

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 16*
- *AISC Design Guide 1, Base Plate and Anchor Rod Design (2nd Edition, 2nd Printing)*
- *AISC Design Guide 24, Hollow Structural Section Connections (2nd Edition)*
- *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 17.

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 16*. The slip critical bolt capacities in RISACONNECTION are compared to those from the published example.

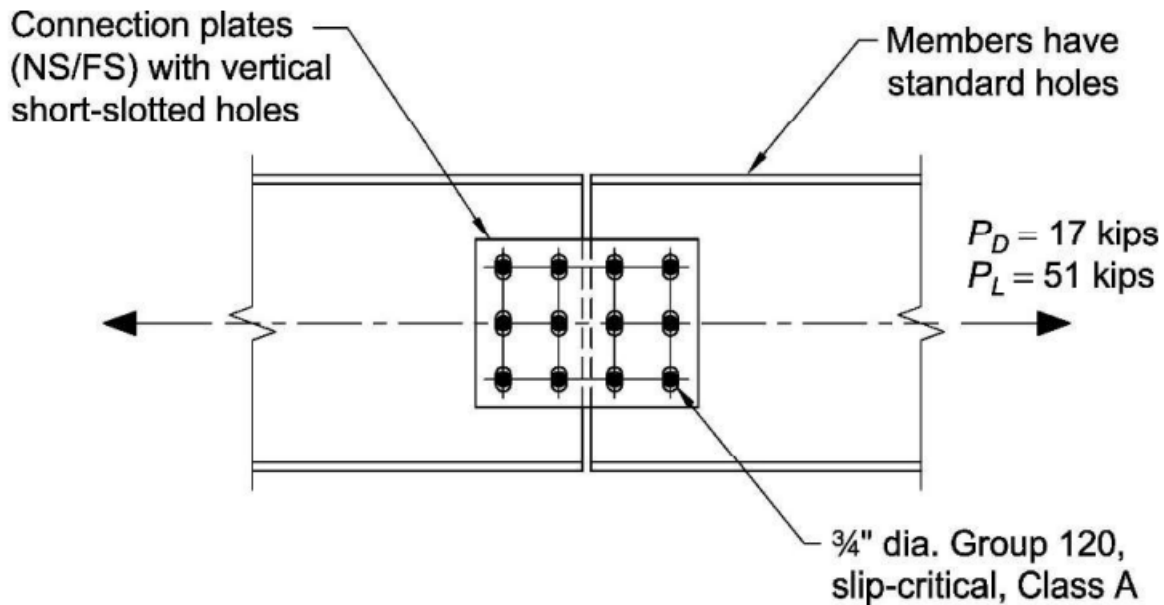


Figure 1.1A - AISC Design Example J.4A Information

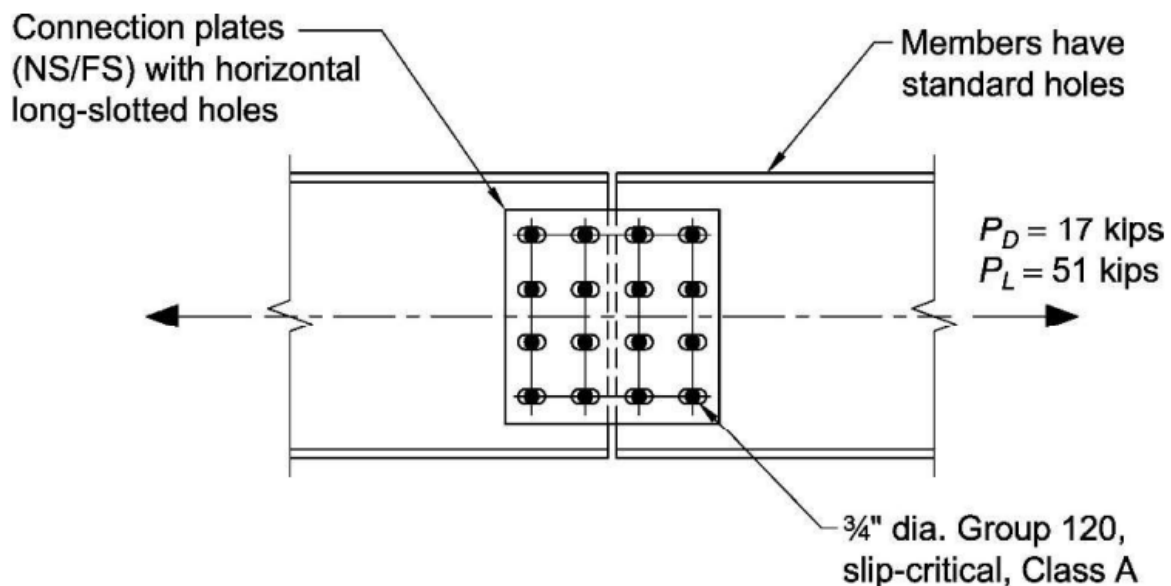


Figure 1.1B - AISC Design Example J.4B Information

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	2.3
Bolt Slip Critical – J.4B	$69.66/8 = 8.71$ kips per bolt ^{1,2}	8.88 kips per bolt	1.95

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-1A in the *AISC Design Examples version 16*. The shear capacity of an all bolted double angle connection in RISACONNECTION is compared to that from the published example.

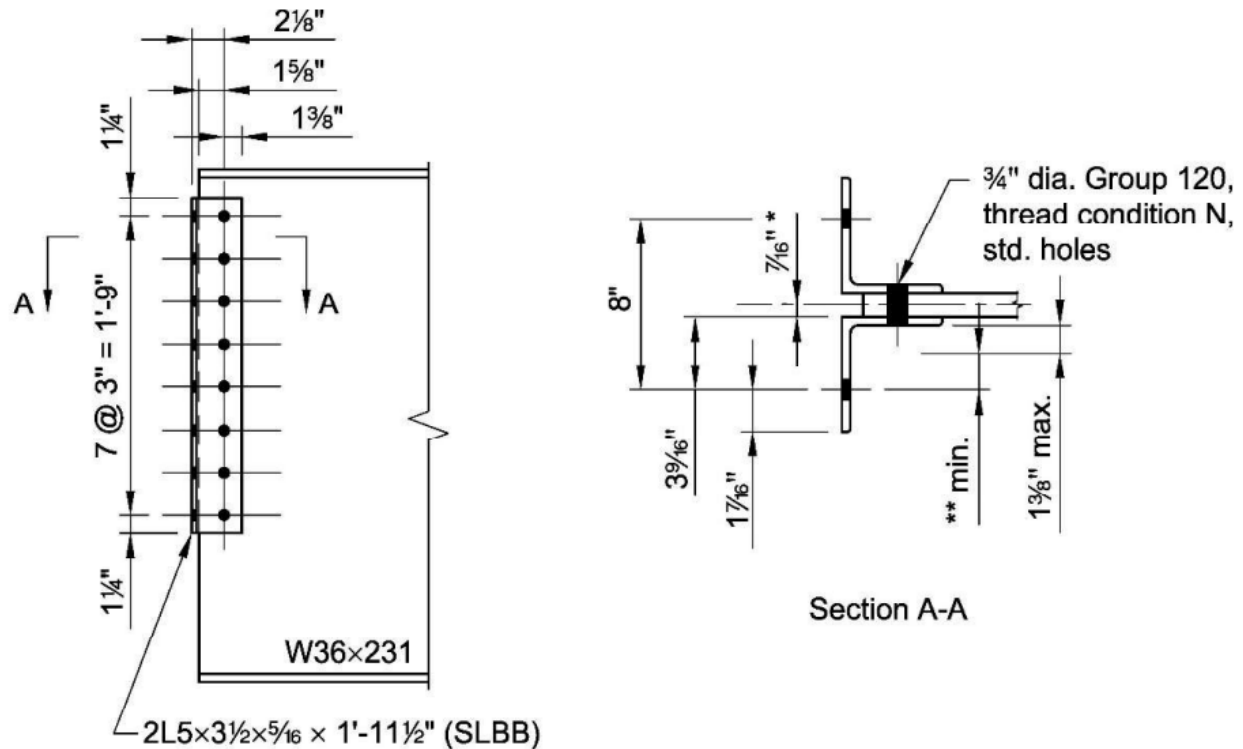


Figure 1.2- AISC Design Example II.A-1A Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISACONNECTION	AISC Example	% Difference
Min Value-1 ¹	201.42	201	0.21
Min Value-2 ²	187.59	187	0.32

Table 1.2a – Capacity Comparison

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

²Note that the **Min Value-2** here is the minimum limit state of bolt bearing checks. See Table 1.2c below for this value.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Clip Angle Shear Yield	294.22
Clip Angle Shear Rupture at Beam	201.42
Clip Angle Shear Rupture at Column	201.42
Clip Angle Block Shear at Beam	208.03
Clip Angle Block Shear at Column	211.84
Min Value-1	201.42

