Gelinas Structural ENGINEERING LLC

Daniel L. Gelinas, MS, PE, SECB
Phone: 978-465-6436
Email: danlg@gelinasstructural.com

579A North End Blvd. | Salisbury, MA 01952-1738 | www.gelinasstructural.com

November 05, 2025

REPORT:
38 Old Colony Avenue
Quincy, MA.
Beam to Beam, Beam to Column & Beam to Wall Connections,
Job – xxxx, Drawings: - Sheet S-001 thru S-303
(GSE Job No - 25743)

Prepared for:
D Cronin's Welding Service, Inc.
70 State Street
Lawrence, MA 01843

Prepared by:
Gelinas Structural Engineering, LLC σ
Daniel L. Gelinas, P.E.
579A North End Blvd.
Salisbury, MA 01952-1738
Phone : 978.465.643 Danlg@GelinasStructural.com

Scope of Project
Design of Beam to Beam, Beam to Column & Beam to Wall Connections @ 38 Old Colony Avenue, Quincy, MA.

Design Criteria
For Connection design, ASCE 7-16, AISC 360-16 edition, IBC 2021 as amended by the Massachusetts State Building Code 10th edition, and the AISC 15th edition manual have been used.

All steel connections have been designed for the ASD Loads given on sheet S-101

Materials
Structural steel shall be designed in accordance with the 'Specification for structural steel buildings' & American Society welding society (AWS D1.1) ''Structural Welding Code - Steel". AISC 15th edition. All Bars, Steel plates, Channels, MC Sections, and angles of grade ASTM A36 shall be used. All W sections ASTM 992 50 ksi shall be used. All HSS sections of grade ASTM A 500 – Grade C shall be used. The high-strength bolts of A325N or A490N shall be used for Shear & Moment connections.

Table of Contents

10) Shear Connection – W18x_ Beam connecting to HSS6x6x_ Column using 1/2" Thick Shear plate with (10) - 1" ϕ A490N Bolts & 5/16" thick weld - For 114 kip Shear – **NO GOOD**

10A) Shear Connection – W18x_ Beam connecting to HSS6x6x_ Column using 3/4" Thick Shear plate With (10) - 1" ϕ A490N Bolts & 3/8" thick weld - For 114 kip Shear - **REVISED**

11) Moment Connection – W14x22 Beam connecting to HSS5x5x1/4 Column using directly flange welded moment connection - For 18 kip-ft moment

12) Moment Connection – W18x46 Beam connecting to HSS5x5x1/4 Column using directly flange welded moment connection - For 23 kip-ft moment

20) Bearing Connection – W14x43 Beam braced with HSS5x5x_ Column using 3/4" Thick plate with (4) - ¾" ϕ A325N Bolts & 5/16" thick weld - For 19 kip Shear.

A) Key plan – Beam to Beam, Beam to Column & Beam to Wall Connections

Refer to next page

WALL NOTES:

1. ALL INTERIOR WALLS SHOWN ARE 2x6 (W2) SEE BEARING WALL TYPE SCHEDULE ON S-006 FOR ADDITIONAL INFORMATION.
2. ALL EXTERIOR WALLS SHOWN ARE 2x6 (W1) w/ ½" PLYWOOD SHEATHING. SHEATHING IS NAILED TO STUDS w/ 10d GALVANIZED COMMON NAILS SPACED @4" OC @PANEL EDGES & 12" OC @INTERMEDIATE SUPPORTS. SEE BEARING WALL SCHEDULE ON S-006 FOR ADDITIONAL INFORMATION.
3. SW-1 INDICATES 2x6 (W1) w/ ½" PLYWOOD SHEATHING ON ONE SIDE, SHEAR WALL SHEATHING IS NAILED TO STUDS w/ 10d GALVANIZED COMMON NAILS SPACED @4" OC @PANEL EDGES & 12" OC @ INTERMEDIATE SUPPORTS.

WALL NOTES CONT:

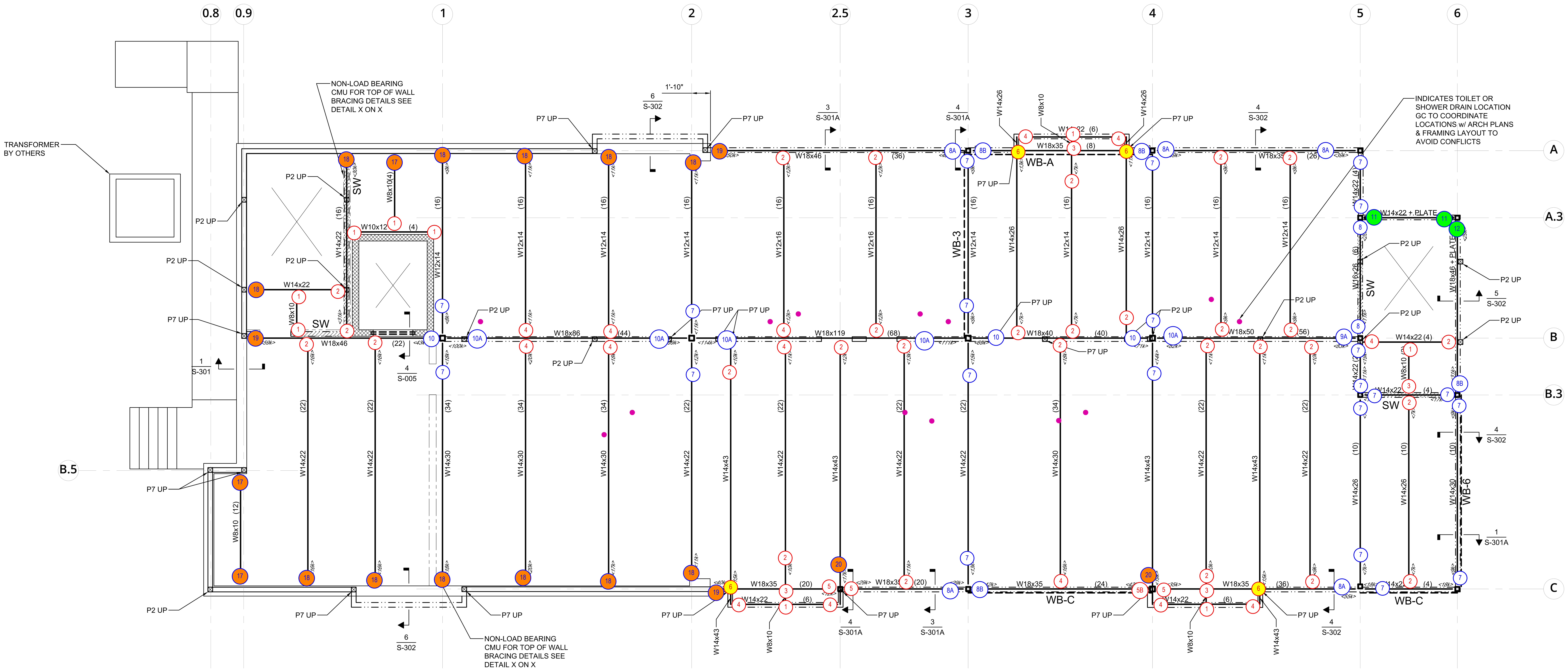
4. SW-2 INDICATES DOUBLE 2x6 (W1) w/ ½" PLYWOOD SHEATHING ON BOTH SIDES, SHEAR WALL SHEATHING IS NAILED TO STUDS w/ 10d GALVANIZED COMMON NAILS SPACED @4" OC @PANEL EDGES & 12" OC @INTERMEDIATE SUPPORTS.
5. HOLD-DOWNS:
   - AT EXTERIOR END OF SW, PROVIDE SIMPSON MST60 STRAP TIE; PROVIDE ADDITIONAL STUDS AS REQUIRED TO MEET SIMPSON CONNECTION REQUIREMENTS.
   - AT INTERIOR END OF SW, PROVIDE SIMPSON HDU5-SDS2.5 HOLD-DOWN; PROVIDE ADDITIONAL STUDS AS REQUIRED TO MEET SIMPSON CONNECTION REQUIREMENTS.

NOTES:

1. FOR GENERAL NOTES SEE S-001.
2. FOR SPECIAL TESTING & INSPECTION REQUIREMENTS SEE S-001A.
3. FOR TYPICAL DETAILS SEE S-002, S-003, S-004, S-005, S-006, & S-007.
4. INDICATES BEARING WALL STARTS ON BEAM.
5. S-1 INDICATES SPAN OF 2" - 20 GAGE GALVANIZED STEEL COMPOSITE FLOOR DECK w/ 3½" NWC TOPPING SLAB (TOTAL THICKNESS = 6½"). REINFORCE w/6x6-W2.9xW2.9 WWF.
6. WB-1 ETC INDICATES BRACING ELEVATION SEE S-201 FOR ELEVATION & DETAILS.
7. FOR COLUMN & BASE PL SIZES & DETAILS SEE S-202.
8. ALL DIMENSIONS SHOWN ARE FOR REFERENCE ONLY. SEE ARCHITECTURAL DRAWINGS FOR ALL DIMENSIONS AND TOP OF CONCRETE ELEVATIONS.
9. TOC EL 25'-5". TOS EL 24-10½".

POST SCHEDULE

MARK	TYPE
P1	2-2x6
P2	3-2x6
P3	4-2x6
P4	4x6
P5	6x6
P6	3½x3½ LVL
P7	3½x5¼ LVL
P8	5¼x5¼ LVL



LEGENDS

SYMBOL	DESCRIPTION
X (red circle)	BEAM TO BEAM SHEAR CONNECTION
X (blue circle)	BEAM TO COLUMN SHEAR CONNECTION
X (yellow circle)	BEAM TO BEAM MOMENT CONNECTION
X (green circle)	BEAM TO COLUMN MOMENT CONNECTION
X (orange circle)	BEAM TO COLUMN & WALL BEARING CONN.

Gelinas Comment

JOB NO : 25743
11-05-2025 GELINAS COMMENTS

COMMONWEALTH OF MASSACHUSETTS
CARMINE GUARRACINO
STRUCTURAL
NO. 40104
REGISTERED PROFESSIONAL ENGINEER

REVISIONS
FOUNDATION PERMIT SET

R&G
Roome & Guarracino LLC
STRUCTURAL ENGINEERS
300 TradeCenter, Suite 3540, Woburn, MA
T: 617-628-1700 F: 617-628-1711

DONAHUE Architects inc.
21 MCGRATH HIGHWAY, QUINCY, MASS. 02169
Phone: 1-617-479-4122

38 OLD COLONY AVENUE
QUINCY, MA.

FIRST FLOOR FRAMING PLAN

Checked by:	BSM
Drawn by:	CG
Date:	10.02.2025
Scale:	3/16" = 1'-0"

Drawing No.

S-101

B)Beam to Beam, Beam to Column & Beam to Wall Connections Summary

NOTES:

1) DL IN THE RAM CONNECT OUTPUT DENOTES "DESIGN LOADS"

2) ALL STRUCTURAL STEEL GRADE A992 (GR50 KSI), FOR WIDE FLANGE SHAPES, FOR PIPE SECTIONS ASTM A53 GR B & A500 GRADE-B (Fy=46) FOR HSS SECTIONS.

3) CONNECTION MATERIAL GRADE : ANGLES AND PLATE (A36). U,N.O.

4) ALL BOLTS ARE A325-N & A490-N BOLTS (U.N.O)

5) WELDING ELECTRODES SHALL CONFORM TO ASTM SPECIFICATION E-70XX.

