity Comparison

[1]The AISC example takes the worst-case capacity between edge tear-out and bearing and compares a single bolt demand to that value. In the RISAConnection output the $R_{n\text{-edge-tearout capacity}}$ controls between these values and is reported as 38.06 kips.

[2]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Conclusion

In this example many of the calculations are very similar between RISAConnection and the AISC example. There is one major difference due to the AISC 360-10 Section J3.10 user note.

Connection 1.13- Bolted Beam Shear Splice

Bolted Single-Plate Beam Shear Splice

This problem was adapted from example II.A-20 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity in RISAConnection is compared to that from of the published example.

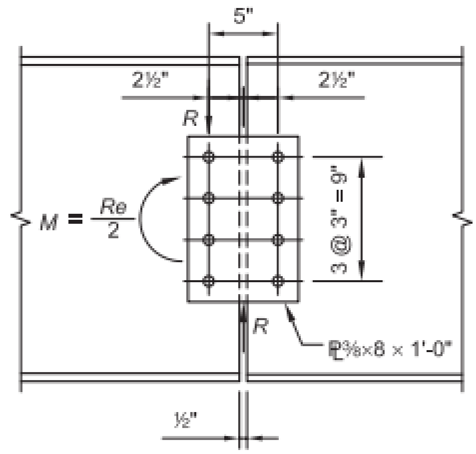


Figure 1.13 – AISC Design Example II.A-20 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Bolt Shear (single bolt)	49.76[1]/4 = 12.44	16.2	23.21
Bolt Bearing on Plate	26.92/2 = 13.46[2]	13.5	0.29
Plate Flexural Yielding	291.02 kip-in	291 kip-in	0.01
Plate Flexural Rupture	261.00 kip-in	261 kip-in	0.00
Plate Shear Yielding	64.80	64.8	0.00
Plate Shear Rupture	52.20	52.2	0.00
Plate Block Shear	53.40	53.3	0.19

Table 1.13 – Capacity Comparison

[1]In RISAConnection the connection eccentricity is accounted for on the bolt group. This does not occur in the AISC design example. From the RISAConnection output we can see that C = 0.77, which accounts for the 23% difference.

[2]The AISC example takes the worst-case capacity between edge tear-out and bearing and compares a single bolt demand to that value. In the RISAConnection output, the controlling capacity is the $R_{n\text{-edge-tearout capacity}}$, listed as 26.92 kips.

Conclusion

In this example it is shown that the RISAConnection calculations match the design examples. The only minor differences are due to rounding.

Connection 1.14- Bolt/Weld Beam Shear Splice

Bolted/Welded Single-Plate Beam Shear Splice

This problem was adapted from example II.A-21 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity in RISAConnection is compared to that from of the published example.

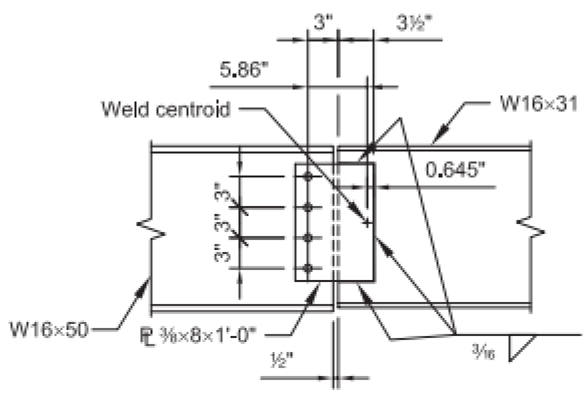


Figure 1.14 – AISC Design Example II.A-21 Information

Comparison

Capacity Comparison (All Results Shown in kips)			
Limit State	RISAConnection	AISC Example	% Difference
Bolt Shear (single bolt)	21.06[1]/4 = 5.265	11.9	55.76
Bolt Bearing on Plate (single bolt)	28.55/2 = 14.275[2]	(37/29.4)*11.0 = 13.875[3]	2.88
Plate Flexural Yielding	291.02 kip-in	291 kip-in	0.01
Plate Shear Yielding	64.8	64.8	0.00
Plate Shear Rupture	55.46	55.5	0.07
Plate Block Shear	54.08	53.9	0.33

Table 1.14 – Capacity Comparison

[1]In RISAConnection the connection eccentricity is accounted for on the bolt group. This does not occur in the AISC design example. From the RISAConnection output we can see that C = 0.44, which accounts for the 56% difference.

[2]The AISC example takes the worst-case capacity between edge tear-out and bearing and compares a single bolt demand to that value. In the RISAConnection output, the controlling capacity is the $R_{n\text{-edge-tearout capacity}}$, listed as 28.55 kips.

[3]The AISC example uses a conservative L_e = 1.25". The actual L_e = 1.5" is the one used in the program. In the table above the AISC value has been factored up by a ratio of 37/29.4. The 37 kips/in is the interpolated value from Table 7-5 of the AISC 14th edition manual.

Conclusion

In this example it is shown that the RISAConnection calculations match the design examples closely. There are minor differences due to rounding and a difference due to the L_e value.

Connection 1.15- Eccentrically Loaded Bolts (IC Method)

Eccentrically Loaded Bolt Group (IC Method)

This problem was adapted from example II.A-24 in the *AISC Design Examples version 14.1 (February 2013 revision).* The eccentric force (per the Instantaneous Center of Rotation method) and shear capacity in RISAConnection are compared to that from of part A of the published example.

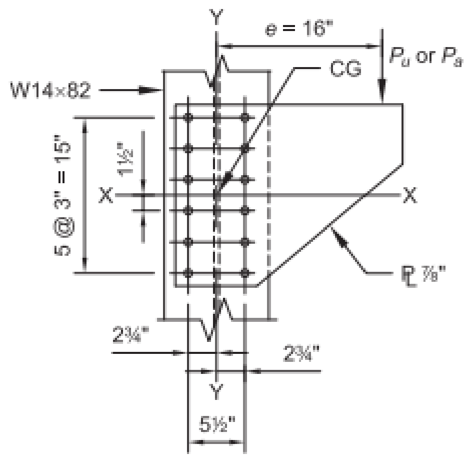


Figure 1.15 – AISC Design Example II.A-24 Information

Comparison (Solution A: θ = 0°)

Capacity Comparison (All Results Shown in kips)			
Limit State	RISAConnection	AISC Example	% Difference
Bolt Shear at Beam	57.69	57.5	0.10
Eccentric Coefficient[1]	3.5535	3.55	0.10

Table 1.15 – Capacity Comparison

[1]Note that in RISAConnection the eccentricity coefficient is obtained from multiplying the eccentricity coefficient (C=0.2961) by the number of bolts (N_{bolt} = 12).

Conclusion

In this example it is shown that the RISAConnection calculations match the design examples closely.

Connection 1.16- Eccentrically Loaded Bolts (Elastic Method)

Eccentrically Loaded Bolt Group (Elastic Method)

This problem was adapted from example II.A-25 in the *AISC Design Examples version 14.1 (February 2013 revision).* The eccentric force (per the Elastic method) and shear capacity in RISAConnection are compared to that from the published example.

Note: To match the results below, go to the **(Global) Project Settings– Solution** tab and choose **Elastic** for the "**Bolt Group Analysis Method**".

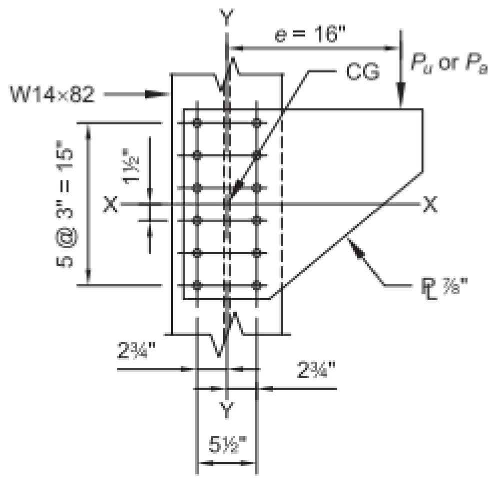


Figure 1.16 – AISC Design Example II.A-25 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Bolt Shear	46.06	45.9	0.35
Eccentricity Coefficient[1]	2.837	1/0.353 = 2.833[2]	0.14

Table 1.16 – Capacity Comparison

[1]Note that in RISAConnection the eccentricity coefficient is obtained from multiplying the eccentricity coefficient (C=0.2364) by the number of bolts (N_{bolt}=12).