Table 1.2b – Capacities from RISACONNECTION

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Bearing on Column	190.85
Bolt Bearing on Beam	190.85
Bolt Bearing on Clip Angle at Beam	187.59
Bolt Bearing on Clip Angle at Column	187.59
Min Value-2	187.59

Table 1.2c – Capacities from RISACONNECTION

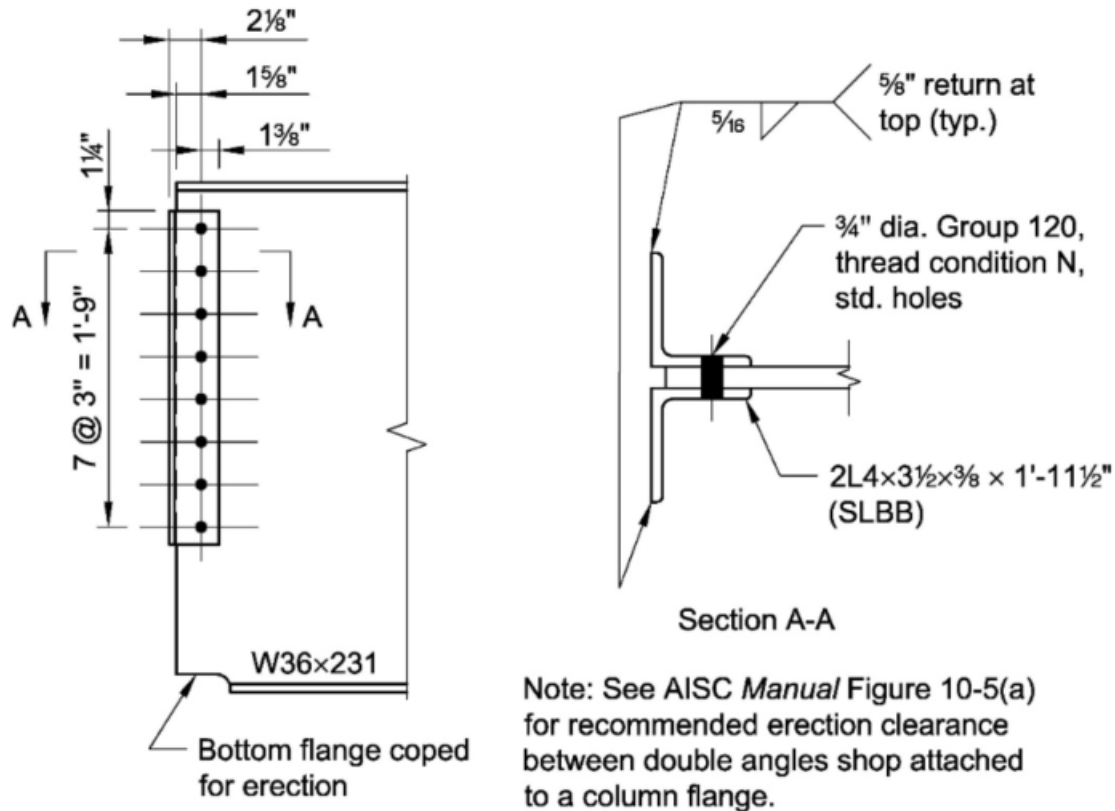
Conclusion

In this example many of the calculations are very similar between RISACONNECTION and the AISC example.

Connection 1.3- Bolt/Weld Double Angle Shear

Beam/Column Bolted/Welded Double-Angle Connection

This problem was adapted from example II.A-2A in the *AISC Design Examples version 16*. 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.



Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISAConnection	AISC Example	% Difference
Column Weld Strength	181.09 ¹	212	17.07
Min Value-1 ²	241.31	241	0.13
Min Value-2 ³	190.85	190	0.45

Table 1.3a – Capacity Comparison

¹The 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.

²Note that the **Min Value-1** here is the minimum limit state of shear rupture and block shear rupture of the angles. See Table 1.3b below for this value and reasoning for the differences.

³Note that the **Min Value-2** here is the minimum limit state of bolt bearing. See Table 1.3c below for this value and reasoning for the differences.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Clip Angle Shear Rupture at Beam	241.31
Clip Angle Shear Rupture at Column	343.69
Clip Angle Block Shear at Beam	249.23
Min Value	241.31

Table 1.3b – Capacities from RISACONNECTION

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Shear at Beam	190.85
Bolt Bearing on Beam	190.85
Bolt Bearing on Clip Angle at Beam	190.85
Min Value	190.85

Table 1.3c – Capacities from RISACONNECTION

Conclusion

In this example many of the calculations are very similar between RISACONNECTION and the AISC example.

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 16*. 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.

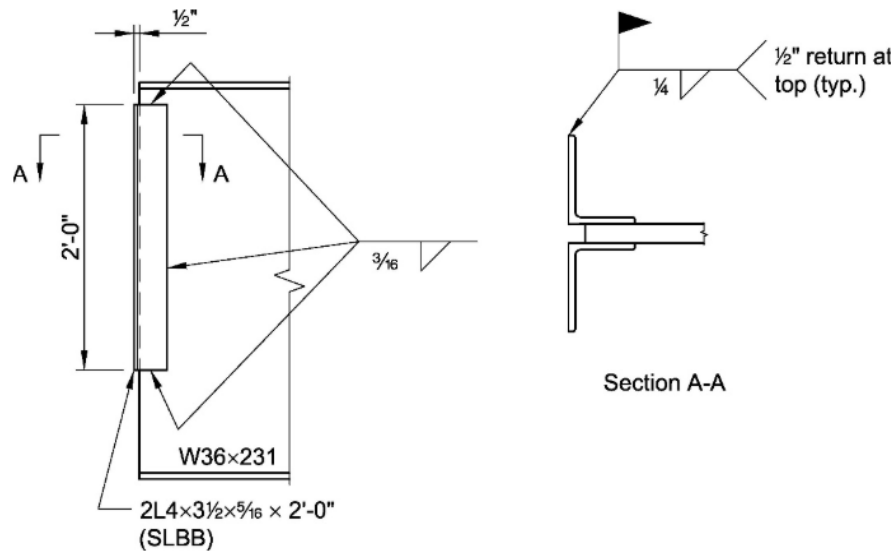


Figure 1.4 - AISC Design Example II.A-3 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISACONNECTION	AISC Example	% Difference
Beam Weld Strength	172.70 ¹	171	0.99
Column Weld Strength	148.91 ¹	153	2.7
Clip Angle Shear Yield	300.48	300	0.16

Table 1.4 - Capacity Comparison

¹The 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 16*. The shear capacity in RISAConnection is compared to that from the published example.

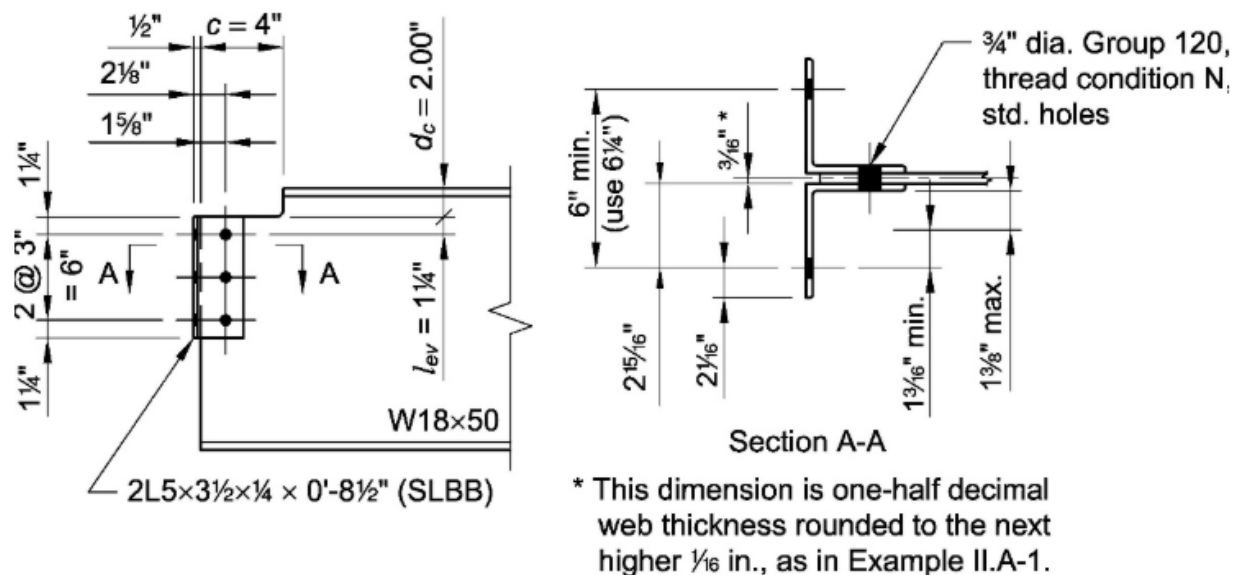


Figure 1.5 – AISC Design Example II.A-4 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISACconnection	AISC Example	% Difference
Min Value ¹	57.28	57.3	0.03
Min (Bolt Bearing on Beam, Beam Block Shear)	(53.22, 50.19 ²) = 50.19	49.0	2.43
Bolt Bearing on Clip Angle at Girder	64.17	64.1	0.11
Coped Beam Local Web Buckling	248.29	249	0.29
Beam Shear Yield	113.60	113	0.53
Beam Block Shear Rupture	50.19	45.8	9.59