6) DESIGN METHOD - ASD PER AISC 15TH EDITION (AISC 360 - 16)

BEAM TO BEAM/COLUMN - SHEAR & BRACE CONNECTIONS SUMMARY

CONN. ID	BEAM SIZE	BEAM OR COLUMN SIZE	TYPE OF CONNECTION	BOLT DESCRIPTION	SHEAR PLATE/ANGLE & WELD THICKNESS	REACTIONS PER SHEET - S101	CONNECTION CAPACITY @ 0.99 UTILIZATION RATIO (KIPS)	REMARKS
1	W8X10 / W10X12	W10X12 / W12X14 W14X22 / W18X46 BEAM	SHEAR CONNECTION	(2) - 3/4" Ø A325N	L3 ½" x 3 ½" x 5/16" CLIP WITH 1/4" THICK WELD	-	33 KIPS	
2	W12X_ / W14X_	W14X_/ W18X_	SHEAR CONNECTION	(3) - 3/4" Ø A325SC	L3 ½" x 3 ½" x 5/16" CLIP WITH 1/4" THICK WELD	31	37 KIPS	
3	W8X10	W14X22 / W18X35 BEAM	SHEAR CONNECTION	(2) - 3/4" Ø A325N	3/8" THICK SHEAR PLATE WITH 1/4" THICK WELD	-	13 KIPS	
4	W12X_ / W14X_	W14X_/ W18X_	SHEAR CONNECTION	(3) - 3/4" Ø A325N	3/8" THICK SHEAR PLATE WITH 1/4" THICK WELD	20	30 KIPS	
5	W18X35	W14X43 BEAM	SHEAR CONNECTION	(4) - 3/4" Ø A325N	3/8" THICK SHEAR PLATE WITH 1/4" THICK WELD	42	42 KIPS	
5A	W18X35	W14X43 BEAM	SHEAR CONNECTION	(4) - 3/4" Ø A325SC	3/8" THICK SHEAR PLATE WITH 1/4" THICK WELD	42		•3/4" Ø A325SC BOLTS FAIL IN SHEAR •(4) - 3/4" Ø A325SC BOLTS CAN CARRY UPTO 22KIPS
5B	**W18X35**	**W14X43 BEAM**	**SHEAR CONNECTION**	**(4) - 1" Ø A490SC**	**3/8" THICK SHEAR PLATE WITH 1/4" THICK WELD**	**42**		**•REVISED CONNECTION WITH (4) - 1" Ø A490SC BOLTS •1/4" THICK DOUBLER PLATE IS ADEQUATE FOR THIS CONNECTION**
6	W14X26 / W14X43	W18X35 BEAM	MOMENT CONNECTION		5/8" THICK FLANGE PLATES @ TOP & BOTTOM WITH 5/16" WELD	-	138 KIP-FT	REFER TO **CONNECTION ID-2** FOR SHEAR CONNECTION DETAILS
7	W12X_ / W14X_	HSS5X5X_ / HSS6X6X_ COLUMN	SHEAR CONNECTION	(3) - 3/4" Ø A325SC	3/8" THICK SHEAR PLATE WITH 1/4" THICK WELD	19	16KIP	•10% OVERAGE ALLOWANCE IS CONSIDERED FOR THIS CONNECTION
8	W16X26	HSS5X5X1/4 / HSS5X5X3/8 COLUMN	SHEAR CONNECTION	(4) - 3/4" Ø A325N	3/8" THICK SHEAR PLATE WITH 1/4" THICK WELD	20	43 KIPS	
8A	**W18X35 W18X46**	**HSS5X5X1/4 / HSS5X5X3/8 HSS5X5X1/2 COLUMN**	**SHEAR CONNECTION**	**(5) - 3/4" Ø A325N**	**3/8" THICK SHEAR PLATE WITH 1/4" THICK WELD**	**55**	**55 KIPS**	**REVISED CONNECTION WITH (5) - 3/4" Ø A325N BOLTS**
8B	**W18X35 W18X46**	**HSS5X5X1/4 / HSS5X5X3/8 HSS5X5X1/2 COLUMN**	**SHEAR CONNECTION**	**(5) - 1" Ø A490SC**	**3/8" THICK SHEAR PLATE WITH 1/4" THICK WELD**	**55**	**55 KIPS**	**REVISED CONNECTION WITH (5) - 1" Ø A490SC BOLTS**
9	W18X50	HSS5X5X3/8 COLUMN	SHEAR CONNECTION	(10) - 1" Ø A490N	1/2" THICK SHEAR THROUGH PLATE WITH 5/16" THICK WELD	83	69KIPS	• 1/2" THICK PLATE FAILS IN RUPTURE • 1/2" THICK PLATE CAN CARRY UPTO 69 KIPS ONLY
9A	**W18X50**	**HSS5X5X3/8 COLUMN**	**SHEAR CONNECTION**	**(10) - 1" Ø A490N**	**3/4" THICK SHEAR THROUGH PLATE WITH 3/8" THICK WELD**	**83**	**83 KIPS**	**• REVISED CONNECTION WITH 3/4" THICK PLATE WITH 3/8" WELD**
10	W18X40 / W18X46 W18X50 / W18X86 W18X119	HSS6X6X1/2 / HSS6X6X5/8 COLUMN	SHEAR CONNECTION	(10) - 1" Ø A490N	1/2" THICK SHEAR THROUGH PLATE WITH 5/16" THICK WELD	114	73KIPS	• 1/2" THICK PLATE FAILS IN RUPTURE • 1/2" THICK PLATE CAN CARRY UPTO 73 KIPS ONLY
10A	**W18X40 / W18X46 W18X50 / W18X86 W18X119**	**HSS6X6X1/2 / HSS6X6X5/8 COLUMN**	**SHEAR CONNECTION**	**(10) - 1" Ø A490N**	**3/4" THICK SHEAR THROUGH PLATE WITH 3/8" THICK WELD**	**114**	**110 KIPS**	**• REVISED CONNECTION WITH 3/4" THICK PLATE WITH 3/8" WELD •10% OVERAGE ALLOWANCE IS CONSIDERED FOR THIS CONNECTION**
11	W14X22	HSS5X5X1/4	MOMENT CONNECTION		DIRECTLY FLANGE WELDED CONNECTION		18 KIP-ft	REFER TO **CONNECTION ID-7** FOR SHEAR CONNECTION DETAILS
12	W18X46	HSS5X5X1/4	MOMENT CONNECTION		DIRECTLY FLANGE WELDED CONNECTION		23 KIP-ft	REFER TO **CONNECTION ID-8A** FOR SHEAR CONNECTION DETAILS
17	W8X10	WALL	SHEAR CONNECTION	(2) - 3/4" Ø A325N	WT6X20 WITH 1/4 " THICK WELD	-	13 KIPS	REFER TO **CONNECTION ID-3** FOR SHEAR CONNECTION DETAILS
18	W12X14 W14X22 / W14X30	WALL	SHEAR CONNECTION	(3) - 3/4" Ø A325N	WT6X20 WITH 1/4 " THICK WELD	30		REFER TO **CONNECTION ID-4** FOR SHEAR CONNECTION DETAILS
19	W18X35 / W18X46	WALL	SHEAR CONNECTION	(5) - 3/4" Ø A325N	L4" x 3 " x 3/8" CLIP WITH 1/4" THICK WELD	59		REFER TO **CONNECTION ID-8A** FOR SHEAR CONNECTION DETAILS, HENCE THIS CONNECTION WILL BE ADEQUATE FOR 59 KIPS
20	W14X43	HSS5X5X3/8 HSS5X5X1/2	BEARING CONNECTION	(4) - 3/4" Ø A325N	3/4" THICK PLATE WITH 5/16 " THICK WELD	19		

NOTE: - ALL THE CONNECTIONS HAVE BEEN DESIGNED FOR THE LOADS GIVEN ON SHEETS S101

NOTE: -

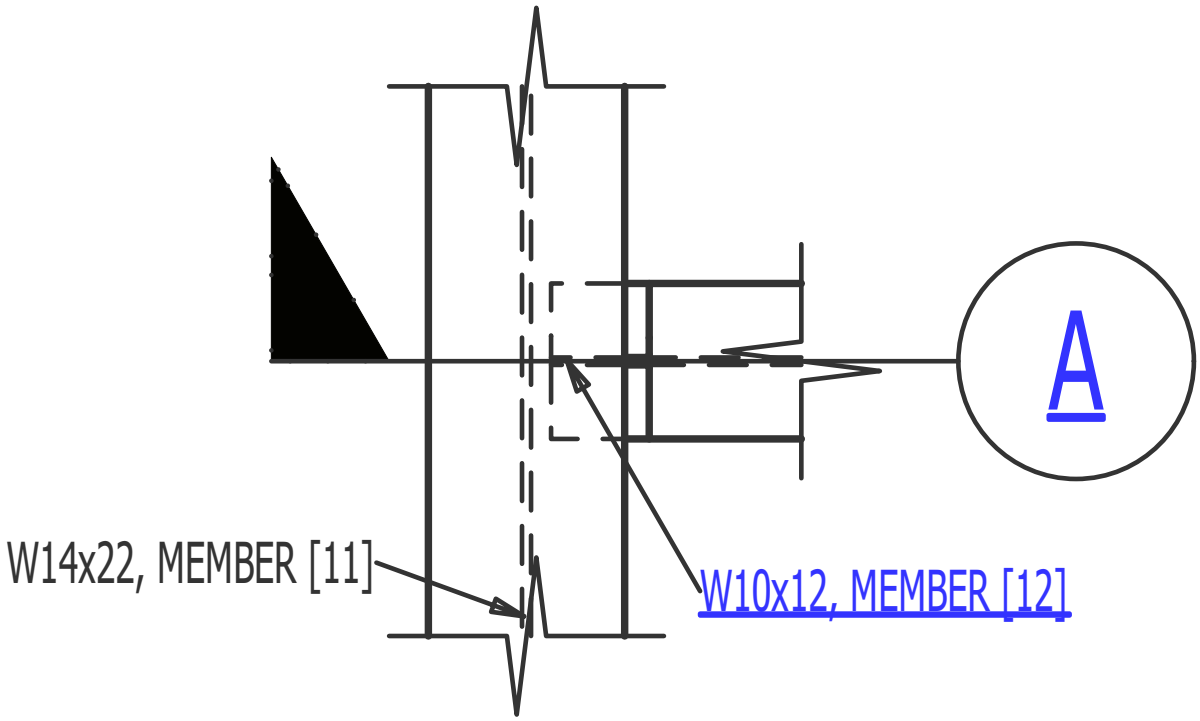
RED TEXT CONNECTIONS ARE NO GOOD.

BLUE TEXT CONNECTIONS ARE REVISION FOR RESPECTIVE CONNECTIONS.

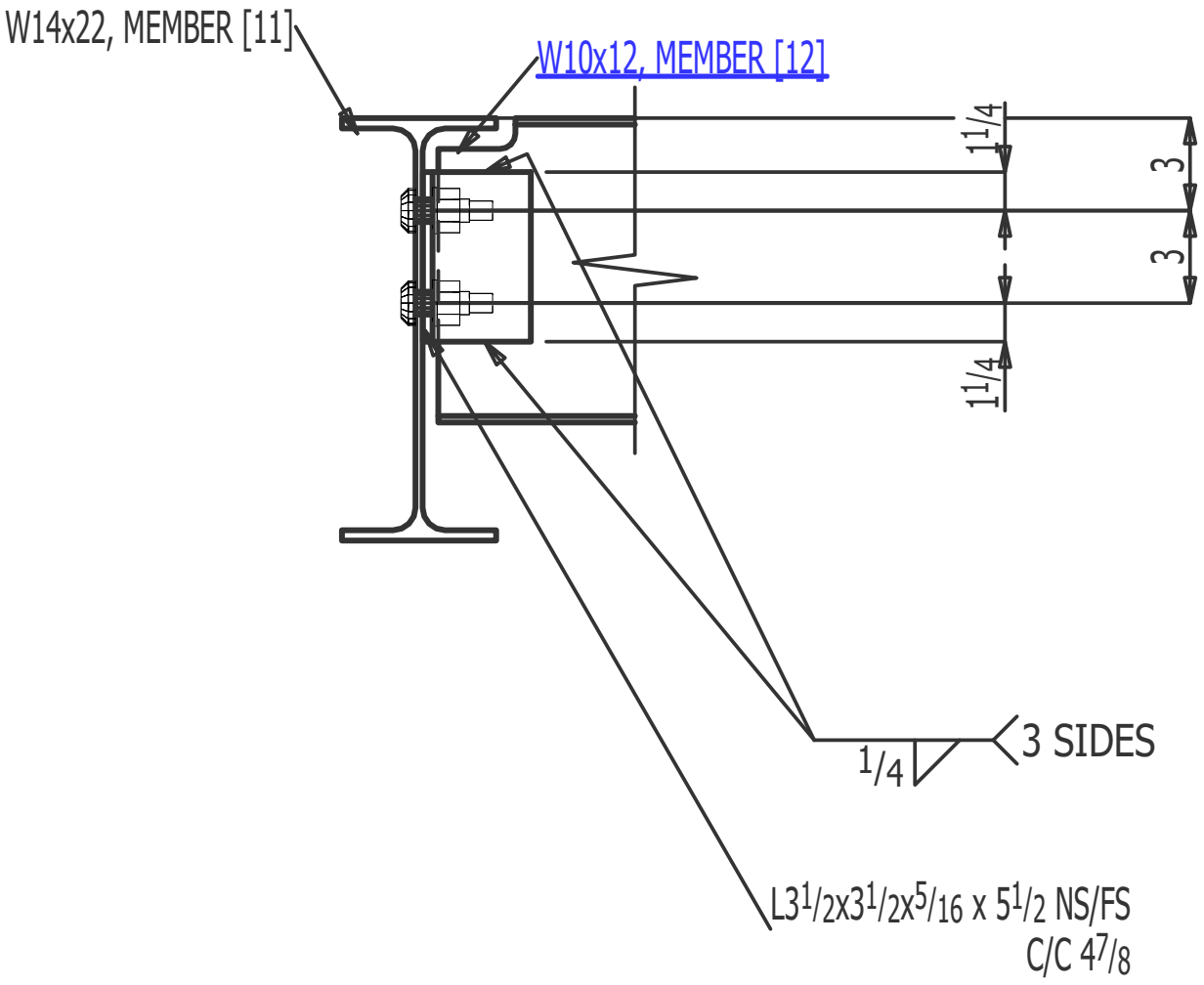
1) Shear Connection – W8x10 & W10x12 Beam connecting to W10x12, W12x14, W14x22 & W18x46 Beam using L 3 1/2x3 1/2x5/16 clip angle with (2) - ¾" ϕ A325N Bolts & 1/4" thick weld - For 33 kip Shear