[2]The values between RISAConnection and the AISC examples are the inverse of one another. The eccentricity coefficient listed here from RISAConnection is equal to C multiplied by the number of bolts.

Conclusion

In this example it is shown that the RISAConnection calculations match the design examples closely.

Connection 1.17- Bolted Single Angle Shear

Beam/Girder Bolted Single-Angle Connection

This problem was adapted from example II.A-28 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity in RISAConnection is compared to that from the published example.

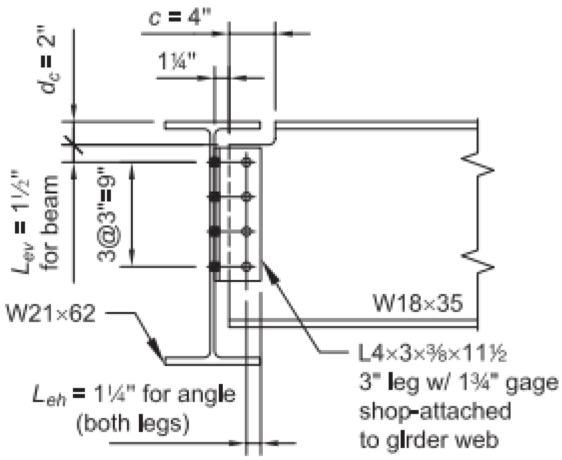


Figure 1.17 – AISC Design Example II.A-28 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Bolt Shear at Girder (single bolt)	40.19/4 = 10.05[1]	11.9	15.5
Bolt Shear at Beam (single bolt)	35.05/4 = 8.76[2]	11.9	26.4
Bolt Bearing on Clip Angle (single bolt)	22.02/2 = 11[3]	11.0	0.00
Clip Angle Shear Yield	62.10	62.1	0.00
Clip Angle Shear Rupture	52.20	52.0	0.38
Clip Angle Block Shear	50.35	50.5	0.29
Flexural Yielding/Rupture of Support-Leg of Angle	NA[4]	267 kip-in/ 245 kip-in	NA
Min Value-1[5]	45.66[7]	51.3	11.0

Table 1.17a – Capacity Comparison

[1]The program is using a Bolt Group Eccentricity that is conservative to what the manual gives. The manual allows you to ignore the eccentricity for certain bolt configurations. In the program you will see that the bolt group eccentricity coefficient is listed as 0.84. If you take 10.05/0.84 = 11.96 kips which nearly matches the AISC example.

[2]The program is using a Bolt Group Eccentricity that is conservative to what the manual gives. The manual allows you to ignore the eccentricity for certain bolt configurations. In the program you will see that the bolt group eccentricity coefficient is listed as 0.73. If you take 8.76/0.73 = 12.0 kips which nearly matches the AISC example.

[3]The AISC example takes the worst-case capacity between edge tear-out and bearing and compares a single bolt demand to that value. In the RISAConnection output, the controlling capacity is the $R_{n\text{-edge-tearout capacity}}$, listed as 22.02 kips.

[4]Because of the eccentricity of the single angle there is flexure in the leg attached to the column. At this time RISAConnection does not check the single angle for these failure modes.

[5]The **Min Value** is the minimum limit state of bolt bearing on beam and beam block shear rupture. See Table 1.17b below for these values.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Bearing on Beam	45.66[7]
Beam Block Shear	52.41[6]
Min Value	**45.66**

Table 1.17b – Capacities from RISAConnection

[6]The AISC example uses a conservative L_{eh} = 1.25”. The actual L_{eh} = 1.5” is the one used in the program. This is why the RISAConnection value is slightly larger than the example

[7]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Conclusion

In this example it is shown that the RISAConnection calculations for the most part match the design examples. There are minor differences are due to rounding, eccentricity assumptions and a difference in the L_{eh} value used. There is also a difference based on the AISC 360-15 Section J3.10 user note.

Connection 1.18- Bolted/Welded Single Angle

Beam/Column Bolted/Welded Single Angle Connection

This problem was adapted from example II.A-29 in the *AISC Design Examples version 14.1 (February 2013 revision).* The shear capacity in RISAConnection is compared to that from the published example.

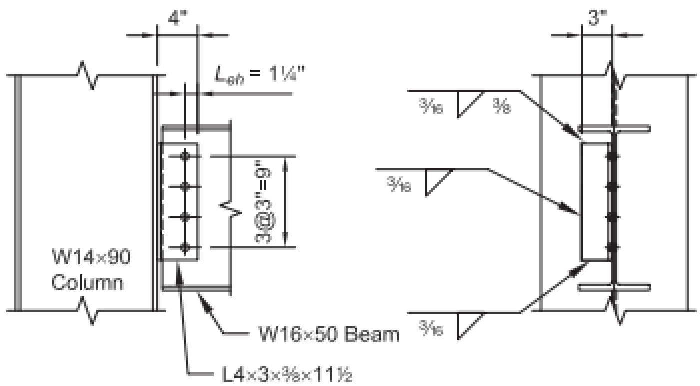


Figure 1.18 – AISC Design Example II.A-29 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Min Value[1]	35.05	47.6	26.4
Column Weld Strength	37.93	37.8	0.34
Bolt Bearing on Beam	47.71[2]	88.9	N/A

Table 1.18a – Capacity Comparison

[1]Note that the **Min Value** here is the minimum limit state of, shear yielding, shear rupture, bolt bearing, block shear and bolt shear. See Table 1.18b below for these values.

[2]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Clip Angle Shear Yield	62.10
Clip Angle Shear Rupture at Beam	52.20
Clip Angle Shear Rupture at Column	75.04
Bolt Bearing on Clip Angle at Beam	46.80[1]
Clip Angle Block Shear at Beam	50.35
Bolt Shear	35.05[2]
Min Value	**35.05**

Table 1.18b – Capacities from RISAConnection

[1]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

[2]The program is using a Bolt Group Eccentricity that is conservative to what the manual gives. The manual allows you to ignore the eccentricity for certain bolt configurations. In the program you will see that the bolt group eccentricity coefficient is listed as 0.73. If you take 35.05/0.73 = 48.01 kips which nearly matches the AISC example.

Conclusion

In this example it is shown that the RISAConnection calculations mostly match the design examples. The only minor differences are due to rounding and eccentricity assumptions, as well as the AISC 360-15 Section J3.10 user note.

Connection 2.1- Bolted Flange Plate Moment

Beam/Column Flange Bolted Flange Plate Moment Connection

This problem was adapted from example II.B-1 in the *AISC Design Examples version 14.1 (February 2013 revision).* The capacity values in RISAConnection are compared to those from the published example.