Table 1.5a – Capacity Comparison

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

²AISC Table 10-1 automatically subtracts 1/4" 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.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Bolt Bearing on Clip Angle at Beam	64.17
Bolt Bearing on Clip Angle at Girder	64.17
Clip Angle Shear Yield	85.0
Clip Angle Shear Rupture at Beam	57.28
Clip Angle Shear Rupture at Girder	57.28
Clip Angle Block Shear at Beam	62.56
Clip Angle Block Shear at Girder	75.60
Min Value	57.28

Table 1.5b – Capacities from RISACONNECTION

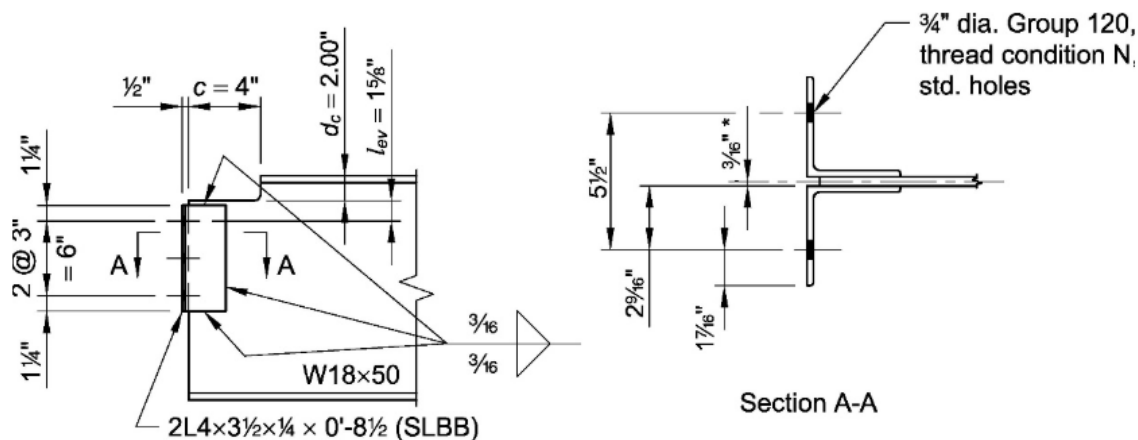
Conclusion

In this example many of the calculations are very similar between RISACONNECTION and the AISC example.

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 16.0*. The shear capacity of bolts and welds in RISACONNECTION are compared to those from the published example.



* This dimension is one-half decimal web thickness rounded to the next higher $\frac{1}{16}$ in., as in Example II.A-1.

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	64.17	64.1	0.27
Min Value ¹	57.28	57.3	0.03
Beam Shear Yield	113.60	113	0.53

Table 1.6a – Capacity Comparison

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

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Clip Angle Block Shear at Girder	70.69
Clip Angle Shear Rupture at Beam	82.88
Clip Angle Shear Rupture at Girder	57.28
Clip Angle Shear Yield	85.00
Min Value	57.28

Table 1.6b – Capacities from RISACONNECTION

Conclusion

In this example many of the calculations are very similar between RISACONNECTION and the AISC example.

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 16.0*. The shear capacity in RISACONNECTION is compared to that from parts A and B of the published example.

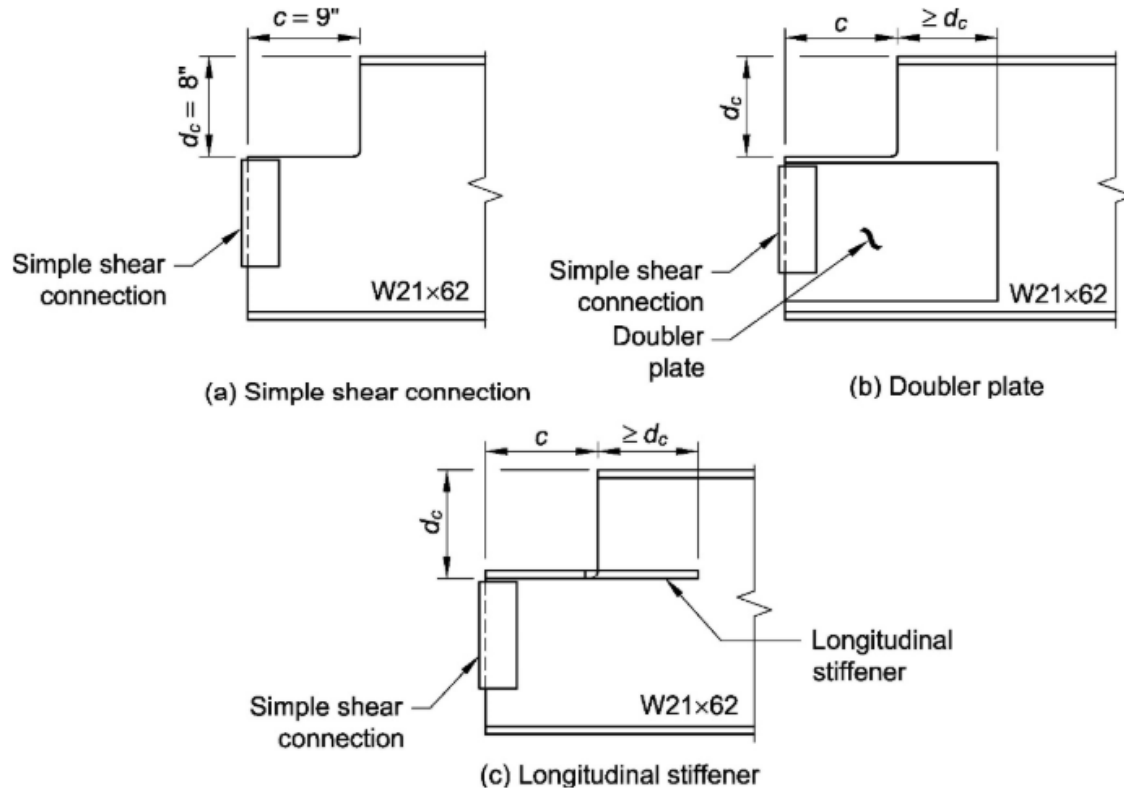


Figure 1.7 – AISC Design Example II.A-6 Information

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	77.69	77.2	0.63
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

¹You 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 16.0*. The Lateral Torsional Buckling limit state capacity in RISACONNECTION is compared to that from of the published example.

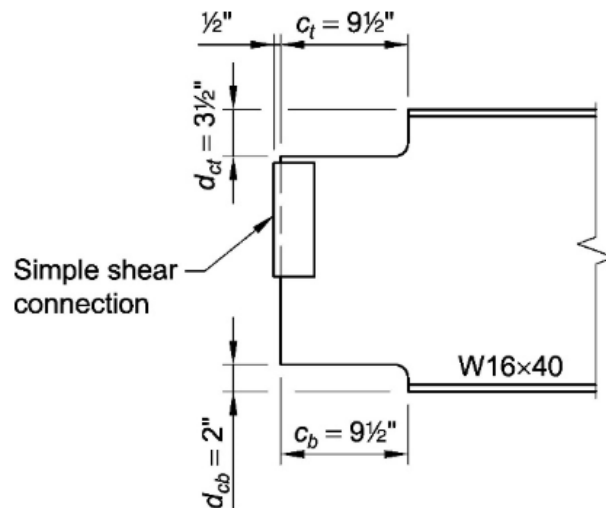


Figure 1.8 – AISC Design Example II.A-7 Information

Calculation and Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISACONNECTION	AISC Example	% Difference
Coped Beam Lateral Torsional Buckling	25.17	25.2	0.12

Table 1.8 – Capacity Comparison

Conclusion

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

Connection 1.9- Bolted Shear End Plate Shear

Beam/Girder Bolted End-Plate Connection

This problem was adapted from example II.A-11A in the *AISC Design Examples version 16.0*. The shear capacity of a beam to girder end plate shear connection in RISACONNECTION is compared to that from of the published example.