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 1

CON - 1 [1] at X=10, Y=6-0 Elev=24-5 1/16



TOP SIDE VIEW



A

Section A
ELEVATION

Beam B_11 [12]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W10x12
Sequence:	1
ABM:	N/Assign
Plan length:	4-8
Camber:	0.00 in
Span length:	4-8
Slope:	0.00 °
Material length:	4-7 3/8
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	9.87 in
Web thickness, t_w:	0.19 in
Flange width, b_f:	3.96 in
Flange thickness, t_f:	0.21 in
Design k distance, k_{des}:	0.51 in
Detail k distance, k_{det}:	0.75 in
Distance between web toes of fillets, T:	8.37 in
Moment of inertia about the major axis, I_x:	53.8 in^4

Design summary

Left end

Connection:	Web cleat angle
	Welded Web, Bolted OSL, Wide gauge
	Welded extended tee: No
	Full depth extended tee: No
	Non-Safety, Create Web Doublers: Never
	Use web extension: Never
	Replace doubler: No
	Skew holes in angle: Yes
	Combine web cleat angles: No
	Expand Vertical bolt spacing: No
	Both sides, Supported
Elevation:	24-10
Field Clear:	0 in
Mtrl Setback:	0.625 in
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	33.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_11 [12] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_12 [11]
Section size:	W14x22
End 0 elevation:	24-10
End 1 elevation:	24-10
Support intersection elevation:	24-10
Supporting beam rotation:	0.00 degrees
	(looking toward left end)
Material grade:	A992
Detail k distance, k_{det}:	1.0625 in
Design k distance, k_{des}:	0.735 in
Supporting member thickness, t_{sup}:	0.23 in

Design loads

***Shear*:** 33.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 88.0 % of the maximum web shear.

Connection summary

- STANDARD CLIP ANGLE, BOLTED/WELDED, SHOP WELDED TO SUPPORTED MEMBER
- TWO-SIDE CONNECTION

Connection details

Angles:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	NS angle section size:	L3 1/2x3 1/2x5/16 x 5 1/2
	NS outstanding leg length, $l_{osl,ns}$:	3.5 in
	FS angle section size:	L3 1/2x3 1/2x5/16 x 5 1/2
	FS outstanding leg length, $l_{osl,fs}$:	3.5 in
Weld to supported:	**Weld leg size, w:**	0.25 in
	Weld metal strength, F_{exx}:	70 ksi
	Effective weld size, w_e:	0.124773 in
Bolts to supporting:	**Bolt type:**	A325N
	Tension control field bolts	
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	2
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Bolt gauge, g:	4.875 in
	OSL row edge distance, $L_{ev,osl}$:	1.25 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	5
Locked Via User Defined:	27

Results summary

Web Cleat Angle on left end of Beam B_11 [12]

Connection ductility check

Min. OSL bolt diameter to preclude bolt fracture: 0.34 in

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Rupture of web weld:	26	0.988	J2, Table J2.5
Shear yielding of beam web:	5	0.979	G2.1
Block shear rupture of beam web:	7	0.909	J4.3
Block shear rupture of OSL(s):	252	0.813	J4.3
Shear rupture of angle(s):	21	0.809	J4.2
Bolt bearing on OSL:	110	0.782	J3.10
Bolt shear of OSL bolts:	3	0.692	J3
Bolt bearing on supporting member:	110	0.692	J3.10
Shear yielding of angle(s):	15	0.667	J4.2
Flexure of coped beam:	41	0.616	Pg 9-6

Connection strength

	Unity ratio:
Shear:	0.988

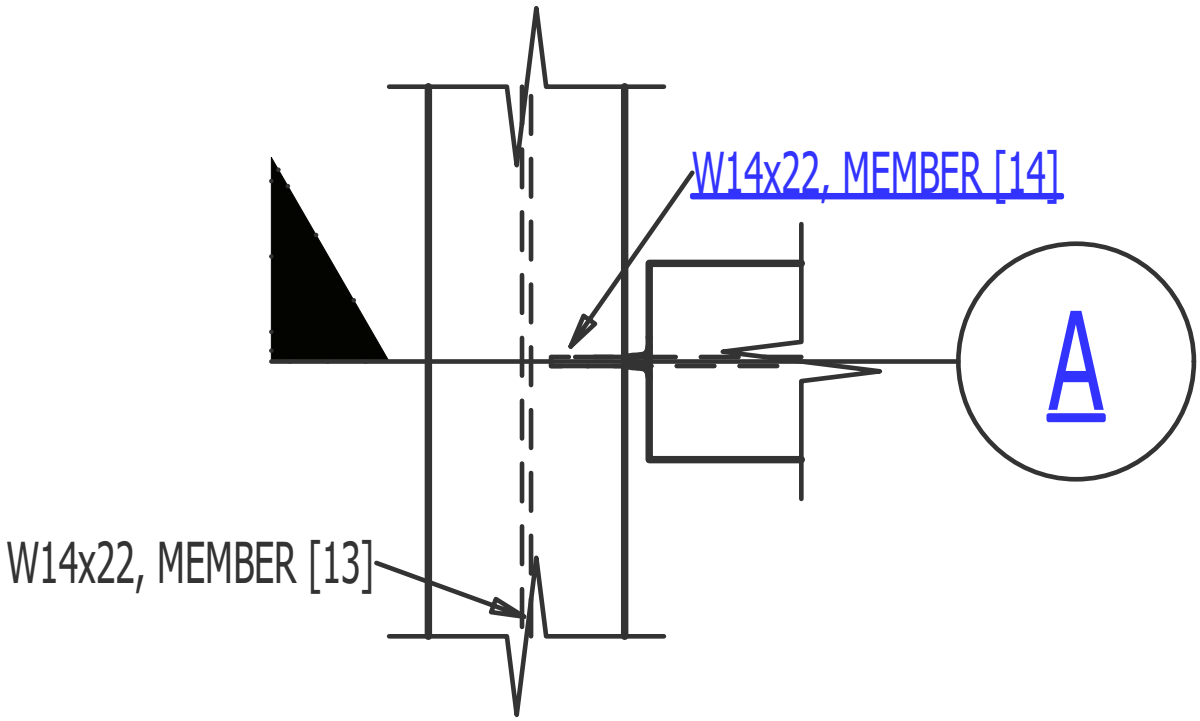
Notes and conclusions

- Maximum unity ratio for design: 0.99
- Design for web doublers is turned off.
- Weld design includes the effect of eccentricity.
- CONNECTION IS OK
  - Strength equals or exceeds design loads.

2) Shear Connection – W12x_ & W14x_ Beam connecting to W14x_ & W18x_ Beam using
L 3 1/2x3 1/2x 5/16 clip angle with (3) - ¾" ϕ A325SC Bolts & 1/4" thick weld - For 31 kip Shear

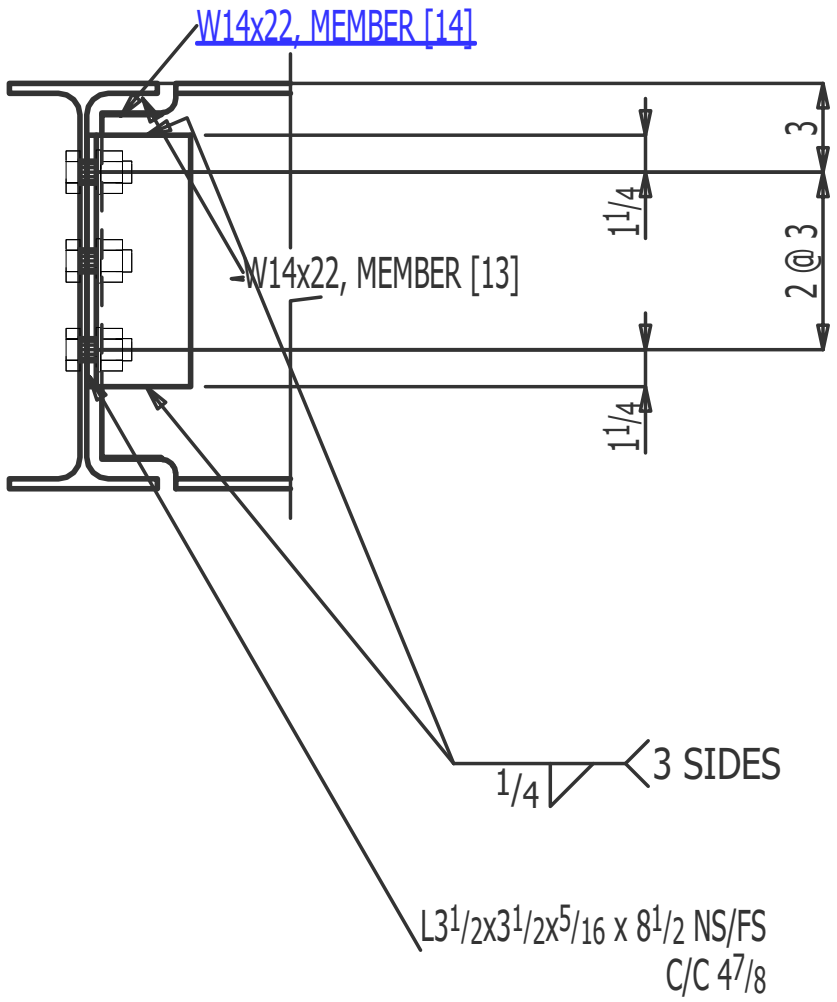
Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON -2

CON -2 [2] at X=10, Y=18-7 1/4 Elev=24-3 1/8



TOP SIDE VIEW

CON -2



A

Section A
CON -2 ELEVATION

Beam B_14 [14]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W14x22
Sequence:	1
ABM:	N/Assign
Plan length:	4-8
Camber:	0.00 in
Span length:	4-8
Slope:	0.00 °
Material length:	3-10
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	13.7 in
Web thickness, t_w:	0.23 in
Flange width, b_f:	5 in
Flange thickness, t_f:	0.335 in
Design k distance, k_{des}:	0.735 in
Detail k distance, k_{det}:	1.0625 in
Distance between web toes of fillets, T:	11.575 in
Moment of inertia about the major axis, I_x:	199 in^4

Design summary

Left end

Connection:	Web cleat angle
	Welded Web, Bolted OSL, Wide gauge
	Welded extended tee: No
	Full depth extended tee: No
	Non-Safety, Create Web Doublers: Never
	Use web extension: Never
	Replace doubler: No
	Skew holes in angle: Yes
	Combine web cleat angles: No
	Expand Vertical bolt spacing: No
	Both sides, Supported
Elevation:	24-10
Field Clear:	0 in
Mtrl Setback:	0.625 in
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	31.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_14 [14] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_13 [13]
Section size:	W14x22
End 0 elevation:	24-10
End 1 elevation:	24-10
Support intersection elevation:	24-10
Supporting beam rotation:	0.00 degrees
	(looking toward left end)
Material grade:	A992
Detail k distance, k_{det}:	1.0625 in
Design k distance, k_{des}:	0.735 in
Supporting member thickness, t_{sup}:	0.23 in