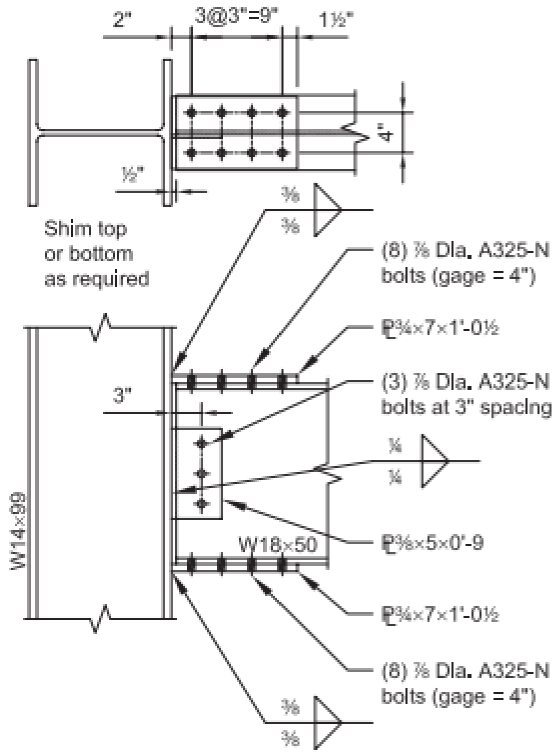


Figure2.1- AISC Design Example II.B-1 Information

Comparison

Capacity Comparison (All Results Shown in Kips unless otherwise noted)			
Limit State	RISAConnection	AISC Example	% Difference
Beam Flexural Rupture/Strength	2537.15 kip-in	2532 kip-in	0.20
Bolt Shear at Beam Web (single bolt)	48.71/3 = 16.23	16.2	0.19
Bolt Bearing at Vert. Plate at Top and Bottom Bolts	26.92/2 = 13.46[1]	13.4	0.45
Bolt Bearing Strength at Vert. Plate at Middle Bolt	45.68/2 = 22.84[1]	22.8	0.18
Vert. Plate Shear Yielding	48.6	48.6	0.00
Vert. Plate Shear Rupture	39.15	39.2	0.13
Plate Block Shear at Beam	46.69	46.7	0.00
Shear Plate Weld Strength at Column	66.82	66.8	0.00
Bolt Bearing at Beam Flange (single bolt)	32.47/2 = 16.235[2]	17.4	6.18
Bolt Bearing at Flange Plate (single bolt)	32.47/2 = 16.235[2]	20.4 kips per bolt	20.4
Flange Plate Tensile Yield	113.17	113	0.15
Flange Plate Tensile Rupture	108.75	109	0.23
Flange Plate Block Shear	213.6[3]	213	0.28
Beam Flange Block Shear	201.92[4]	197	2.50
Flange Plate Compression	113.17	113	0.15
Column Flange Bending	113.85	114	0.13
Column Web Yielding	135.8[5]	124	9.52
Column Web Crippling	166.01[5]	155	7.10

Table 2.1 – Capacity Comparison

[1]In RISAConnection the bolt bearing strength at vertical plate at top and bottom bolts value comes from the $R_{n\text{-edge-tearout}}/\Omega$ value. The bolt bearing strength at vertical plate at middle bolt value comes from the $R_{n\text{-bearing}}/\Omega$ value.

[2]The AISC example takes the worst-case capacity between edge tear-out and bearing and compares a single bolt demand to that value. In the RISAConnection output, the controlling capacity is the $R_{n\text{-bolt}}$ capacity, listed as 32.47 kips. Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value.

[3]The AISC shows 3 cases and does calculations for Case 1 and Case 3. RISAConnection only considers Case 1 and our value matches the AISC value.

[4]The program uses a conservative Le = 1.25". The actual Le = 1.5" (to center of bolt). RISAConnection uses the actual values and give slightly higher capacities than the AISC.

[5]The difference between RISAConnection and the AISC example is the bearing length, N. The example assumes it is the thickness of the flange plate (0.75"). RISAConnection assumes it is equal

to the thickness of the flange plate + the double fillet leg size (0.75" + 0.375" + 0.375" = 1.5"). If you substitute 1.5" in the AISC equation you will get an identical result.

Conclusion

In this example it is shown that the RISAConnection calculations mostly match the design examples. There are differences due to rounding and many different assumptions made.

Connection 2.2- Direct Weld Moment

Beam/Column Direct Weld Moment Connection

This problem was adapted from example II.B-3 in the *AISC Design Examples version 14.1 (February 2013 revision).* The moment weld capacity values in RISAConnection are compared to those from the published example.

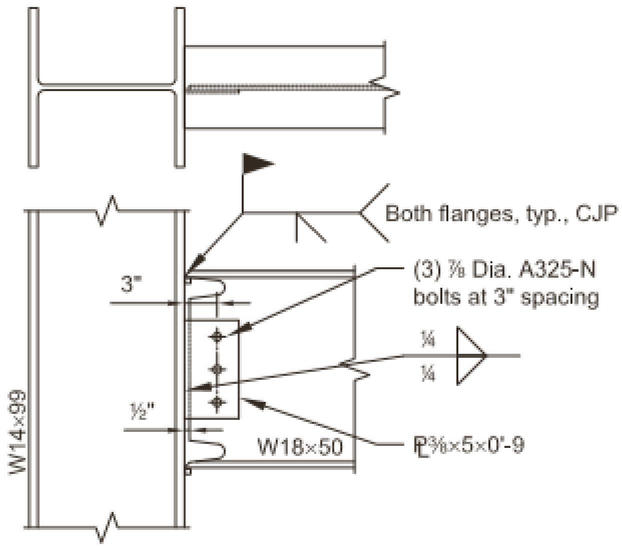


Figure 2.2- AISC Design Example II.B-3

Comparison

Results Comparison		
Limit State	RISAConnection	AISC Example
Beam Web Shear Yield	Pass	Pass (by inspection)
Beam Web Shear Rupture	Pass	Pass (by inspection)
Beam Block Shear	Pass	Pass (by inspection)
Flange Weld Strength	Pass	Pass

Table 2.2 – Capacity Comparison

Conclusion

In this example it is shown that the RISAConnection calculations match the results of the design example.

Connection 2.3- Four Bolt Unstiffened Extended End Plate Moment

Beam/Column Bolted Unsymmetrical Flange Plate Moment Connection

This problem was adapted from example II.B-4 in the *AISC Design Examples version 14.1 (February 2013 revision).* The geometry and capacity values in RISAConnection are compared to those from the published example.

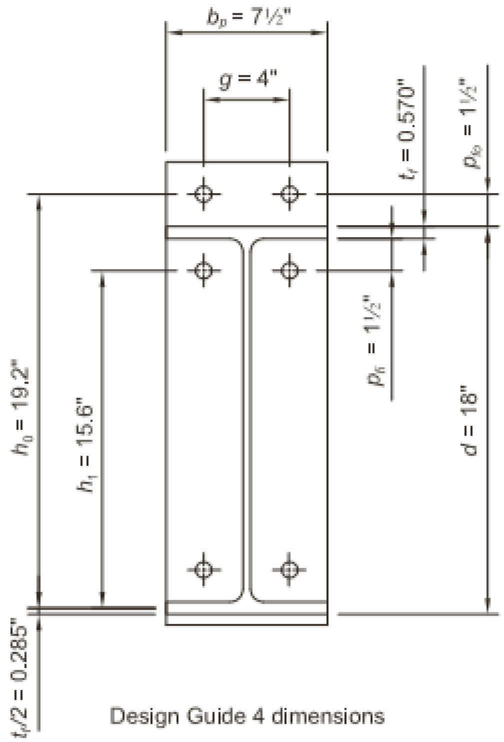


Figure 2.3- AISC Design Example II.B-4 Information

Comparison

Results Comparison			
Limit State/Variable	RISAConnection	AISC Example (solution a)	% Difference
h_0 (per End Plate Flexural Yielding limit state)	19.21 in	19.2 in	0.05
h_1 (per End Plate Flexural Yielding limit state)	15.64 in	15.6 in	0.27
Y_p (per End Plate Flexural Yielding limit state)	139.85 in	140 in	0.11
End Plate Bending Strength (per End Plate Flexural Yielding limit state)	172.97 kips	156 kips[1]	n/a[2]
Required Flange Force (per End Plate Flexural Yielding limit state)	115.66 kips	116 kips	0.29
End Plate Shear Yielding	108.00 kips	97 kips	n/a[3]
End Plate Shear Rupture	91.35 kips	91.4 kips	0.05
Bolt Shear Strength	42.41 kips	42.4 kips	0.02
Bolt Bearing on Plate at Column	42.41 kips	69.6 kips	n/a[4]
Bolt Bearing on Column	42.41 kips	60.8 kips	n/a[5]

Table 2.3 – Capacity and Calculation Comparisons

[1]The example compares the moment (M_{pl}/Ω = 2720 kip-in) whereas RISAConnection converts this to a flange force by dividing by the moment arm between flanges.
[2]The example includes the 1.11 factor in the denominator to account for thin plate bending, however, RISAConnection omits this because it confirms thick plate behavior in the Verify Bolt Prying Assumption check.
[3]The example uses an omega of 1.67, but this is not stated explicitly in the design guide. Therefore RISAConnection uses omega =1.5 per AISC 360-10 section J4.2.
[4]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.
[5]Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

Conclusion

In this example it is shown that the RISAConnection calculations match the results of the design example.