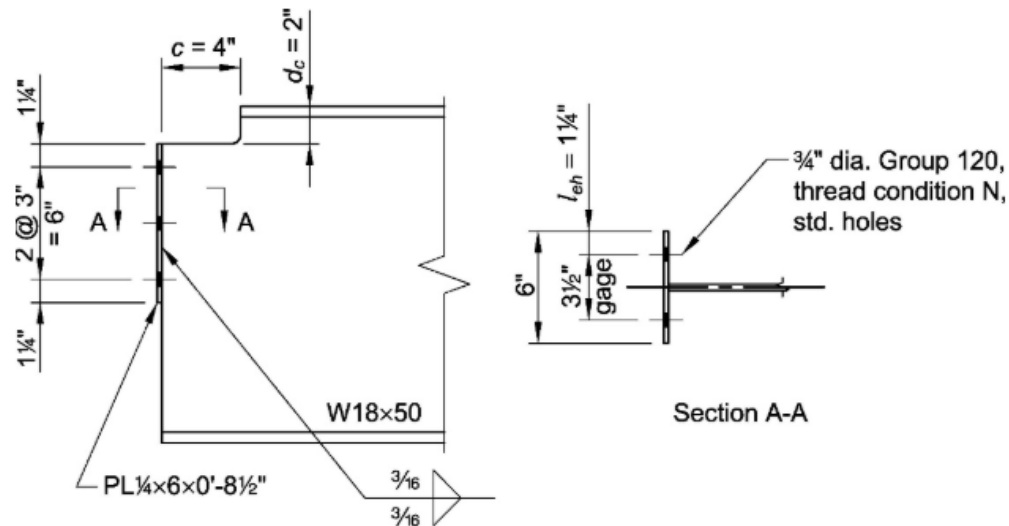


Figure 1.9 – AISC Design Example II.A-11A Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISACONNECTION	AISC Example	% Difference
Min Value-1 ¹	57.28	57.3	0.03
Min Value-2 ²	45.24	45.2	0.09
Bolt Bearing on Plate at Girder	64.17	64.1	0.11

Table 1.9a – Capacity Comparison

¹Note that the **Min Value-1** here is the minimum limit states of shear rupture of the end plate and block shear rupture of the end plate. See Table 1.9b below for these values.

²Note 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.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Plate Shear Rupture at Girder	57.28
Plate Block Shear at Girder	62.56
Min Value	57.28

Table 1.9b – Capacities from 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.

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 16*.

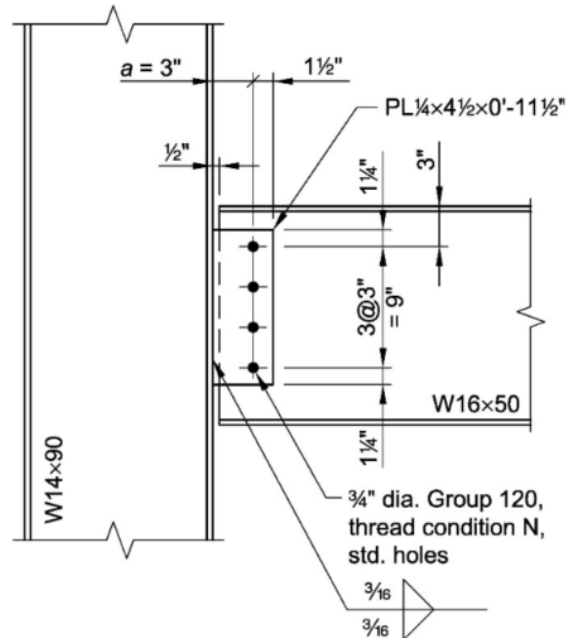


Figure 1.10 – AISC Design Example II.A-17 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISACONNECTION	AISC Example	% Difference
Min Value ¹	39.00	38.9	0.26

Table 1.10a – Capacity Comparison

¹Note 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.

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
Beam Shear Yield	123.88
Plate Shear Yield	41.40
Beam Shear Rupture	94.85
Plate Shear Rupture at Beam	39.00
Plate Block Shear at Beam	43.67
Min Value	39.00

Table 1.10b – Capacities from RISACONNECTION

¹The 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.

Conclusion

In this example many of the calculations are very similar between RISACONNECTION and the AISC example.

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 16.0*. The shear capacity in RISACONNECTION is compared to that from of the published example.

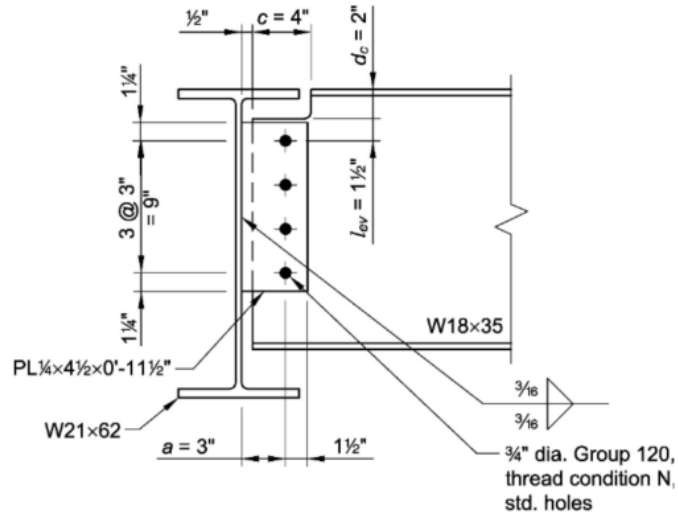


Figure 1.11 – AISC Design Example II.A-18 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISACONNECTION	AISC Example	% Difference
Min Value ¹	39.0	39.0	0.00
Beam Block Shear	63.62	58.7	8.38

Table 1.11a – Capacity Comparison

¹Note 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 1.11b 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
Bolt Bearing on Beam	47.71
Bolt Bearing on Plate at Beam	44.01
Beam Shear Yield	94.20
Plate Shear Yield	57.50
Beam Shear Rupture	71.37
Plate Shear Rupture at Beam	39.00
Plate Block Shear at Beam	43.67
Min Value	39.00

Table 1.11b – Capacities from RISACONNECTION

¹The 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.

Conclusion

In this example many of the calculations are very similar between RISACONNECTION and the AISC example.

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 16.0*. The shear capacity in RISAConnection is compared to that from of the published example.

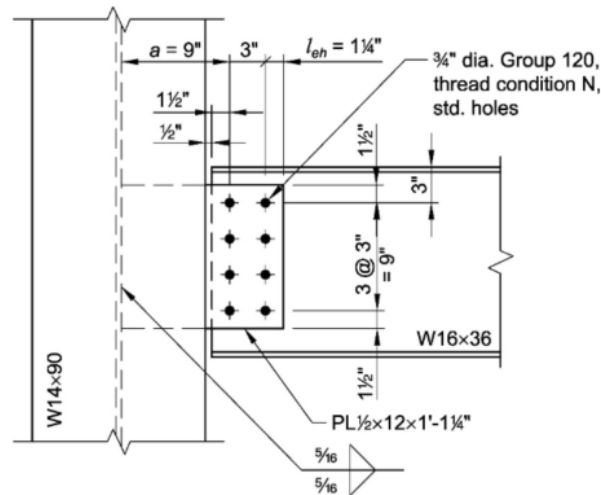


Figure 1.12 – AISC Design Example II.A-19 Information

Comparison

Capacity Comparison (All Results Shown in kips unless noted otherwise)			
Limit State	RISAC Connection	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)	$42.66/2 = 21.33^1$	21.3	0.14
Plate Shear Yield	120.0	120.0	0.00
Plate Shear Rupture at Beam	82.8	83.0	0.24
Plate Block Shear	96.38	96.5	0.12
Plate Flexural Yielding	538.92 k-in	539 k-in	0.01
Plate Flexural Rupture	414.38 k-in	416 k-in	0.39
Column Web Flexural Yielding	193.54 k-in	N/A ²	N/A

Table 1.12 – Capacity Comparison

¹The 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 $R_{n-edge-tearout}$ capacity controls between these values and is reported as 42.66 kips.

²The AISC V16.0 Design Guide Example II.A-19A notes "all dimensional limitations are satisfied" without explicitly showing this calculation. RISACONNECTION performs the column web flexural yielding check per AISC 16th Manual Eq. 9-47 (yield line analysis), which is required for single-plate connections to column webs per Manual Part 9. This check is not shown in the design guide example but is not omitted from the code. This is an additional required check that RISACONNECTION correctly implements.

Conclusion

In this example many of the calculations are very similar between RISACONNECTION and the AISC example with the exception of Column Web Flexural Yielding requirements.

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 16.0*. The shear capacity in RISACONNECTION is compared to that from of the published example.