Design loads

***Shear*:** 31.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 49.2 % of the maximum web shear.

Connection summary

- STANDARD CLIP ANGLE, BOLTED/WELDED, SHOP WELDED TO SUPPORTED MEMBER
- TWO-SIDE CONNECTION

Connection details

Angles:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	NS angle section size:	L3 1/2x3 1/2x5/16 x 8 1/2
	NS outstanding leg length, $l_{osl,ns}$:	3.5 in
	FS angle section size:	L3 1/2x3 1/2x5/16 x 8 1/2
	FS outstanding leg length, $l_{osl,fs}$:	3.5 in
Weld to supported:	**Weld leg size, w:**	0.25 in
	Weld metal strength, F_{exx}:	70 ksi
	Effective weld size, w_e:	0.151041 in
Bolts to supporting:	**Bolt type:**	A325SC
	Surface:	Class A
	Mean slip coefficient:	0.30 (J3.8)
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	3
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Bolt gauge, g:	4.875 in
	OSL row edge distance, $L_{ev,osl}$:	1.25 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	7
Locked Via User Defined:	27

Results summary

Web Cleat Angle on left end of Beam B_14 [14]

Connection ductility check

Min. OSL bolt diameter to preclude bolt fracture: 0.34 in

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Bolt shear of OSL bolts:	3	0.816	J3
Bolt bearing on OSL:	110	0.816	J3.10
Bolt bearing on supporting member:	110	0.816	J3.10
Shear rupture of beam web:	4	0.590	J4.2
Shear yielding of beam web:	5	0.576	G2.1
Block shear rupture of beam web:	7	0.571	J4.3
Rupture of web weld:	26	0.525	J2, Table J2.5
Block shear rupture of OSL(s):	252	0.510	J4.3
Shear rupture of angle(s):	21	0.485	J4.2
Shear yielding of angle(s):	15	0.405	J4.2
Flexure of coped beam:	41	0.394	Pg 9-6

Connection strength

	Unity ratio:
Shear:	0.816

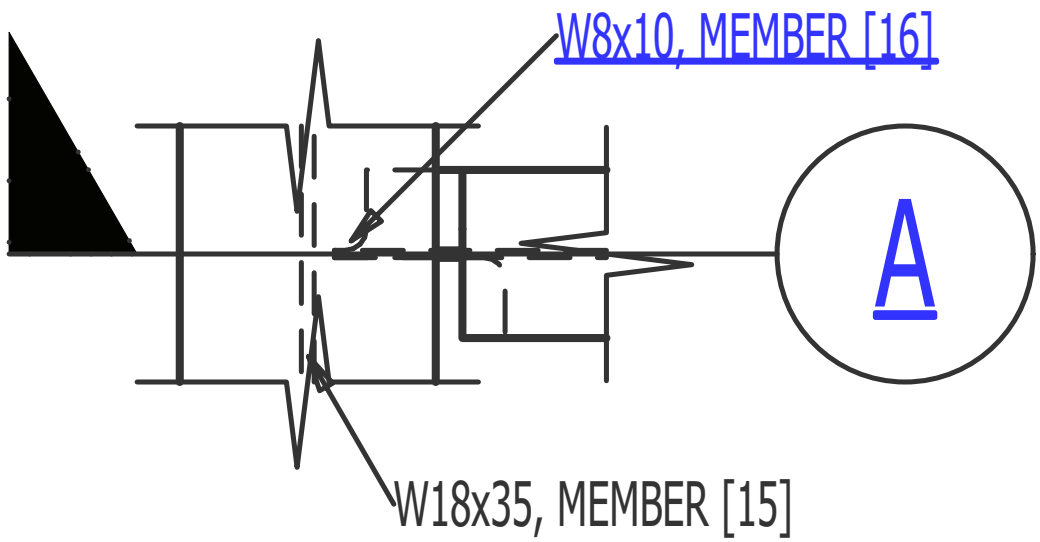
Notes and conclusions

- Maximum unity ratio for design: 0.99
- Design for web doublers is turned off.
- Weld design includes the effect of eccentricity.
- CONNECTION IS OK
  - Strength equals or exceeds design loads.

3) Shear Connection – W8x10 Beam connecting to W14x22 & W18x35 Beam using 3/8" Thick Shear plate with (2) - ¾" ϕ A325N Bolts & 1/4" thick weld - For 13 kip Shear

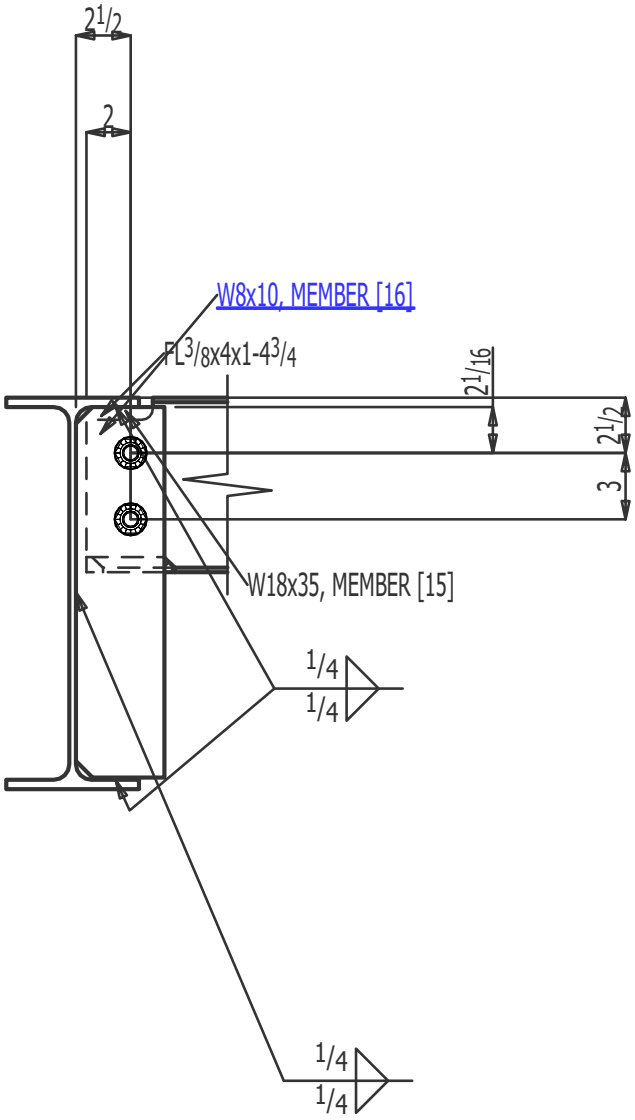
Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 3

CON - 3 [6] at X=5-6, Y=10-0 3/4 Elev=24-6 1/16



TOP SIDE VIEW

CON - 3



A

Section A
CON - 3 ELEVATION

Beam B_16 [16]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W8x10
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-4
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	7.89 in
Web thickness, t_w:	0.17 in
Flange width, b_f:	3.94 in
Flange thickness, t_f:	0.205 in
Design k distance, k_{des}:	0.505 in
Detail k distance, k_{det}:	0.6875 in
Distance between web toes of fillets, T:	6.515 in
Moment of inertia about the major axis, I_x:	30.8 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Extend to both Flanges
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: Automatic
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	0.5 in
Mtrl Setback:	0.625 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	13.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_16 [16] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_15 [15]
Section size:	W18x35
End 0 elevation:	24-10
End 1 elevation:	24-10
Support intersection elevation:	24-10
Supporting beam rotation:	0.00 degrees
	(looking toward left end)
Material grade:	A992
Detail k distance, k_{det}:	1.125 in
Design k distance, k_{des}:	0.827 in
Supporting member thickness, t_{sup}:	0.3 in

Design loads

***Shear*:** 13.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 48.5 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Width, b:	4 in
	Depth, d:	16.75 in
Shear tab extension:	To top & bottom flange	
	Weld line to bolt group c.g., a:	2.475 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.25 in
	Total effective weld throat, t_e:	0.35 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A325N
	Tension control field bolts	
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	2
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 21

Results summary

Web Side Plate on left end of Beam B_16 [16]

Bolt capacity notes

- It is mandatory that SC joints also meet the strength requirements to resist the factored load as a bearing bolt, type N, joint; RCSC section 5.4 commentary

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Flexure of coped beam:	41	0.963	Pg 9-6
Shear rupture of beam web:	4	0.765	J4.2
Bolt bearing on beam web:	110	0.756	J3.10
Bolt shear of web bolts:	3	0.718	J3
Block shear rupture of beam web:	6	0.677	J4.3
Shear yielding of beam web:	5	0.556	G2.1
Bolt bearing on plate:	110	0.544	J3.10
Block shear rupture of plate:	6	0.256	J4.3
Shear yielding of plate:	38	0.144	J4.2
Shear rupture of plate:	21	0.133	J4.2
Shear of support:	36	0.066	J4.2
Flexure of plate:	314	0.028	Pg 10-90

Connection strength

	Unity ratio:
Shear:	0.963

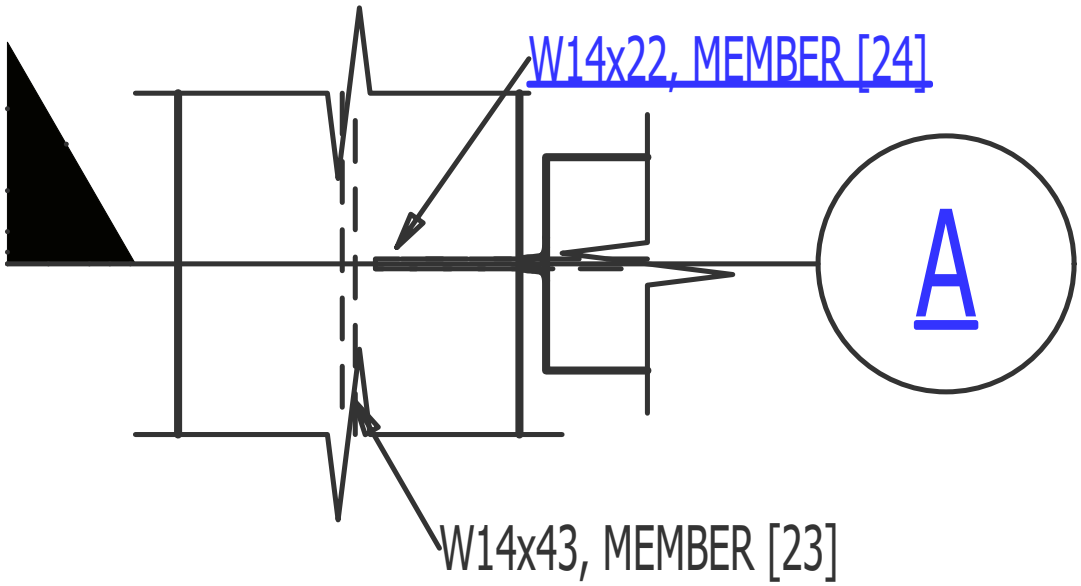
Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- CONNECTION IS OK
  - Strength equals or exceeds design loads.