Connection 3.1- Double Angle to HSS Column

Beam/ Rectangular HSS Column Bolted Double Angle

This problem was adapted from example K.3 in the *AISC Design Examples version 14.1 (February 2013 revision).* The bolt and weld capacity values in RISAConnection are compared to those from the published example.

Note: RISAConnection does not allow the user to cope the beam in this example.

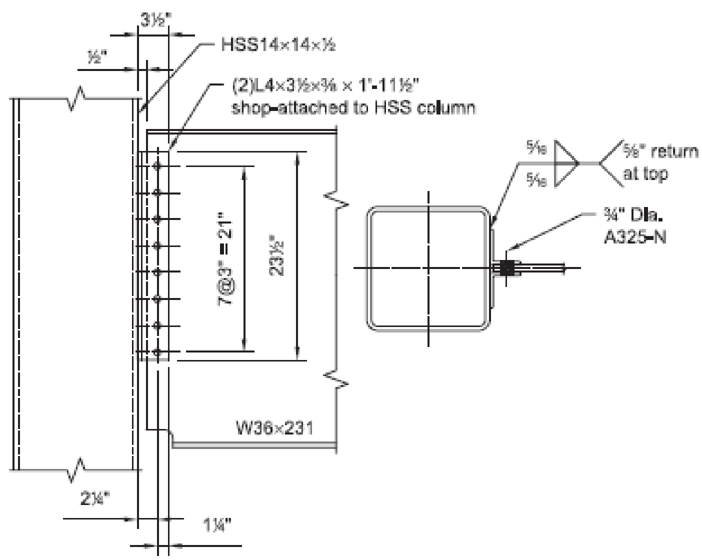


Figure 3.1- AISC Design Example K.3 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Bolt Bearing Strength	190.85	191	0.08
Column Weld Strength	186.14	186	0.08
Beam Web Strength	355.68[1]	356	0.09

Table 3.1 – Capacity Comparisons

[1]RISAConnection compares the bolt bearing on beam to the beam web strength in the AISC example. This is calculated by multiplying the $R_{n\text{-bearing}}$ strength (88.92k) in the Bolt Bearing on Beam limit state by the number of bolts (8) and dividing by Ω (2).

Conclusion

In this example it is shown that the RISAConnection calculations match the values in the design example.

Connection 3.2- Shear Tab to HSS Column

Beam/ Rectangular HSS Column Bolted Shear Tab

This problem was adapted from example K.6 in the *AISC Design Examples version 14.1 (February 2013 revision).* The capacity values in RISAConnection are compared to those from the published example.

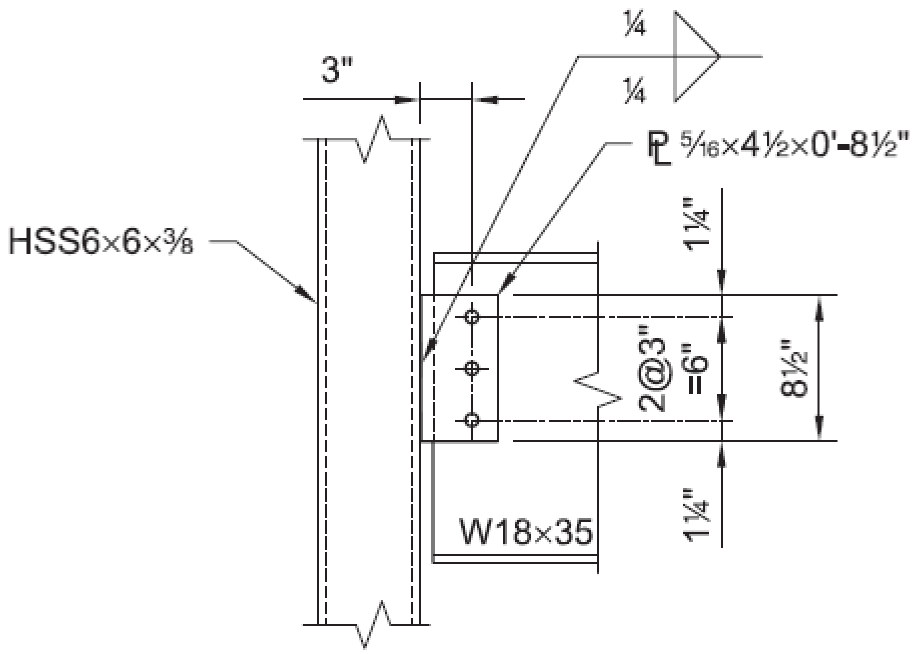


Figure 3.2- AISC Design Example K.6 Information

Comparison

Capacity Comparison			
Limit State	RISAConnection	AISC Example	% Difference
HSS Wall Slenderness[1]	14.19 in	14.2 in	0.08
Material Strength[1]	46 ksi	46 ksi	0.0
Ductility Unity Check[1]	0.793	0.793	0.0
Single-Plate Connection Strength per the Bolt Shear at Beam check	29.60 kips	28.8 kips	2.78
Beam Web Bearing Strength	52.65 kips[2]	52.8 kips	0.28

Table 3.2 – Capacity Comparisons

[1] This limit state can be seen in the HSS Punching Shear limit state.

[2] RISAConnection compares the bolt bearing on beam to the beam web strength in the AISC example. This is calculated by multiplying the $R_{n\text{-bearing}}$ strength (35.1k) in the Bolt Bearing on Beam limit state by the number of bolts (3) and dividing by Ω (2).

Conclusion

In this example it is shown that the RISAConnection calculations match the design example; with exception of the Single-Plate Connection Strength limit state which requires adjustments to the model to match the value.

Connection 3.3- Through-Plate Connection

Beam/ Rectangular HSS Column Through-Plate Connection (Modified)

This problem was adapted from example K.7 in the *AISC Design Examples version 14.1 (February 2013 revision).* The capacity values in RISAConnection are compared to those from the published example.

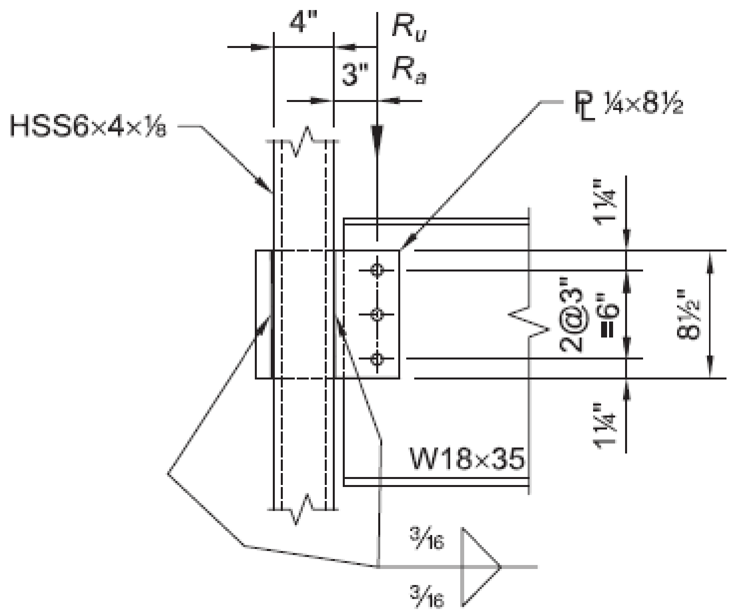


Figure 3.3- AISC Design Example K.7 Information

Comparison

Capacity Comparison			
Limit State	RISAConnection	AISC Example	% Difference
Plate Shear Rupture at Beam	25.56 kips	25.6 kips	0.16
Weld at Column (Near) Unity Check	0.49[1]	0.486	0.0
Required Weld Strength	23.1 kips	23.1 kips	0.0
HSS Shear Rupture Strength	34.31 kips[2,3]	34.3 kips	0.15
Beam Web Bearing Strength	35.78 kips[4]	52.8 kips	N/A

Table 3.3 – Capacity Comparisons

[1]AISC calculates this value by dividing $D_{req'd}$, (1.46) by D_{actual}, (3.0). This value is compared to the unity check in the weld at column limit state section in RISAConnection. If you take away the base material proration factor of 0.73 to only consider the weld, then the required capacity will become 16.86k (=23.1*0.73). Dividing this value by the available capacity gives a unity check value of 0.49.