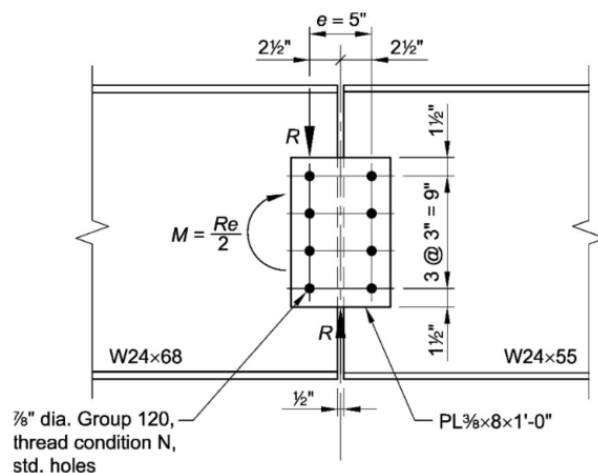


Figure 1.13 – AISC Design Example II.A-20 Information

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	30.23
Bolt Bearing on Plate	$30.16/2 = 15.08^2$	15.1	0.13
Plate Flexural Yielding	404.19 kip-in	404 kip-in	0.05
Plate Flexural Rupture	292.50 kip-in	293 kip-in	0.17
Plate Shear Yielding	90.0	90.0	0.00
Plate Shear Rupture	58.5	58.5	0.00
Plate Block Shear	63.38	63.6	0.35

Table 1.13 – Capacity Comparison

¹In 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 30% difference.

²The 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 -edge-tearout capacity, listed as 30.16 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 16.0*. The shear capacity in RISACONNECTION is compared to that from of the published example.

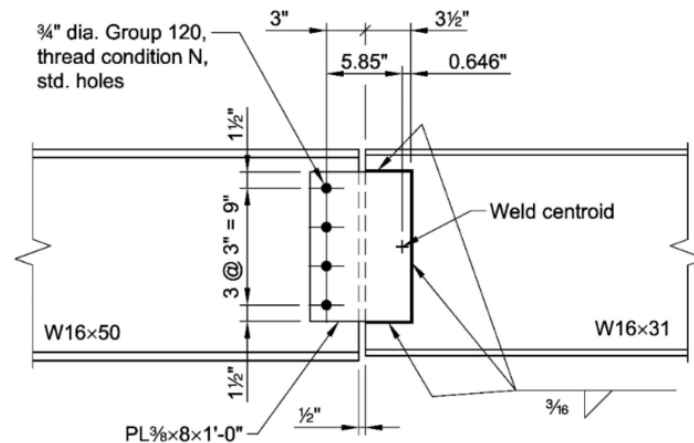


Figure 1.14 – AISC Design Example II.A-21 Information

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)	$47.71/2 = 23.86$	22.0	8.45
Plate Flexural Yielding	404.19 kip-in	404 kip-in	0.05
Plate Shear Yielding	90.0	90.0	0.00
Plate Shear Rupture	62.16	62.0	0.26
Plate Block Shear	67.34	67.3	0.06

Table 1.14 – Capacity Comparison

¹In 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.

Conclusion

In this example it is shown that the RISACONNECTION calculations match the design examples. The only minor differences are due to how RISACONNECTION calculates the eccentricity on the bolt group.

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 16.0*. 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**”.

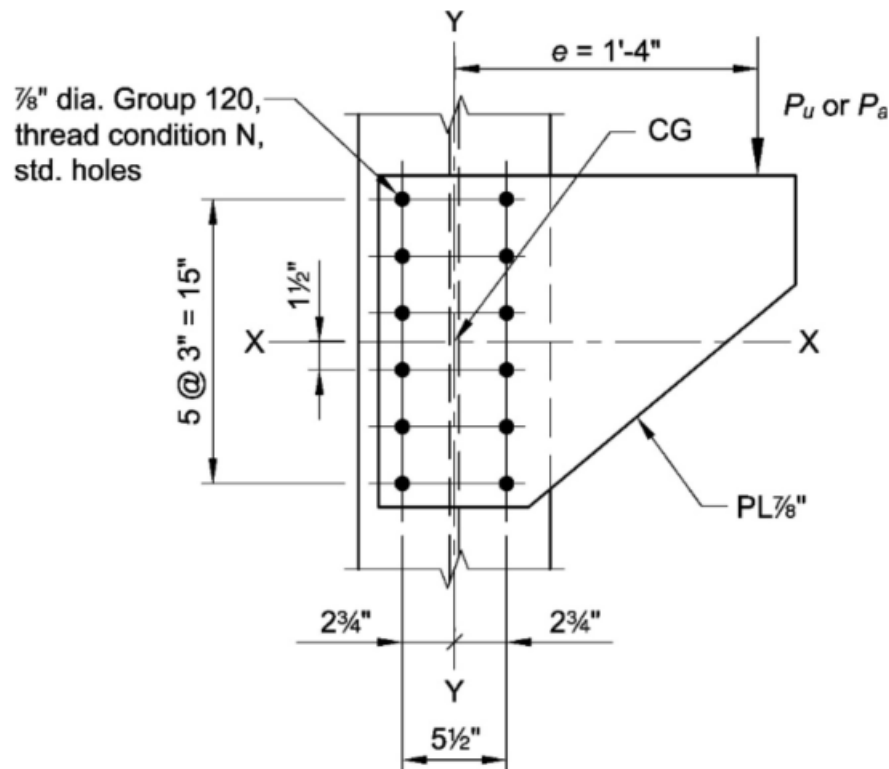


Figure 1.16 – AISC Design Example II.A-25 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISACONNECTION	AISC Example	% Difference
Bolt Shear	46.06	46.0	0.13
Eccentricity Coefficient ¹	2.88	1/0.352 = 2.84 ²	1.41

Table 1.16 – Capacity Comparison

¹Note that in RISACONNECTION the eccentricity coefficient is obtained from multiplying the eccentricity coefficient ($C=0.24$) by the number of bolts ($N_{\text{bolt}}=12$).

²The 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.

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.

²The 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.

³The 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 -edge-tearout capacity, listed as 24.68 kips.

⁴Because of the eccentricity of the single angle there is flexure in the leg attached to the column. Currently RISACONNECTION does not check the single angle for these failure modes.

⁵The **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
Beam Block Shear	52.41 ⁶
Min Value	45.66

Table 1.17b – Capacities from RISACONNECTION

⁶The 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

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.

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 16.0*. The shear capacity in RISAConnection is compared to that from the published example.

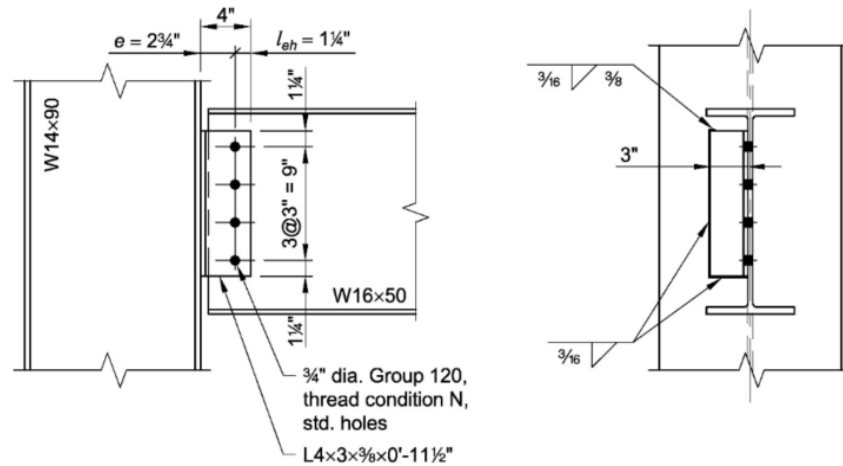


Figure 1.18 – AISC Design Example II.A-29 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISACconnection	AISC Example	% Difference
Min Value ¹	35.05	58.5	66.9
Column Weld Strength	37.93	37.8	0.34
Bolt Bearing on Beam	47.71	47.6	0.23

Table 1.18a – Capacity Comparison

¹Note 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.

Capacity Comparison (All Results Shown in Kips)	
Limit State	Capacity
Clip Angle Shear Yield	86.25
Clip Angle Shear Rupture at Beam	58.50
Clip Angle Shear Rupture at Column	84.09
Bolt Bearing on Clip Angle at Beam	47.71
Clip Angle Block Shear at Beam	62.46
Bolt Shear	35.05 ¹
Min Value	35.05

Table 1.18b – Capacities from RISACONNECTION

¹The 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 difference is due to eccentricity assumptions.

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 16.0*. The capacity values in RISAConnection are compared to those from the published example.