4) Shear Connection – W12x_ & W14x_ Beam connecting to W14x_ & W18x_ Beam using 3/8" Thick Shear plate with (3) - ¾" ϕ A325N Bolts & 1/4" thick weld - For 20 kip Shear

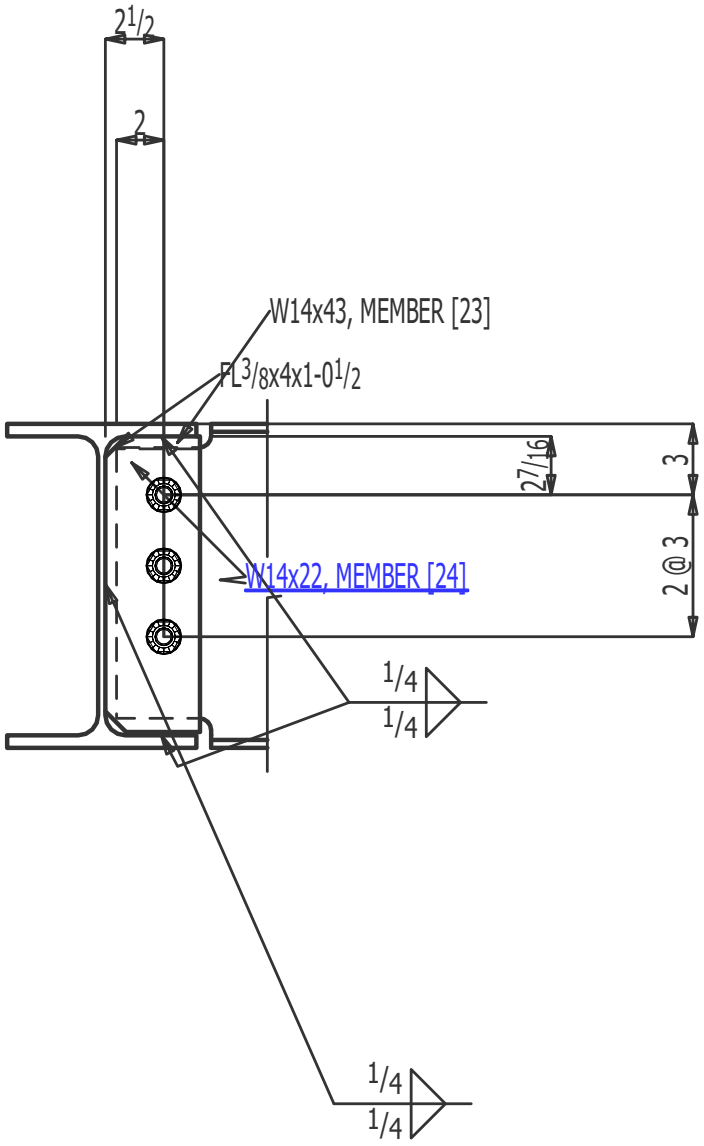
Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 4

CON - 4 [7] at X=8-6, Y=6-0 Elev=24-3 1/8



TOP SIDE VIEW

CON - 4



A

Section A
CON - 4 ELEVATION

Beam B_24 [24]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W14x22
Sequence:	1
ABM:	N/Assign
Plan length:	9-2 3/4
Camber:	0.00 in
Span length:	9-2 3/4
Slope:	0.00 °
Material length:	9-2 1/8
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	13.7 in
Web thickness, t_w:	0.23 in
Flange width, b_f:	5 in
Flange thickness, t_f:	0.335 in
Design k distance, k_{des}:	0.735 in
Detail k distance, k_{det}:	1.0625 in
Distance between web toes of fillets, T:	11.575 in
Moment of inertia about the major axis, I_x:	199 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Extend to both Flanges
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: Automatic
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	0.5 in
Mtrl Setback:	0.625 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	20.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_24 [24] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_23 [23]
Section size:	W14x43
End 0 elevation:	24-10
End 1 elevation:	24-10
Support intersection elevation:	24-10
Supporting beam rotation:	0.00 degrees
	(looking toward left end)
Material grade:	A992
Detail k distance, k_{det}:	1.375 in
Design k distance, k_{des}:	1.12 in
Supporting member thickness, t_{sup}:	0.305 in

Design loads

***Shear*:** 20.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 31.7 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Width, b:	4 in
	Depth, d:	12.5 in
Shear tab extension:	To top & bottom flange	
	Weld line to bolt group c.g., a:	2.4725 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.25 in
	Total effective weld throat, t_e:	0.35 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A325N
	Tension control field bolts	
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	3
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 8

Results summary

Web Side Plate on left end of Beam B_24 [24]

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Bolt shear of web bolts:	3	0.641	J3
Bolt bearing on beam web:	110	0.559	J3.10
Bolt bearing on plate:	110	0.559	J3.10
Block shear rupture of beam web:	6	0.531	J4.3
Shear rupture of beam web:	4	0.505	J4.2
Block shear rupture of plate:	6	0.414	J4.3
Flexure of coped beam:	41	0.396	Pg 9-6
Shear yielding of beam web:	5	0.380	G2.1
Shear rupture of plate:	21	0.311	J4.2
Shear yielding of plate:	38	0.296	J4.2
Shear of support:	36	0.135	J4.2
Flexure of plate:	314	0.078	Pg 10-90

Connection strength

	Unity ratio:
Shear:	0.641

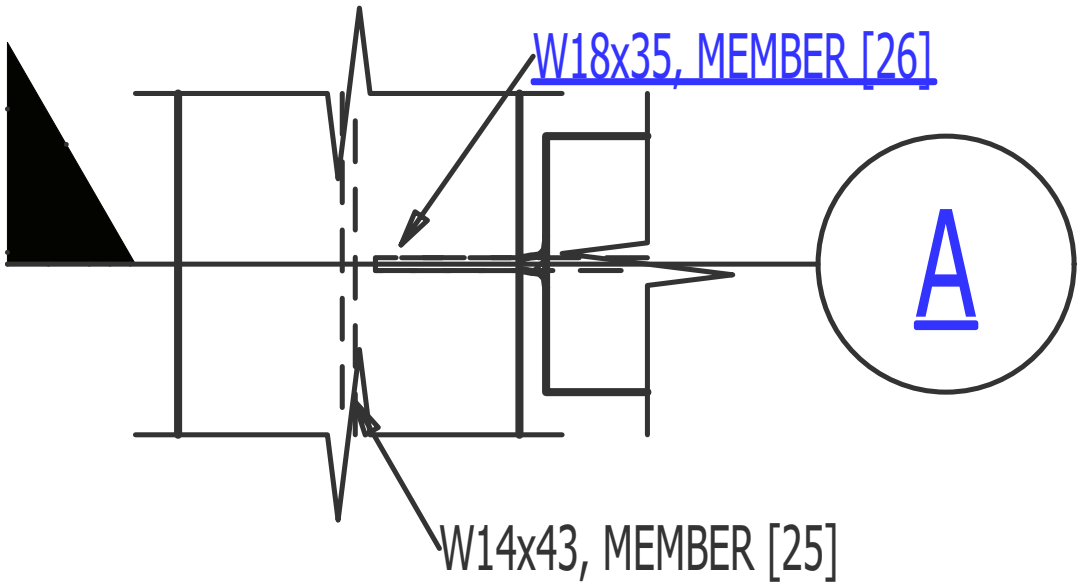
Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- CONNECTION IS OK
  - Strength equals or exceeds design loads.

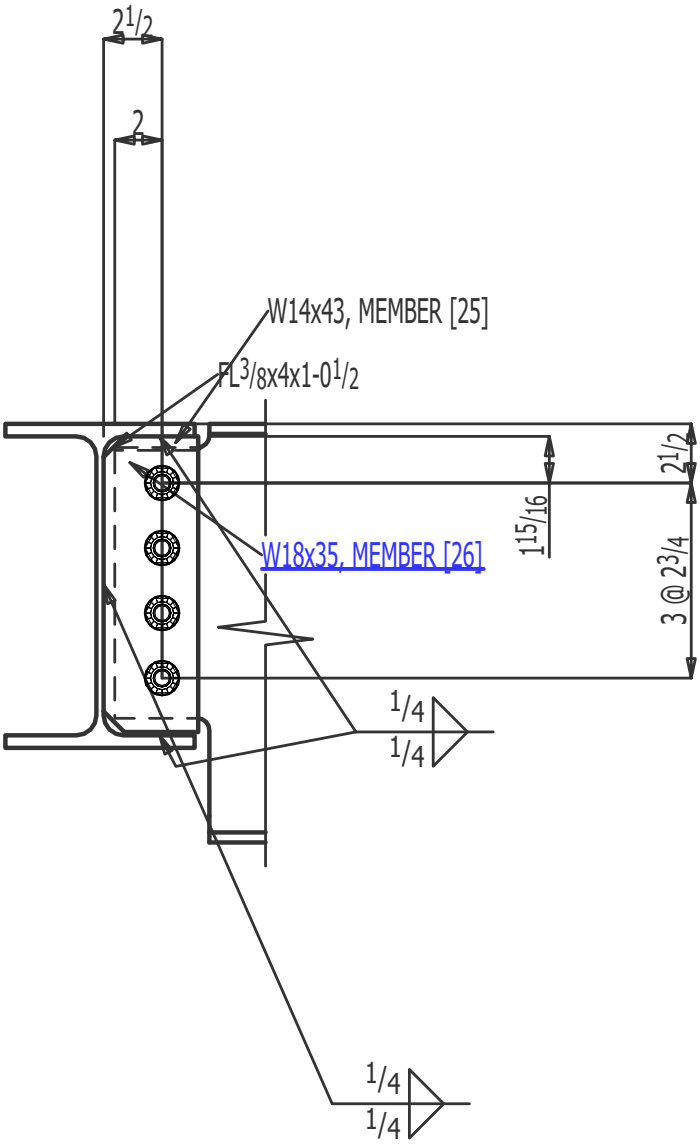
5) Shear Connection – W18x35 Beam connecting to W14x43 Beam using 3/8" Thick Shear plate with (4) - ¾" ϕ A325N Bolts & 1/4" thick weld - For 42 kip Shear

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON -5

CON -5 [8] at X=8-6, Y=18-7 1/4 Elev=24-1 1/8



TOP SIDE VIEW



A

Section A
ELEVATION

Beam B_26 [26]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x35
Sequence:	1
ABM:	N/Assign
Plan length:	9-2 3/4
Camber:	0.00 in
Span length:	9-2 3/4
Slope:	0.00 °
Material length:	9-2 1/8
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	17.7 in
Web thickness, t_w:	0.3 in
Flange width, b_f:	6 in
Flange thickness, t_f:	0.425 in
Design k distance, k_{des}:	0.827 in
Detail k distance, k_{det}:	1.125 in
Distance between web toes of fillets, T:	15.45 in
Moment of inertia about the major axis, I_x:	510 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Extend to both Flanges
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: Automatic
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	0.5 in
Mtrl Setback:	0.625 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	42.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_26 [26] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_25 [25]
Section size:	W14x43
End 0 elevation:	24-10
End 1 elevation:	24-10
Support intersection elevation:	24-10
Supporting beam rotation:	0.00 degrees
	(looking toward left end)
Material grade:	A992
Detail k distance, k_{det}:	1.375 in
Design k distance, k_{des}:	1.12 in
Supporting member thickness, t_{sup}:	0.305 in

Design loads

***Shear*:** 42.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 39.5 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Width, b:	4 in
	Depth, d:	12.5 in
Shear tab extension:	To top & bottom flange	
	Weld line to bolt group c.g., a:	2.4725 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.25 in
	Total effective weld throat, t_e:	0.35 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A325N
	Tension control field bolts	
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	4
	Bolt row spacing, s:	2.75 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 11

Results summary

Web Side Plate on left end of Beam B_26 [26]

Bolt capacity notes

- It is mandatory that SC joints also meet the strength requirements to resist the factored load as a bearing bolt, type N, joint; RCSC section 5.4 commentary

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Bolt shear of web bolts:	3	0.972	J3
Shear rupture of beam web:	4	0.903	J4.2
Bolt bearing on beam web:	110	0.881	J3.10
Bolt bearing on plate:	110	0.881	J3.10
Block shear rupture of plate:	6	0.794	J4.3
Block shear rupture of beam web:	6	0.772	J4.3
Shear rupture of plate:	21	0.716	J4.2
Flexure of coped beam:	41	0.638	Pg 9-6
Shear yielding of plate:	38	0.622	J4.2
Shear yielding of beam web:	5	0.611	G2.1
Shear of support:	36	0.282	J4.2
Flexure of plate:	314	0.164	Pg 10-90

Connection strength

	Unity ratio:
Shear:	0.972

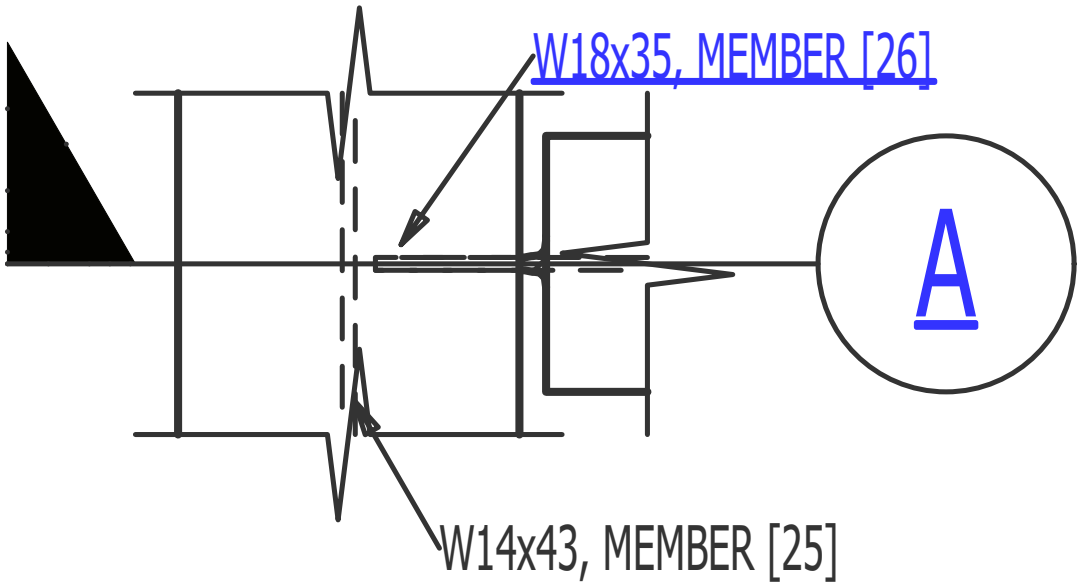
Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- CONNECTION IS OK
  - Strength equals or exceeds design loads.