[2]The AISC HSS shear rupture strength is compared to RISAConnection weld strength because of the base material proration factor.
[3]RISAConnection does not check HSS shear yielding strength because it is always more conservative than the HSS shear rupture strength limit state.
[4]RISAConnection calculates a different value for beam web strength because the AISC example fails to account for fact that bearing strength is not allowed to exceed the bolt shear strength.

Conclusion

In this example it is shown that the RISAConnection calculations match the design example with exception of the beam web bearing strength.

Connection 3.4- Shear Tab to Round HSS Pipe

Beam/ Round HSS Column Welded Shear Tab

This problem was adapted from example K.9 in the *AISC Design Examples version 14.1 (February 2013 revision).* The capacity values in RISAConnection are compared to those from the published example.

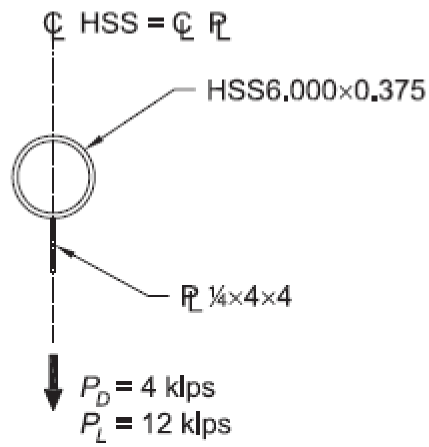


Figure 3.4- AISC Design Example K.9 Information

Comparison

Capacity Comparison			
Limit State	RISAConnection	AISC Example	% Difference
HSS Transverse Plastification	19.66 kips	19.6 kips	0.31
HSS Wall Slenderness	17.19	17.2	0.06
Material Strength	42 ksi	42 ksi	0.0
Ductility Unity Check	0.724	0.724	0.0

Table 3.4 – Capacity Comparisons

Conclusion

In this example it is shown that the RISAConnection calculations match the AISC design example.

Connection 3.5- HSS Truss Connection

HSS Branch to HSS Chord Truss Connection

This problem was adapted from example 9.2 in the *AISC Design Guide 24 (Hollow Structural Section Connections).* The capacity values in RISAConnection are compared to those from the published example.

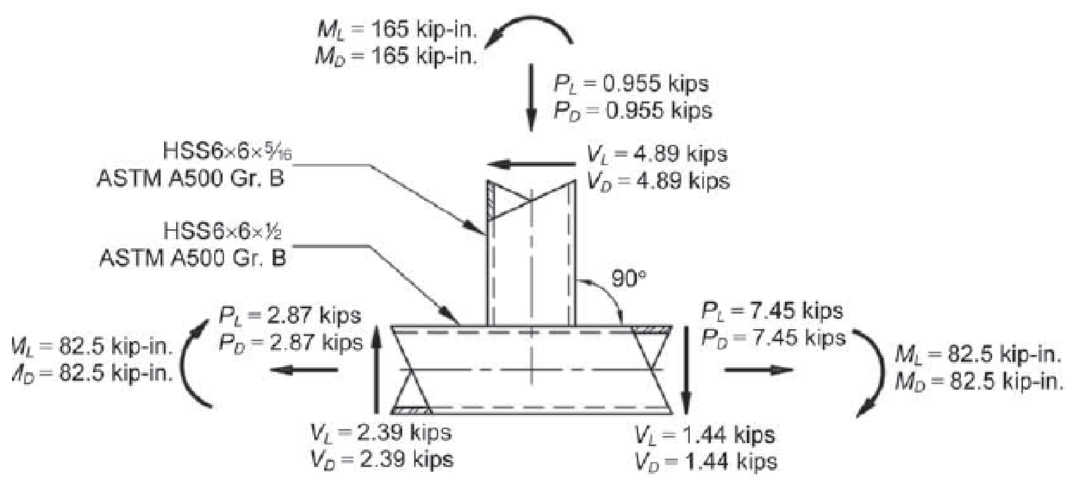


Figure 3.7- AISC Design Guide 24 Example 9.2 Information

Comparison

Results Comparison			
Limit State	RISAConnection	AISC Design Guide Example	% Difference
Chord Slenderness B/t (HSS Limitations)	12.9	12.9	0.00
Branch Slenderness B_b/t_b (HSS Limitations)	20.6	20.6	0.00
Branch Axial Local Yielding	194 kips	194 kips	0.00
Chord Sidewall Local Yielding	494 kips	494 kips	0.00
Branch Flexural Local Yielding	396 kips	396 kips	0.00

Table 3.7 – Capacity Comparisons

Conclusion

In this example it is shown that the RISAConnection calculations match the AISC design guide example exactly.

Connection 4.1- HSS Chevron Brace

Rectangular HSS Tube Chevron Brace

This problem was adapted from example II.C-5 in the *AISC Design Examples version 14.1 (February 2013 revision).* The load distribution and capacity values in RISAConnection are compared to those from the published example.

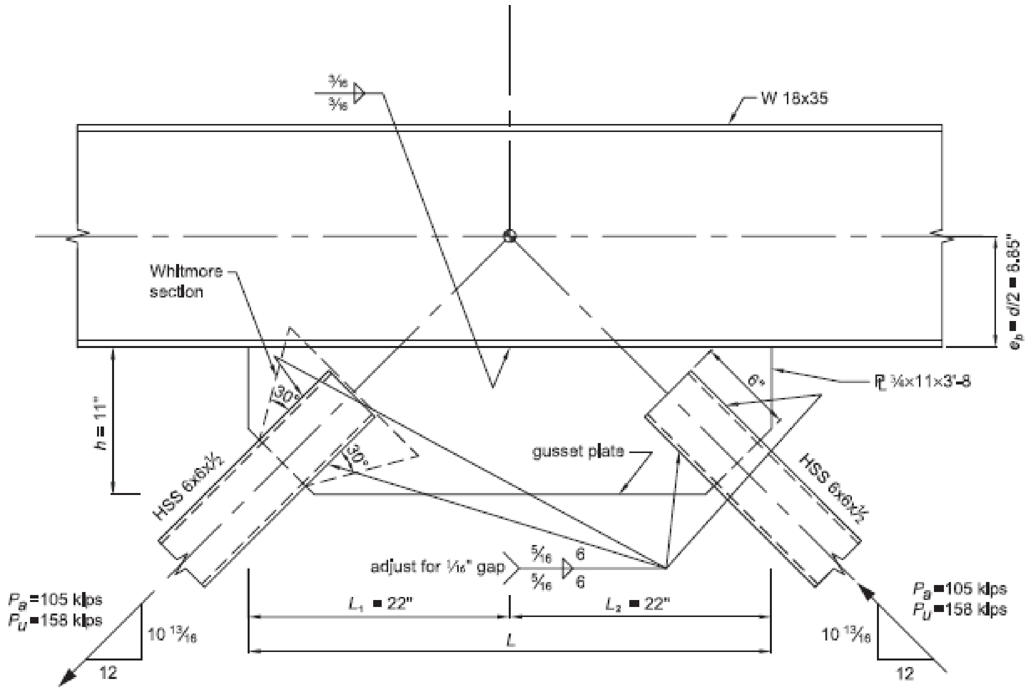


Figure 4.2a- AISC Design Example II.C-5 Information

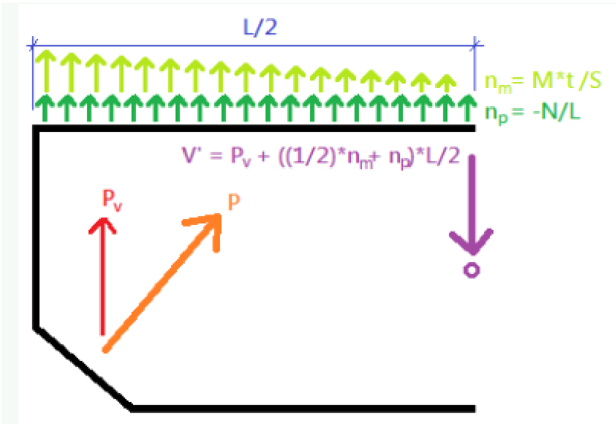
Comparison

Capacity Comparison			
Limit State	RISAConnection	AISC Example	% Difference
Axial (section a-a)[1]	0.0 k	0.0	0.0
Shear (section a-a)[1]	156.06 k	156	0.04
Moment (section a-a)[1]	1381.13 k-in	1380 k-in	0.08
Axial (section b-b)[2]	0.0 k	0.0	0.0
Shear (section b-b)[2,7]	23.17 k	7.57	NA
Moment (section b-b)[2]	0.0 k-in	0.0 k-in	0.0
Brace Tensile Yield	268.29 k	268	0.11
Brace Tensile Rupture	161.79 k	162	0.13
Brace Weld Strength UC[3]	0.94	0.943	0.31