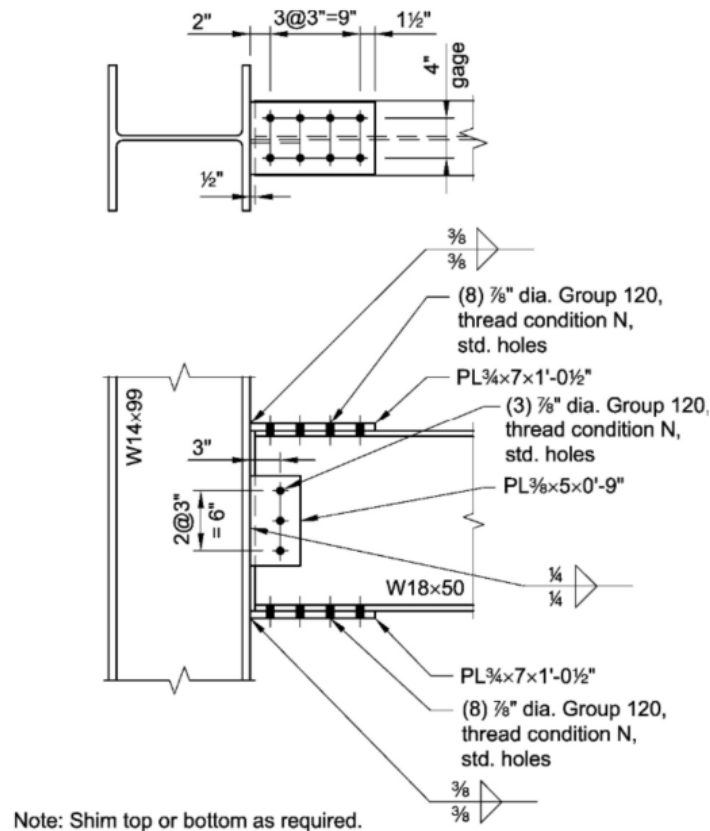


Figure2.1- AISC Design Example II.B-1 Information

Comparison

Capacity Comparison (All Results Shown in Kips unless otherwise noted)			
Limit State	RISACONNECTION	AISC Example	% Difference
Beam Flexural Rupture/Strength	211.4 kip-in	211 kip-ft	0.19
Bolt Shear at Beam Web (single bolt)	$48.71/3 = 16.23$	16.2	0.19
Bolt Bearing Strength at Vert. Plate at Middle Bolt	$51.19/2 = 25.60^1$	22.9	11.80
Vert. Plate Shear Yielding	67.5	67.3	0.30
Vert. Plate Shear Rupture	43.88	43.9	0.13
Plate Block Shear at Beam	46.69	54.9	0.05
Shear Plate Weld Strength at Column	66.82	66.8	0.03
Flange Plate Tensile Yield	157.19	157	0.12
Flange Plate Tensile Rupture	121.88	122	0.10
Flange Plate Block Shear	253.50^2	254	0.20
Beam Flange Block Shear	201.92^3	197	2.50
Flange Plate Compression	157.19	157	0.12
Column Flange Bending	113.85	114	0.13
Column Web Yielding	135.8^4	124	9.52
Column Web Crippling	166.01^4	155	7.10

Table 2.1 – Capacity Comparison

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

²The 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.

³The program uses a conservative $L_e = 1.25"$. The actual $L_e = 1.5"$ (to center of bolt). RISACONNECTION uses the actual values and give slightly higher capacities than the AISC.

⁴The 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 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 16.0*. 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.

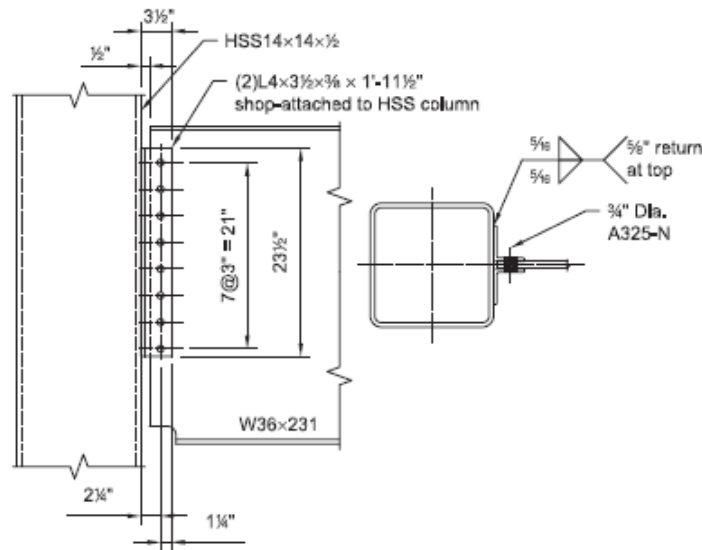


Figure 3.1- AISC Design Example K.3 Information

Comparison

Capacity Comparison (All Results Shown in Kips)			
Limit State	RISACONNECTION	AISC Example	% Difference
Bolt Bearing Strength	190.85	190	0.45
Column Weld Strength	186.14	212	13.90
Clip Angle Shear Rupture	248.62	241	3.16

Table 3.1 – Capacity Comparisons

Conclusion

In this example it is shown that the RISACONNECTION calculations mostly 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 16.0*. The capacity values in RISACONNECTION are compared to those from the published example.

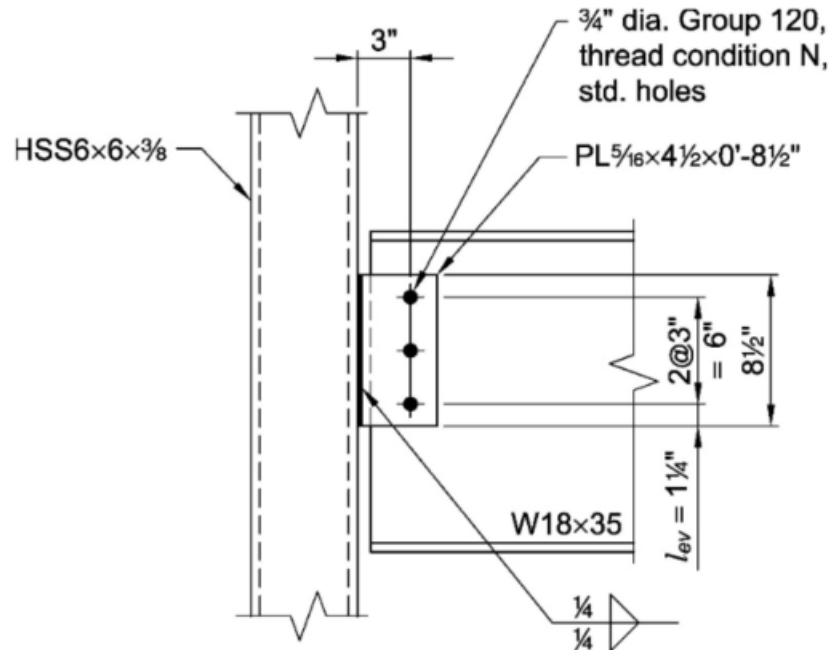


Figure 3.2- AISC Design Example K.6 Information

Comparison

Capacity Comparison			
Limit State	RISACONNECTION	AISC Example	% Difference
HSS Wall Slenderness	14.19 in	14.2 in	0.08
Material Strength	46 ksi	46 ksi	0.0
Governing Plate Strength (Rupture)	35.8	35.8	0.0
Bolt Eccentricity coefficient	0.83	0.823	0.85
Single-Plate Connection Strength per the Bolt Shear at Beam check	29.60 kips	28.1 kips	5.34

Table 3.2 – Capacity Comparisons

Conclusion

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

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 16.0*. The capacity values in RISACONNECTION are compared to those from the published example.

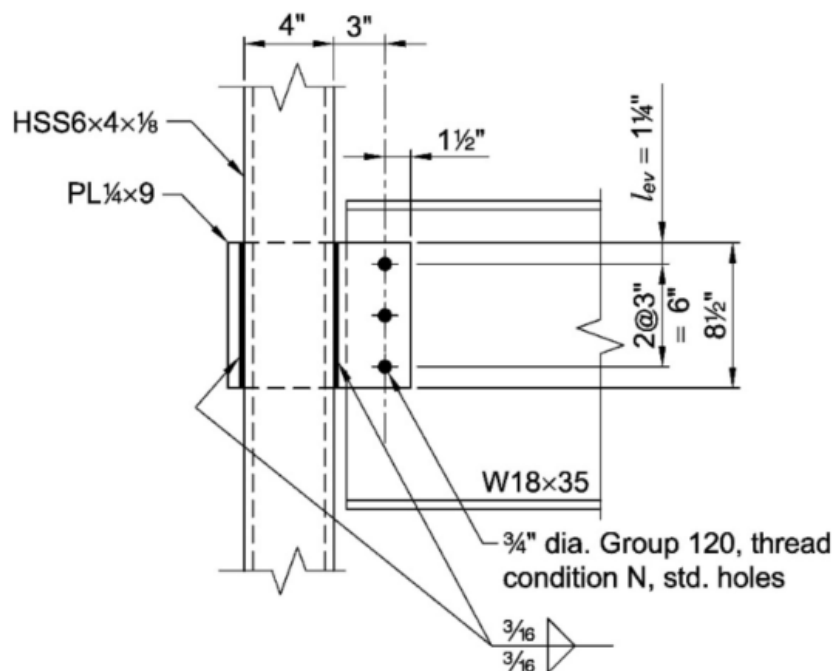


Figure 3.3- AISC Design Example K.7 Information

Comparison

Capacity Comparison			
Limit State	RISACONNECTION	AISC Example	% Difference
Plate Shear Rupture at Beam	28.64 kips	28.6 kips	0.14
Weld at Column (Near) Unity Check	0.485 ¹	0.486	0.21
Required Weld Strength	23.1 kips	23.1 kips	0.0
HSS Shear Rupture Strength	34.31 kips ^{2,3}	36.7 kips	0.15

Table 3.3 - Capacity Comparisons

¹AISC 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 0.72 to only consider the weld, then the required capacity will become 16.63k (=23.1*0.72). Dividing this value by the available capacity gives a unity check value of 0.485.