5A) Shear Connection – W18x35 Beam connecting to W14x43 Beam using 3/8" Thick Shear plate with (4) - ¾" ϕ A325SC Bolts & 1/4" thick weld - For 42 kip Shear **- NO GOOD**

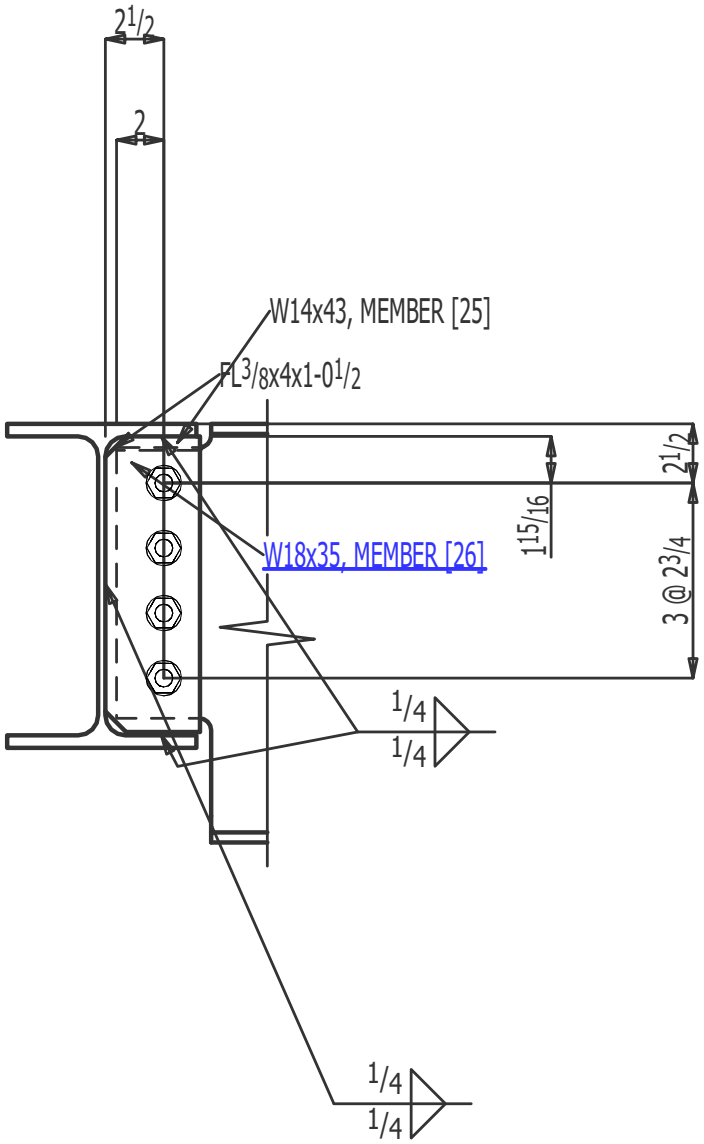
Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON -5A

CON -5A [8] at X=8-6, Y=18-7 1/4 Elev=24-1 1/8



TOP SIDE VIEW

CON -5A



A

Section A
CON -5A ELEVATION

Beam B_26 [26]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x35
Sequence:	1
ABM:	N/Assign
Plan length:	9-2 3/4
Camber:	0.00 in
Span length:	9-2 3/4
Slope:	0.00 °
Material length:	8-11 1/8
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	17.7 in
Web thickness, t_w:	0.3 in
Flange width, b_f:	6 in
Flange thickness, t_f:	0.425 in
Design k distance, k_{des}:	0.827 in
Detail k distance, k_{det}:	1.125 in
Distance between web toes of fillets, T:	15.45 in
Moment of inertia about the major axis, I_x:	510 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Extend to both Flanges
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	0.5 in
Mtrl Setback:	0.625 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	42.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_26 [26] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_25 [25]
Section size:	W14x43
End 0 elevation:	24-10
End 1 elevation:	24-10
Support intersection elevation:	24-10
Supporting beam rotation:	0.00 degrees
	(looking toward left end)
Material grade:	A992
Detail k distance, k_{det}:	1.375 in
Design k distance, k_{des}:	1.12 in
Supporting member thickness, t_{sup}:	0.305 in

Design loads

***Shear*:** 42.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 39.5 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Width, b:	4 in
	Depth, d:	12.5 in
Shear tab extension:	To top & bottom flange	
	Weld line to bolt group c.g., a:	2.4725 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.25 in
	Total effective weld throat, t_e:	0.35 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A325SC
	Surface:	Class A
	Mean slip coefficient:	0.30 (J3.8)
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	4
	Bolt row spacing, s:	2.75 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 12

Results summary

Web Side Plate on left end of Beam B_26 [26]

Bolt capacity notes

- It is mandatory that SC joints also meet the strength requirements to resist the factored load as a bearing bolt, type N, joint; RCSC section 5.4 commentary

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Bolt shear of web bolts:	3	1.834	J3
Bolt bearing on beam web:	110	1.660	J3.10
Bolt bearing on plate:	110	1.660	J3.10
Shear rupture of beam web:	4	0.903	J4.2
Block shear rupture of plate:	6	0.794	J4.3
Block shear rupture of beam web:	6	0.772	J4.3
Shear rupture of plate:	21	0.716	J4.2
Flexure of coped beam:	41	0.638	Pg 9-6
Shear yielding of plate:	38	0.622	J4.2
Shear yielding of beam web:	5	0.611	G2.1
Shear of support:	36	0.282	J4.2
Flexure of plate:	314	0.164	Pg 10-90

Warnings

- FORCED CONNECTION
  - Engineering review required to evaluate strength and the application of SDS2 design calculations to the specific material and geometry.

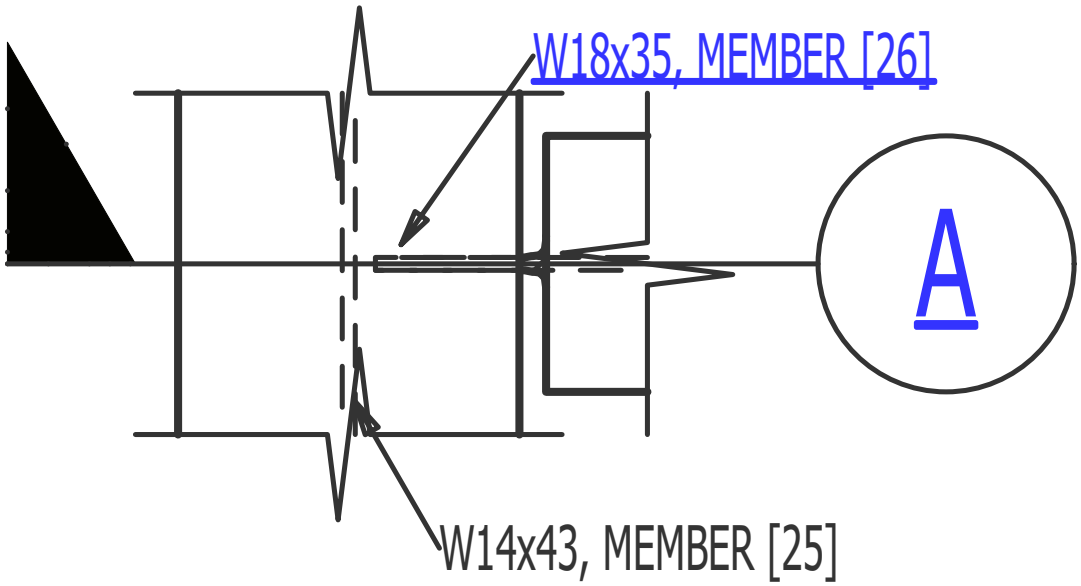
Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- CONNECTION DESIGN FAILURE
  - Bolt bearing strength on conn./supported member exceeded ⚠

5B) Shear Connection – W18x35 Beam connecting to W14x43 Beam using 3/8" Thick Shear plate with (4) - 1" ϕ A490SC Bolts & 1/4" thick weld - For 42 kip Shear **-REVISED**

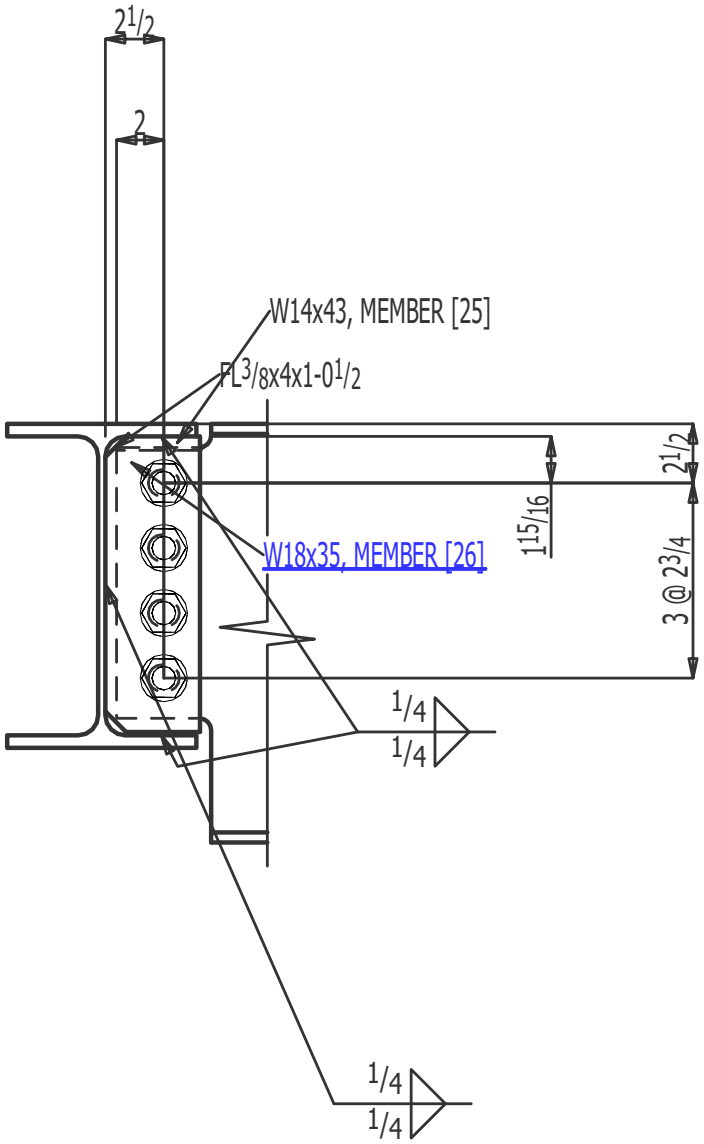
Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON -5B