Gusset Plate Tensile Yield	209.02 k	208	0.49
Plate Flexural Yield (section a-a) UC[4]	0.176	0.176	0.0
Plate Shear Yield (section a-a) UC[5]	0.33	0.328	0.6
Beam Weld Strength UC[6]	0.850	0.843	0.79

Table 4.2 – Capacity Comparisons

[1]Axial, Shear, and Moment for section a-a are found in Plate Flexural Yield (section a-a)
[2]Axial, Shear, and Moment for section b-b are found in Plate Flexural Yield (section b-b)
[3]This unity check is comparing the Brace Weld Strength in RISAConnection with the Brace Shear Rupture check found in the AISC example. Because the example bases this capacity on weld length, our value is compared with the demand weld length divided by the design weld length (5.66/6.00 = 0.943).
[4]The unity check in RISAConnection is found in Plate Flexural Yield (section a-a). RISAConnection uses a combined shear and moment unity check, however. To mirror the AISC example dividing the Calculated Moment over the Available Moment (note that the axial force is zero and the interaction equation is ignored) will give a similar result (1381.13 k-in/7825.15 k-in = 0.176). AISC calculates this value by dividing the available stress over the allowable stress (3.80 ksi/21.6 ksi = 0.176).
[5]The unity check in RISAConnection is found in Plate Shear Yield (section a-a) and is calculated by dividing the demand force divided by the allowable force. AISC calculates this value by dividing the available stress over the allowable stress (4.73 ksi/ 14.4 ksi = 0.328).
[6]The unity check in RISAConnection is found in the Beam Weld Strength section. This value is compared to AISC by dividing the Dreq'd by the D of the fillet weld used (2.53/3 = 0.843).
[7]The AISC calculation procedure for this value could not be verified. In RISAConnection, the shear value is calculated as V' from figure 4.4b below:



$$n_m := \frac{M \cdot t_p}{S} = -4.28\ \frac{kip}{in} \qquad n_p := \frac{-N}{L_p} = 0\ \frac{kip}{in}$$

$$V' := -P_{Vleft} + \left(\left(\frac{n_m}{2}\right) + n_p\right) \cdot \frac{L_p}{2} = 23.175\ kip$$

Figure 4.2b- RISAConnection Chevron Brace Force Distribution

Conclusion

In this example it is shown that the RISAConnection calculations match the design example, with the exception of the shear force demand calculation at Section b-b.

Connection 5.1- Base Plate Axial Only (4.1)

Base Plate with Concentric Compressive Load (No Concrete Confinement)

This problem was adapted from example 4.1 in the *AISC Design Guide 1, Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing).* The geometry and capacity values in RISAConnection are compared to those from the published example.

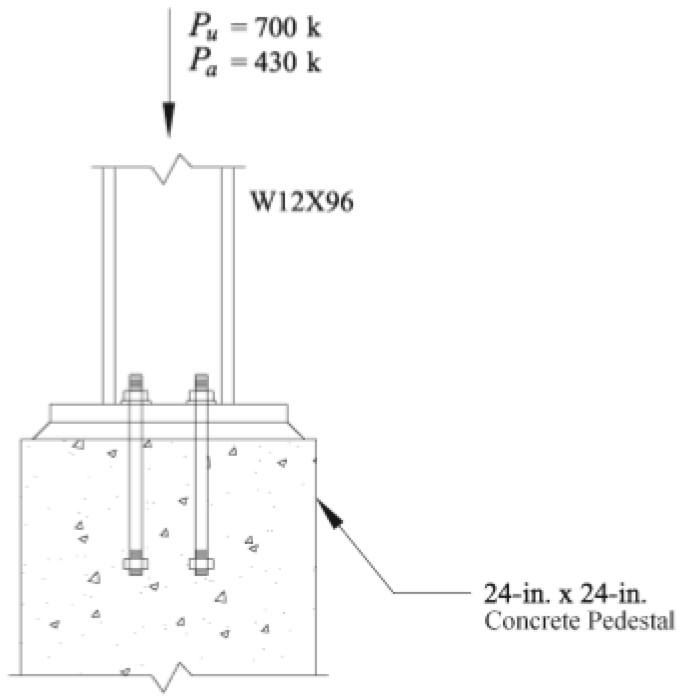


Figure 5.1- AISC Design Guide 1 Example 4.1 Information

Comparison

Results Comparison			
Limit State/Variable	RISAConnection	AISC Design Guide Example	% Difference
A_1 (per Concrete Bearing)	440 in^2	440 in^2	0.00
Concrete Bearing	529 kips	449 kips	n/a[1, 2, 3]
m (per Plate Flexural Yielding)	4.97 in	4.97 in	0.00
n (per Plate Flexural Yielding)	5.12 in	5.12 in	0.00
X (per Plate Flexural Yielding)	0.88	0.96	n/a[4]
λ (per Plate Flexural Yielding)	1.00	1.00	0.00
λn' (per Plate Flexural Yielding)	3.11 in	3.11 in	0.00

Table 5.1 – Capacity and Geometry Comparisons

[1]The axial compressive strength of the concrete can be calculated as the available bearing stress (output in RISAConnection) multiplied by the area A_1 = 1.2043 ksi * 440 in^2 = 529 kips.
[2]The Design Guide uses Ω = 2.5, however RISAConnection uses Ω = 2.31 which comes directly from AISC 360-10 section J8.
[3]The Design Guide makes the assumption that $A_1=A_2$ while RISAConnection calculates A_2 a bit higher based on the ratio of the support length to the plate length.
[4]The X variable is calculated based on the unity check from the Concrete Bearing limit state. Therefore the differences in footnotes 1 & 2 above also apply to X.

Conclusion

In this example it is shown that the RISAConnection calculations match the design example very closely.

Connection 5.2- Base Plate Axial Only (4.2)

Base Plate with Concrete Confinement

This problem was adapted from example 4.2 in the *AISC Design Guide 1, Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing).* The geometry and capacity values in RISAConnection are compared to those from the published example.