²The AISC HSS shear rupture strength is compared to RISACONNECTION weld strength because of the base material proration factor.

³RISACONNECTION does not check HSS shear yielding strength because it is always more conservative than the HSS shear rupture strength limit state.

Conclusion

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

Connection 3.4- Shear Tab to Round HSS Pipe

Beam/ Round HSS Column Welded Shear Tab

This problem was adapted from example K.8 in the *AISC Design Examples version 16.0*. The capacity values in RISACONNECTION are compared to those from the published example.

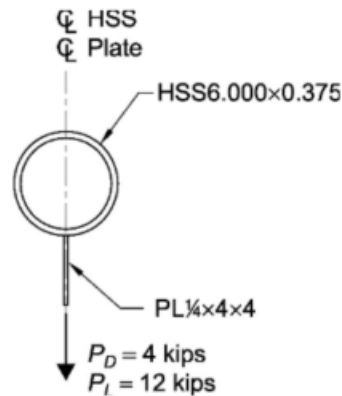


Figure 3.4- AISC Design Example K.8 Information

Comparison

Capacity Comparison			
Limit State	RISACONNECTION	AISC Example	% Difference
HSS Transverse Plastification	21.53 kips	23.4 kips	8.69
HSS Wall Slenderness	17.19	17.2	0.06
Material Strength	50 ksi	50 ksi	0.0

Table 3.4 – Capacity Comparisons

Conclusion

In this example it is shown that the RISACONNECTION calculations mostly 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.

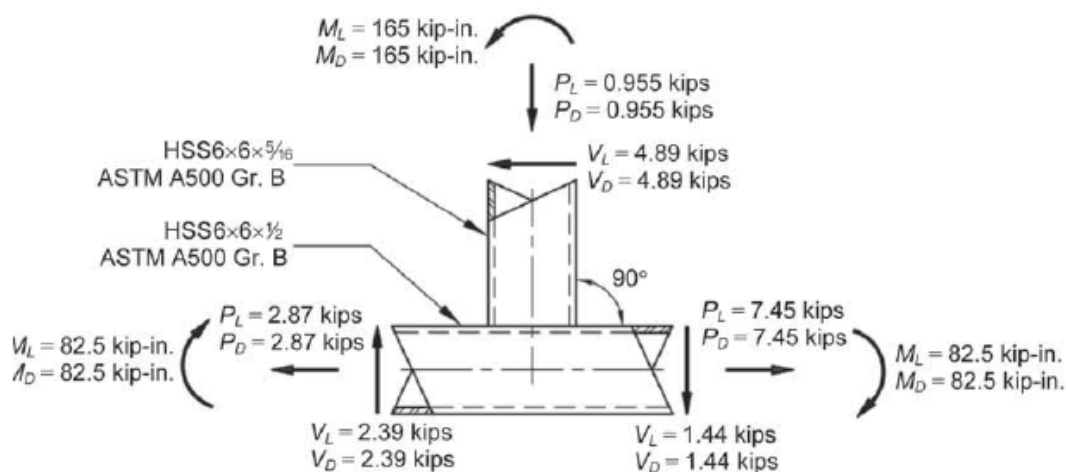


Figure 3.7- AISC Design Guide 24 Example 9.2 Information

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- 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.

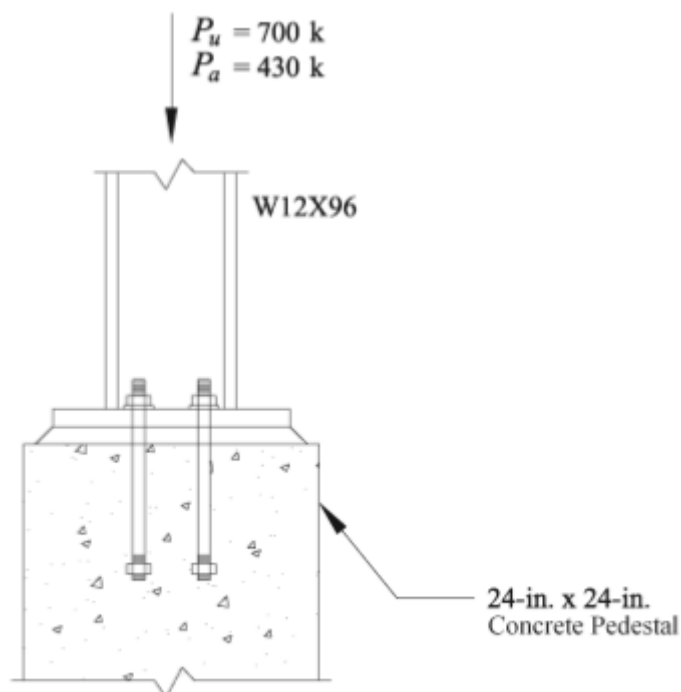


Figure 4.1- AISC Design Guide 1 Example 4.1 Information

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	532 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)	0.81	0.96	n/a ⁴
λ (per Plate Flexural Yielding)	1.00	1.00	0.00
$\lambda n'$ (per Plate Flexural Yielding)	3.11 in	3.11 in	0.00

Table 4.1 – Capacity and Geometry Comparisons

¹The axial compressive strength of the concrete can be calculated as the available bearing stress (output in RISACONNECTION) multiplied by the area $A_1 = 1.209 \text{ ksi} * 440 \text{ in}^2 = 532 \text{ kips}$.

²The Design Guide uses $\Omega = 2.5$, however RISACONNECTION uses $\Omega = 2.31$ which comes directly from AISC 360-10 section J8.

³The 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.

⁴The 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 4.2- 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.

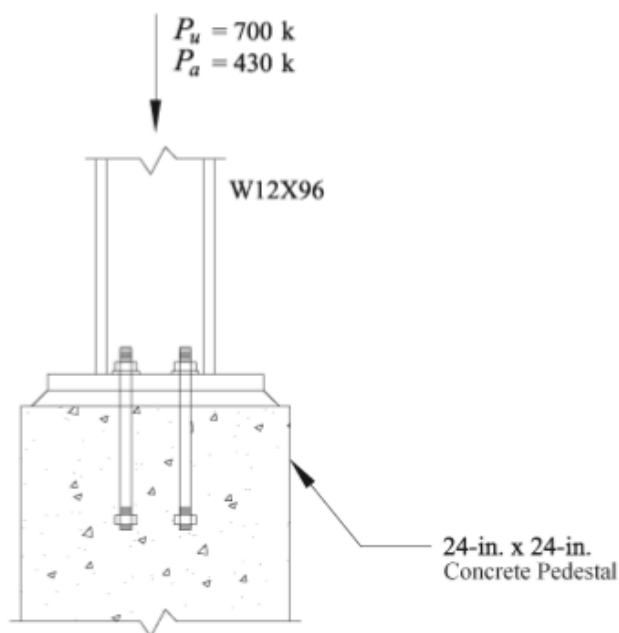


Figure 4.2- AISC Design Guide 1 Example 4.2 Information

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)	528 in ²	518 in ²	1.93
Concrete Bearing	482 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)	0.89	0.98	n/a^3
λ (per Plate Flexural Yielding)	1.00	1.00	0.00
$\lambda n'$ (per Plate Flexural Yielding)	3.11 in	3.11 in	0.00

Table 4.2 – Capacity and Geometry Comparisons

¹The axial compressive strength of the concrete can be calculated as the available bearing stress (output in RISACONNECTION) multiplied by the area $A_1 = 1.34 \text{ ksi} * 360 \text{ in}^2 = 482 \text{ kips}$.

²The Design Guide uses $\Omega = 2.5$, however RISACONNECTION uses $\Omega = 2.31$ which comes directly from AISC 360-10 section J8.

³The 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 4.3- 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 4.3 – Capacity Comparison

Conclusion

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

Connection 4.4- 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.