CON -5B [8] at X=8-6, Y=18-7 1/4 Elev=24-1 1/8



TOP SIDE VIEW

CON -5B



A

Section A
CON -5B ELEVATION

Beam B_26 [26]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x35
Sequence:	1
ABM:	N/Assign
Plan length:	9-2 3/4
Camber:	0.00 in
Span length:	9-2 3/4
Slope:	0.00 °
Material length:	8-11 1/8
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	17.7 in
Web thickness, t_w:	0.3 in
Flange width, b_f:	6 in
Flange thickness, t_f:	0.425 in
Design k distance, k_{des}:	0.827 in
Detail k distance, k_{det}:	1.125 in
Distance between web toes of fillets, T:	15.45 in
Moment of inertia about the major axis, I_x:	510 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Extend to both Flanges
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	0.5 in
Mtrl Setback:	0.625 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	42.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_26 [26] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_25 [25]
Section size:	W14x43
End 0 elevation:	24-10
End 1 elevation:	24-10
Support intersection elevation:	24-10
Supporting beam rotation:	0.00 degrees
	(looking toward left end)
Material grade:	A992
Detail k distance, k_{det}:	1.375 in
Design k distance, k_{des}:	1.12 in
Supporting member thickness, t_{sup}:	0.305 in

Design loads

***Shear*:** 42.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 39.5 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Width, b:	4 in
	Depth, d:	12.5 in
Shear tab extension:	To top & bottom flange	
	Weld line to bolt group c.g., a:	2.4725 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.25 in
	Total effective weld throat, t_e:	0.35 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A490SC
	Surface:	Class A
	Mean slip coefficient:	0.30 (J3.8)
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	1
	Bolt rows, n:	4
	Bolt row spacing, s:	2.75 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 12

Results summary

Web Side Plate on left end of Beam B_26 [26]

Bolt capacity notes

- It is mandatory that SC joints also meet the strength requirements to resist the factored load as a bearing bolt, type N, joint; RCSC section 5.4 commentary

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Shear rupture of beam web:	4	1.071	J4.2
Bolt bearing on beam web:	20	1.037	J3.10
Bolt shear of web bolts:	3	0.974	J3
Bolt bearing on plate:	20	0.974	J3.10
Block shear rupture of beam web:	6	0.905	J4.3
Block shear rupture of plate:	6	0.845	J4.3
Shear rupture of plate:	21	0.830	J4.2
Flexure of plate:	345	0.704	Pg 10-90
Flexure of coped beam:	41	0.638	Pg 9-6
Shear yielding of plate:	38	0.622	J4.2
Shear yielding of beam web:	5	0.611	G2.1
Plate flexural rupture:	460	0.367	Pg 9-6
Buckling of plate:	346	0.329	Pg 9-7,8
Shear of support:	36	0.282	J4.2

Warnings

- FORCED CONNECTION
  - Engineering review required to evaluate strength and the application of SDS2 design calculations to the specific material and geometry.

Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- Neither exception from design check 2 on page 10-90 applies. Maximum thickness is 0.6531 in.
- CONNECTION DESIGN FAILURE
  - Beam net shear strength exceeded ⚠

1/4" Thick web doubler plate is adequate for this connection

6)Moment Connection – W14x_ Beam connecting to W18x35 Beam using 5/8" Thick flange plates @ top and bottom with 5/16" weld. - For 138 kip-ft Moment

RAM Connection

Units system: English

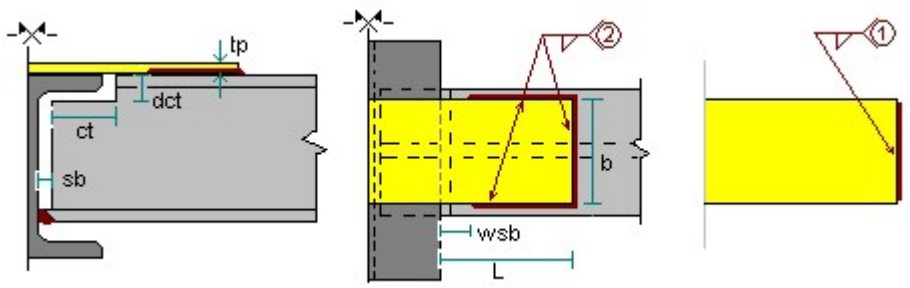
Steel Connections
Data

Connection: 6

Family: Beam - Girder (BG)
Type: Flange-plated
Description: Beam to girder

General information

Connector



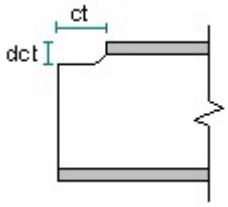
Members

Beam

General

Beam section	:	W 14X43
Beam material	:	A992 Gr50
Horizontal angle (deg)	:	0

Coped



dct: Top cope depth	:	0.75 in
ct: Top cope length	:	2.5 in

Girder

General

Support section	:	W 18X35
Support material	:	A992 Gr50

Flange-plated

Connector

Top plate section	:	PL 5/8x7 1/2x12

L: Top plate length	:	12 in
b: Top plate width	:	7.5 in
tp: Top plate thickness	:	0.625 in
Plate material	:	A572 Gr50

Beam side

Connection type	:	Welded
efb: Longitudinal distance to beam edge (bottom ...	:	0 in
Welding electrode to beam	:	E70XX
Weld fillet arrangement on beam	:	Full
D: Weld size to top plate (1/16 in)	:	5
wsb: Weld setback on top plate	:	2 in

RAM Connection

Units system: English

Steel Connections
Results

Connection: 6

Family: Beam - Girder (BG)
Type: Flange-plated
Description: Beam to girder

Design code: AISC 360-16 ASD

Demands

Description	Ru [kip]	Pu [kip]	Mu [kip*ft]	PufTop [kip]	PufBot [kip]	Load type
DL	0.00	0.00	138.00	-125.74	125.74	Design

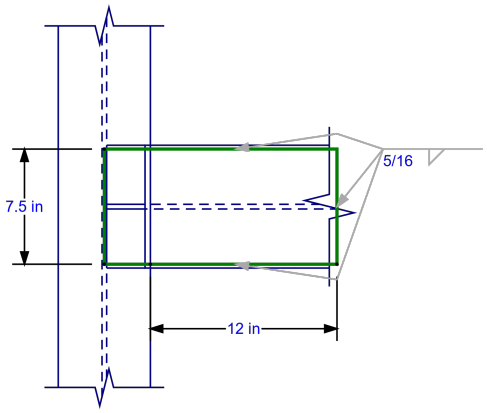
Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Plate (beam side)						
Top plate to beam weld size	[1/16in]	5	4	7	✔	Sec. J2.2b
Beam						
Top cope length	[in]	2.50	0.00	27.40	✔	
Top cope depth	[in]	0.75	0.00	6.85	✔	

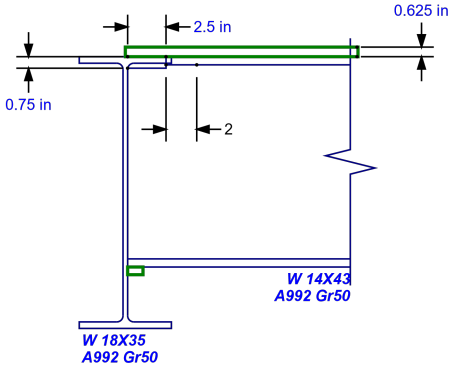
Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Top plate (beam side)						
Compression	[Kip]	140.34	118.18	DL	0.84	Sec. J4.4
Tension yielding	[Kip]	140.34	0.00	DL	0.00	Eq. J4-1
Weld to beam	[Kip]	131.09	125.74	DL	0.96	Eq. J2-9
Weld backup plate						
Full penetration weld capacity	[Kip]	126.95	125.74	DL	0.99	Eq. J2-2
Beam						
Shear rupture of base metal at fillet weld (top)	[in]	0.53	0.24	DL	0.45	Eq. 9-2
Global critical strength ratio	**0.99**					

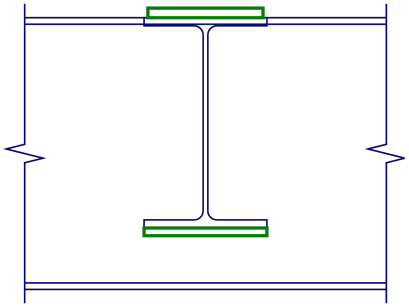
Top view



Lateral view



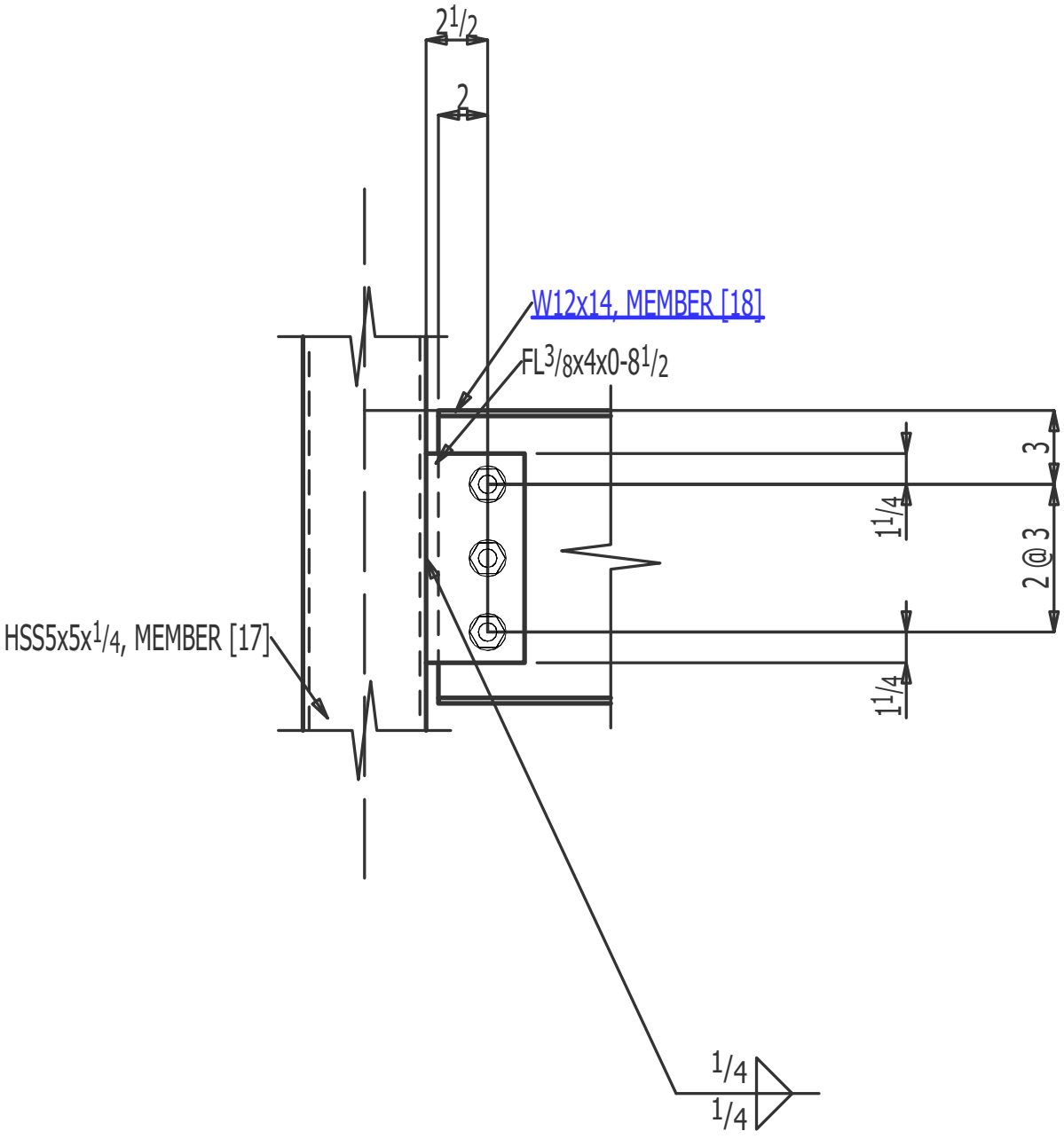
Front view



7) Shear Connection – W12x_ & W14x_ Beam connecting to HSS5x5x_ & HSS6x6_ Column using 3/8" Thick Shear plate with (3) - ¾" ϕ A325SC Bolts & 1/4" thick weld - For 19 kip Shear