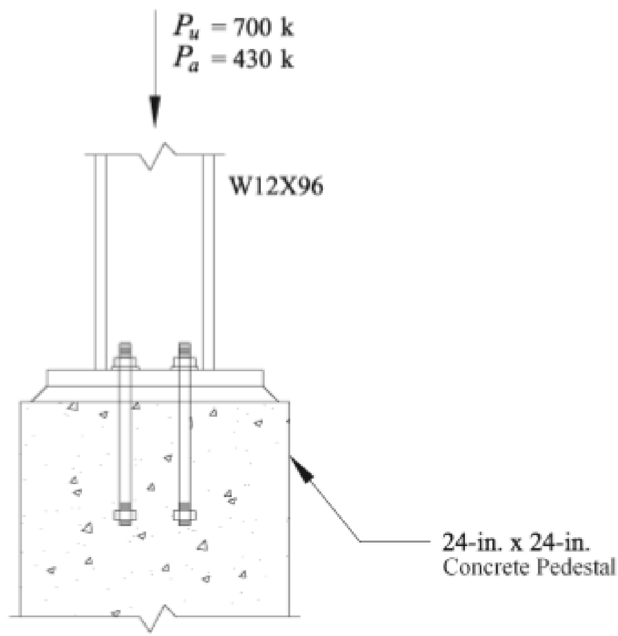


Figure 5.2- AISC Design Guide 1 Example 4.2 Information

Comparison

Results Comparison			
Limit State/Variable	RISAConnection	AISC Design Guide Example	% Difference
A_1 (per Concrete Bearing)	360 in^2	360 in^2	0.00
A_2 (per Concrete Bearing)	518 in^2	518 in^2	0.00
Concrete Bearing	477 kips	440 kips	n/a[1,2]
m (per Plate Flexural Yielding)	3.97 in	3.97 in	0.00
n (per Plate Flexural Yielding)	4.12 in	4.12 in	0.00
X (per Plate Flexural Yielding)	0.63	0.98	n/a[3]
λ (per Plate Flexural Yielding)	0.98	1.00	2
λn' (per Plate Flexural Yielding)	3.06 in	3.11 in	1.61

Table 5.2 – Capacity and Geometry Comparisons

[1]The axial compressive strength of the concrete can be calculated as the available bearing stress (output in RISAConnection) multiplied by the area A_1 = 1.3247 ksi * 360 in^2 = 477 kips.
[2]The Design Guide uses Ω = 2.5, however RISAConnection uses Ω = 2.31 which comes directly from AISC 360-10 section J8.
[3]The X variable is calculated based on the unity check from the Concrete Bearing limit state. Therefore the differences in footnotes 1 & 2 above also apply to X.

Conclusion

In this example it is shown that the RISAConnection calculations match the design example very closely.

Connection 5.3- Anchor Bolt Tension

Base Plate Anchor Bolt Tension

This problem was adapted from example 4.3 in the *AISC Design Guide 1, Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing).* The capacity value in RISAConnection is compared to that from the published example.

Comparison

Results Comparison			
Limit State/Variable	RISAConnection	AISC Design Guide Example	% Difference
Anchor Bolt Tension	9.61 kips	9.60 kips	0.10

Table 5.3 – Capacity Comparison

Conclusion

In this example it is shown that the RISAConnection calculations match the design example very closely.

Connection 5.4- Base Plate Tension Uplift

Base Plate with Column Anchorage for Tensile Loads

This problem was adapted from example 4.5 in the *AISC Design Guide 1, Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing).* The capacity values in RISAConnection are compared to those from the published example.

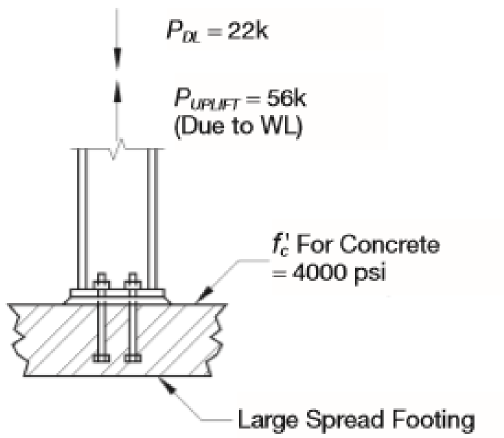


Figure 5.4- AISC Design Guide 1 Example 4.5 Information

Comparison

Results Comparison			
Limit State/Variable	RISAConnection	AISC Design Guide Example	% Difference
Anchor Bolt Tension	10.7 kips/rod	10.7 kips/rod	0.00
Plate Flexural Yielding (Tension) Required Strength	21.4 kip-in	19.5 kip-in	n/a[1]
Column Web Weld Required Strength	2.86 kips/in	2.93 kips/in	n/a[2]

Table 5.4 – Capacity Comparisons

[1]The Design Guide example calculates the tension bolt moment arm to the face of the column web whereas RISAConnection conservatively takes the moment arm as the distance to the column web centerline.
[2]The Design Guide example uses a slightly different method to determine the effective width of the column web weld. RISAConnection simplifies this to $T/(2*(d-k_{det}))$.

Conclusion

In this example it is shown that the RISAConnection calculations match the design example very closely with a few exceptions based on differing assumptions.

Connection 5.5- Base Plate with Small Moment

Base Plate with Small Moment

This problem was adapted from example 4.6 in the *AISC Design Guide 1, Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing).* The geometry and capacity values in RISAConnection are compared to those from the published example.

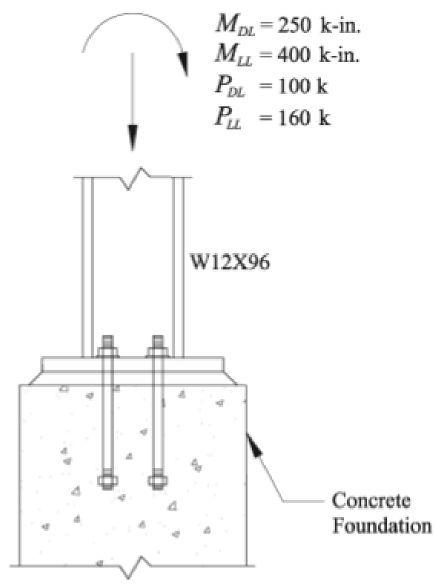


Figure 5.5- AISC Design Guide 1 Example 4.6 Information

Comparison

Results Comparison			
Limit State/Variable	RISAConnection	AISC Design Guide Example	% Difference
e (per Load Distribution Strong Axis)	2.5 in	2.5 in	0.00
e_{crit} (per Load Distribution Strong Axis)	4.85 in	4.46 in	n/a[1]
Y_z (per Load Distribution Strong Axis)	14 in	14 in	0.00
f_{pz} (per Concrete Bearing)	0.72 ksi	0.977 ksi	26.31
m (per Plate Flexural Yielding Strong Axis)	3.47 in	3.47 in	0.00
n (per Plate Flexural Yielding Strong Axis)	4.62 in	4.62 in	0.00

Table 5.5 – Capacity and Geometry Comparisons

[1]e_{crit} is calculated using q_{max} with depends on the Concrete Bearing unity check value. The Design Guide assumes Ω = 2.5 for Concrete Bearing, but RISAConnection uses Ω = 2.31 which comes from AISC 360-10 section J8.

Conclusion

In this example it is shown that the RISAConnection calculations match the design example very closely with a few exceptions based on differing assumptions.

Connection 5.6- Base Plate with Large Moment

Base Plate with Large Moment

This problem was adapted from example 4.7 in the *AISC Design Guide 1, Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing).* The geometry and capacity values in RISAConnection are compared to those from the published example.

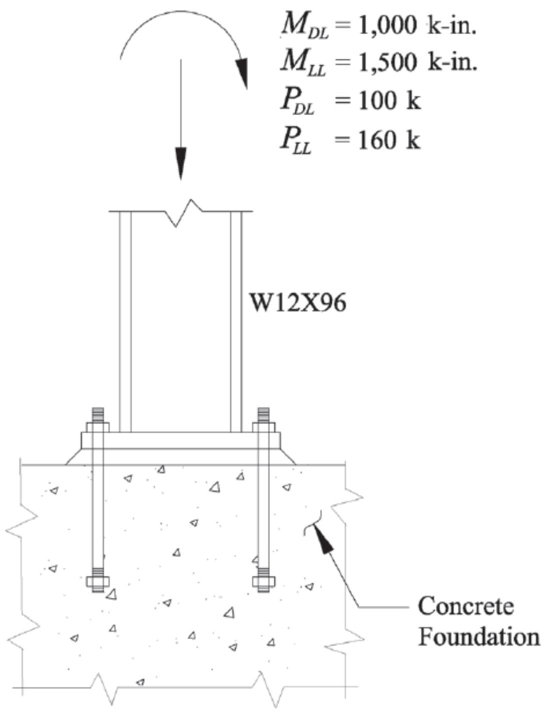


Figure 5.6- AISC Design Guide 1 Example 4.7 Information

Comparison

Results Comparison			
Limit State/Variable	RISAConnection	AISC Design Guide Example	% Difference
e (per Load Distribution Strong Axis)	9.62 in	9.62 in	0.00
e_{crit} (per Load Distribution Strong Axis)	8.48 in	6.65 in	21.58[1]
n (per Plate Flexural Yielding Strong Axis)	6.12 in	6.12 in	0.00

Table 5.6 – Capacity and Geometry Comparisons

[1]e_{crit} is calculated using q_{max} with depends on the Concrete Bearing unity check value. The Design Guide assumes Ω = 2.5 for Concrete Bearing, but RISAConnection uses Ω = 2.31 which comes from AISC 360-10 section J8.