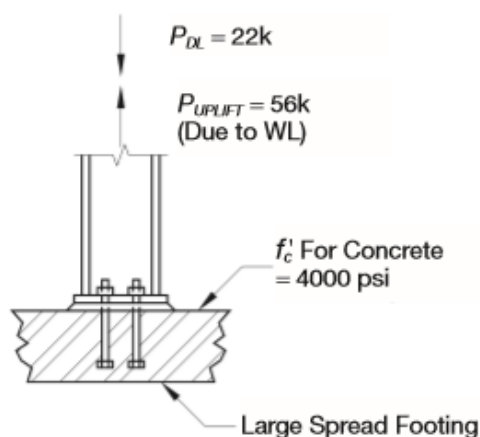


Figure 4.4- AISC Design Guide 1 Example 4.5 Information

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	21.56 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 4.4 – Capacity Comparisons

¹The 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.

²The 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 4.5- 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.

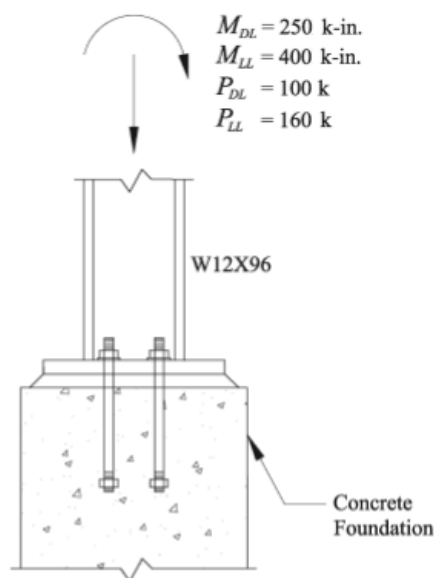


Figure 4.5- AISC Design Guide 1 Example 4.6 Information

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 4.5 – Capacity and Geometry Comparisons

¹ e_{crit} is calculated using q_{max} which depends on the Concrete Bearing unit check value. The Design Guide assumes $\Omega = 2.5$ for Concrete Bearing, but RISACONNECTION uses $\Omega = 2.31$ 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 4.6- 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.

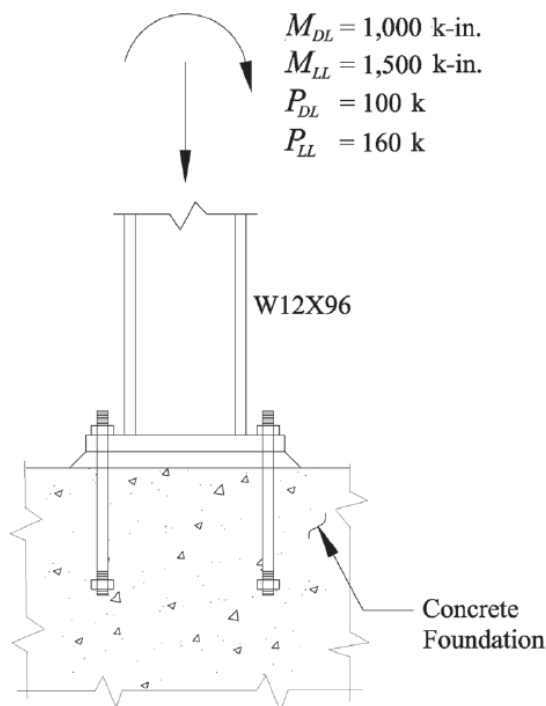


Figure 4.6- AISC Design Guide 1 Example 4.7 Information

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)	6.99 in	6.65 in	5.11 ¹
n (per Plate Flexural Yielding Strong Axis)	6.12 in	6.12 in	0.00

Table 4.6 – Capacity and Geometry Comparisons

¹e_{crit} is calculated using q_{max} with depends on the Concrete Bearing unity check value. The Design Guide assumes $\Omega = 2.5$ for Concrete Bearing, but RISACONNECTION uses $\Omega = 2.31$ 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 4.7- 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 360-22 (16th 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 4.7 – Capacity Comparison

¹This is a strength level (LRFD) result.

²The 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 4.8- 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.

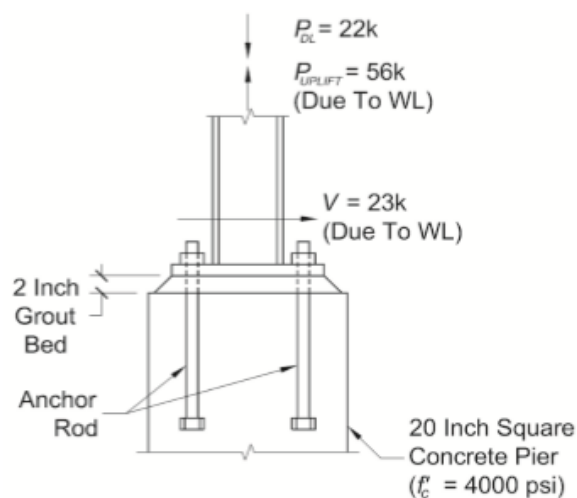


Figure 4.8 – AISC Design Guide 1 Example 4.11 Information

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

¹The Design Guide calculates $F_{nv} = 0.4 \cdot F_u$. The 0.4 factor comes from the AISC 360-05 (13th edition) code. RISACONNECTION defaults to use the AISC 360-22 (16th edition) code which calculates $F_{nv} = 0.45 \cdot F_u$.

²Anchor 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 5.1- 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.

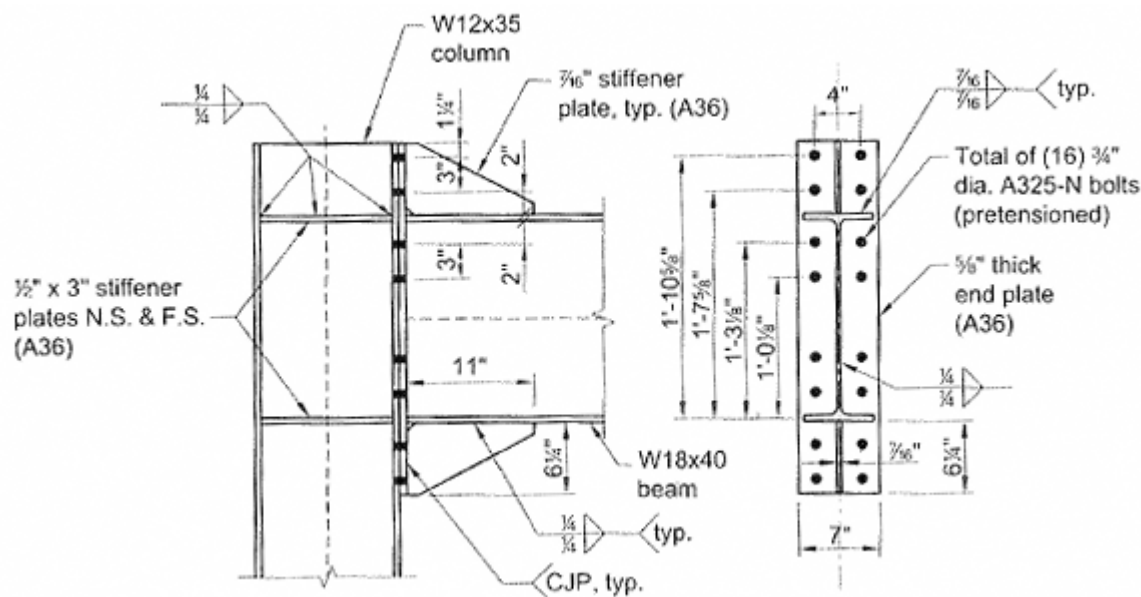


Figure 5.1 – AISC Seismic Design Manual Example 4.2.4 Information

Comparison

Results Comparison			
Limit State/Variable	RISACONNECTION	AISC Design Example	% Difference
M_{pr} (per Seismic Moment at Face of Column)	3162.13 kip-in	$4,740 / 1.5 = 3,160$ kip-in	0.07
Y_p (per End Plate Flexural Yielding)	110.71 in	110 in	0.65
Bolt Shear Strength	84.82 kips	84.8 kips	0.02
Y_c (per Column Flexural Yielding)	194.32 in	194 in	0.16
R_{cf}/Ω (per Column Flexural Yielding) ¹	$137.16 \text{ kip} \times 17.38 \text{ in} = 2,383.8 \text{ kip-in}$	2,380 kip-in	0.16
Column Panel Zone Shear	$91.05 \text{ kips} \times 1.67 \times 1.1 \times 1.1 = 184 \text{ kips}$	184 kips	0.0

Table 6.1 – Capacity and Geometry Comparisons

¹The design example presents the strength as a moment. The RISACONNECTION force is multiplied by the moment arm to compare the value to the AISC Design Example.

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