Conclusion

In this example it is shown that the RISAConnection calculations match the design example very closely with a few exceptions based on differing assumptions.

Connection 5.7- Anchor Bolt Shear

Base Plate Anchor Bolts in Shear

This problem was adapted from example 4.10 in the *AISC Design Guide 1, Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing).* The capacity values in RISAConnection are compared to those from the published example.

Note: You must change the design code to "AISC 360-10 (14th Edition) – LRFD" to match the results below.

Comparison

Results Comparison			
Limit State/Variable	RISAConnection	AISC Design Guide Example	% Difference
Anchor Bolt Shear	30.75/4 = 7.69 kips[1]	7.69 kips	0.00[2]

Table 5.7 – Capacity Comparison

[1]This is a strength level (LRFD) result.
[2]The Design Guide calculates the shear strength as the sum of shear strength for all four bolts. RISAConnection conservatively assumes that only two bolts resist the shear. This suggestion comes from section 3.5.3 of the Design Guide.

Conclusion

In this example it is shown that the RISAConnection calculations match the design example very closely with a few exceptions based on differing assumptions.

Connection 5.8- Anchor Bolt Combined Shear & Tension

Base Plate Anchor Bolts in Combined Shear and Tension

This problem was adapted from example 4.11 in the *AISC Design Guide 1, Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing).* The geometry and capacity values in RISAConnection are compared to those from the published example.

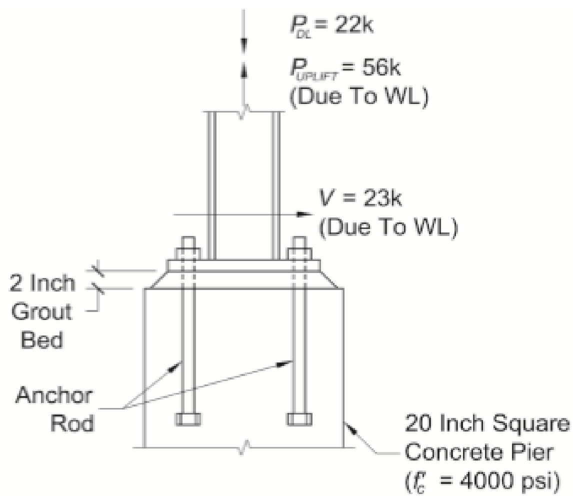


Figure 5.8 – AISC Design Guide 1 Example 4.11 Information

Comparison

Results Comparison			
Limit State/Variable	RISAConnection	AISC Design Guide Example	% Difference
f_{tb} (per Anchor Bolt Bending)	18.2 ksi	18.2 ksi	0.00
f_{rt} (per Anchor Bolt Bending)	10.8 ksi	10.8 ksi	0.00
F_{nt} (per Anchor Bolt Bending)	43.5 ksi	43.5 ksi	0.00
F_{nv} (per Anchor Bolt Bending)	26.1 ksi	23.2 ksi	n/a[1]
Anchor Bolt Bending	18.63 ksi	17.4 ksi	n/a[2]

Table 5.8 – Capacity Comparisons

[1]The Design Guide calculates F_{nv} = 0.4*Fu. The 0.4 factor comes from the AISC 360-05 (13th edition) code. RISAConnection defaults to use the AISC 360-10 (14th edition) code which calculates F_{nv} = 0.45*Fu.

[2]Anchor Bolt Bending depends on the F_{nv} bolt shear strength. Therefore the discrepancies of note 1 apply to this check as well.

Conclusion

In this example it is shown that the RISAConnection calculations match the design example very closely with a few exceptions based on differing assumptions.

Connection 6.1- OMF Extended End Plate

Ordinary Moment Frame Extended End Plate Seismic Connection

This problem was adapted from example 4.2.4 in the *AISC Seismic Design Manual (2010).* The geometry and capacity values in RISAConnection are compared to those from the published example.

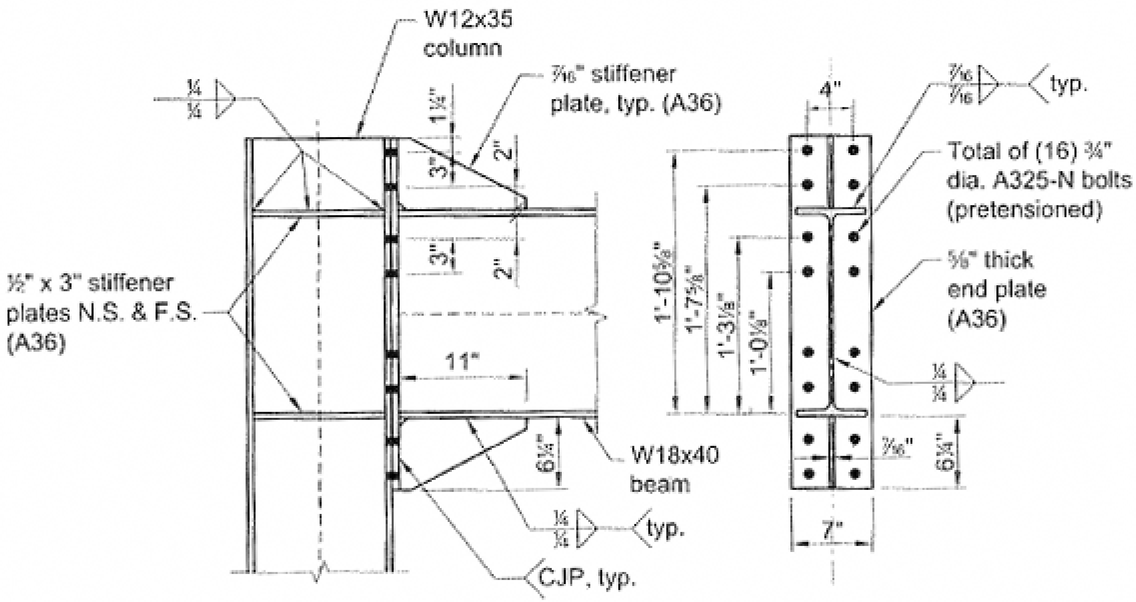


Figure 6.1 – AISC Seismic Design Manual Example 4.2.4 Information

Comparison

Results Comparison			
Limit State/Variable	RISAConnection	AISC Design Example	% Difference
M_{pr} (per Seismic Moment at Face of Column)	3162 kip-in	3100 kip-in	1.96
Y_p (per End Plate Flexural Yielding)	232 in	232 in	0.00
Bolt Shear Strength	95.4 kips	95.5 kips	0.10
Bolt Bearing on Plate at Column	95.4 kips	233 kips	n/a[1]
Y_c (per Column Flexural Yielding)	239 in	239 in	0.00
R_{cf}/Ω (per Column Flexural Yielding)	111.35 kips	111.08 kips[2]	0.24
Column Web Yielding	67 kips	74.7 kips	n/a[3]
Column Web Crippling	68 kips	60 kips	n/a[4]

Table 6.1 – Capacity and Geometry Comparisons

[1] Per Section J3.10 user note, RISAConnection takes the minimum of both bolt bearing and bolt shear on an individual bolt and uses that value. The AISC design example only uses the bolt bearing value. Because bolt shear controls for many bolts this value is much less in RISAConnection.

[2]The design example presents the strength as a moment. This is converted into a force by dividing by the moment arm in order to compare to RISAConnection.

[3] The design example uses fillet welds but RISAConnection only offers CJP welds at the beam flange to end plate. Therefore the bearing length is different.

[4] The design example uses a bearing length equal to the beam flange thickness. RISAConnection uses a longer projected bearing length for the column web which results in a higher capacity.

Conclusion

In this example it is shown that the RISAConnection calculations match the design example very closely with a few exceptions based on program assumptions