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 7

CON - 7 [3] at X=17-8 3/4, Y=10 Elev=24-4 1/16



ELEVATION VIEW

Beam B_18 [18]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W12x14
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-9
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	11.9 in
Web thickness, t_w:	0.2 in
Flange width, b_f:	3.97 in
Flange thickness, t_f:	0.225 in
Design k distance, k_{des}:	0.525 in
Detail k distance, k_{det}:	0.75 in
Distance between web toes of fillets, T:	10.4 in
Moment of inertia about the major axis, I_x:	88.6 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Size as required
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	0.5 in
Mtrl Setback:	3 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	19.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_18 [18] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_17 [17]
Section size:	HSS5x5x1/4
End 0 elevation:	0
End 1 elevation:	35-0
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.25 in
Design thickness, t_d:	0.233 in

Design loads

***Shear*:** 19.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 44.4 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Width, b:	4 in

Connection details (continued)

	Depth, d:	8.5 in
	Weld line to bolt group c.g., a:	2.5 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.25 in
	Total effective weld throat, t_e:	0.35 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A325SC
	Surface:	Class A
	Mean slip coefficient:	0.30 (J3.8)
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	3
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Column reinforcement plate:	**Grade:**	A572-42
	Tensile strength, F_u:	60 ksi
	Yield stress, F_y:	42 ksi
	Thickness, t:	0
	Width, b:	3.75 in
	Length, l:	10 in
	Weld size:	0.125 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 23

Results summary

Web Side Plate on left end of Beam B_18 [18]

Bolt capacity notes

- It is mandatory that SC joints also meet the strength requirements to resist the factored load as a bearing bolt, type N, joint; RCSC section 5.4 commentary

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Bolt shear of web bolts:	3	1.152	J3
Bolt bearing on beam web:	110	1.000	J3.10
Bolt bearing on plate:	110	1.000	J3.10
Shear rupture of plate:	21	0.496	J4.2
Punching shear of support:	459	0.486	(10-7b)
Block shear rupture of plate:	6	0.476	J4.3
Shear yielding of plate:	38	0.414	J4.2
Shear yielding of beam web:	2	0.399	G2.1
Shear of support:	36	0.276	J4.2
Flexure of plate:	314	0.163	Pg 10-90

Warnings

- FORCED CONNECTION
  - Engineering review required to evaluate strength and the application of SDS2 design calculations to the specific material and geometry.

Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- Supporting HSS column slenderness check

$$\left(\frac{b}{t} = 18.4592\right) \leq \left(1.4 \cdot \sqrt{\left(\frac{E}{F_y}\right)} = 35.1518\right) \quad (AISC\ B4.1).$$

- CONNECTION DESIGN FAILURE

Notes and conclusions (continued)

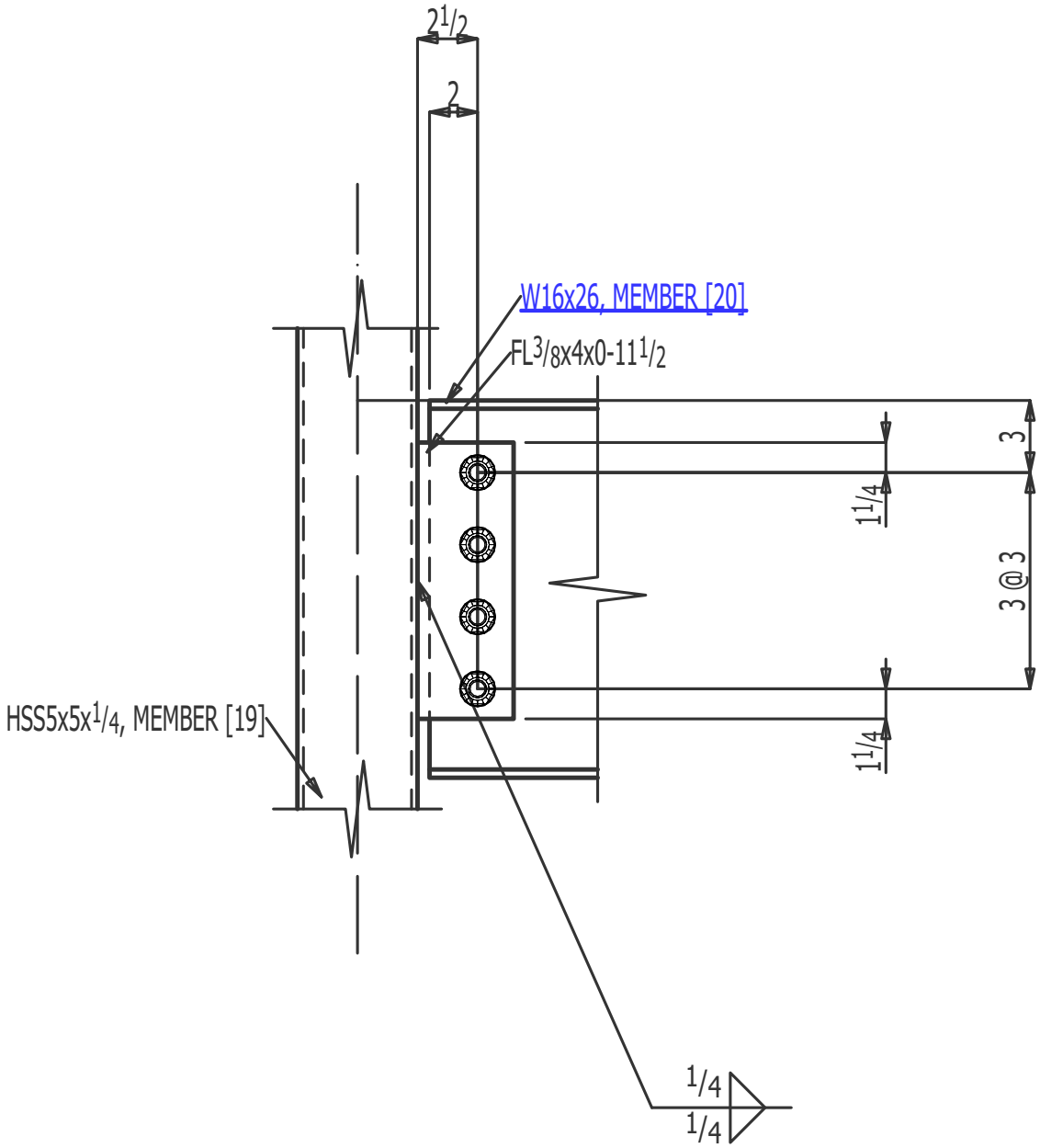
- Bolt bearing strength on conn./supported member exceeded ⚠

  10% OVERAGE ALLOWANCE IS CONSIDERED FOR THIS CONNECTION **Gelinas Comment**

8) Shear Connection – W16x26 Beam connecting to HSS5x5x_ Column using 3/8" Thick Shear plate with (4) - ¾" ϕ A325N Bolts & 1/4" thick weld - For 20 kip Shear

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 8

CON - 8 [4] at X=29-11 1/2, Y=10 Elev=24-2 1/8



ELEVATION VIEW
CON - 8

Beam B_20 [20]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W16x26
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-9
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	15.7 in
Web thickness, t_w:	0.25 in
Flange width, b_f:	5.5 in
Flange thickness, t_f:	0.345 in
Design k distance, k_{des}:	0.747 in
Detail k distance, k_{det}:	1.0625 in
Distance between web toes of fillets, T:	13.575 in
Moment of inertia about the major axis, I_x:	301 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Size as required
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: Automatic
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	0.5 in
Mtrl Setback:	3 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	20.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_20 [20] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_19 [19]
Section size:	HSS5x5x1/4
End 0 elevation:	0
End 1 elevation:	35-0
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.25 in
Design thickness, t_d:	0.233 in

Design loads

***Shear*:** 20.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 28.4 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Width, b:	4 in

Connection details (continued)

	Depth, d:	11.5 in
	Weld line to bolt group c.g., a:	2.5 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.25 in
	Total effective weld throat, t_e:	0.35 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A325N
	Tension control field bolts	
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	4
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 22

Results summary

Web Side Plate on left end of Beam B_20 [20]

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Bolt shear of web bolts:	3	0.458	J3
Bolt bearing on plate:	110	0.427	J3.10
Bolt bearing on beam web:	110	0.419	J3.10
Block shear rupture of plate:	6	0.385	J4.3
Shear rupture of plate:	21	0.383	J4.2
Shear yielding of plate:	38	0.322	J4.2
Punching shear of support:	459	0.280	(10-7b)
Shear yielding of beam web:	2	0.255	G2.1
Shear of support:	36	0.214	J4.2
Flexure of plate:	314	0.094	Pg 10-90

Connection strength

	Unity ratio:
Shear:	0.458

Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- Supporting HSS column slenderness check

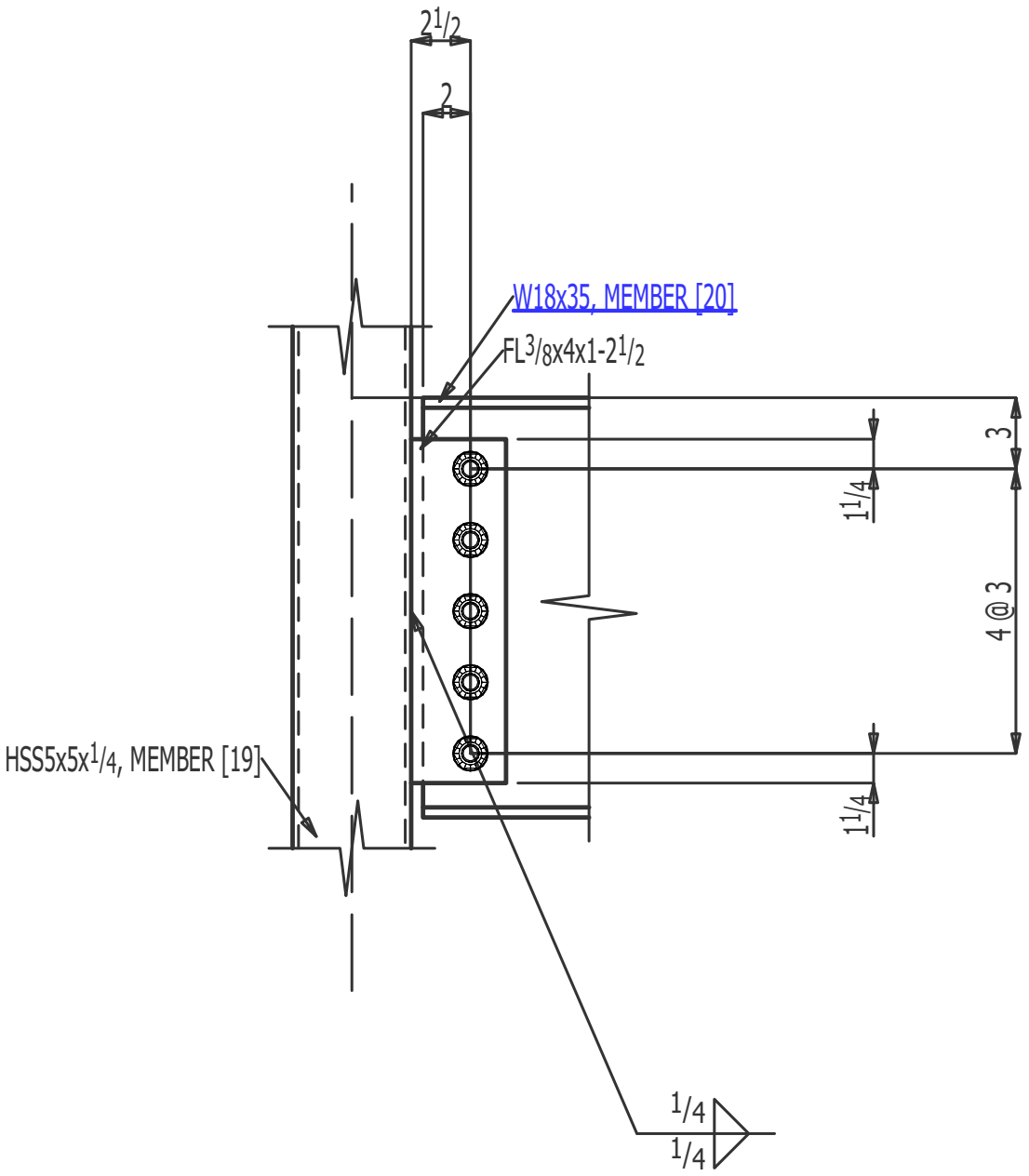
$$\left(\frac{b}{t} = 18.4592\right) \leq \left(1.4 \cdot \sqrt{\left(\frac{E}{F_y}\right)} = 35.1518\right) \quad (AISC\ B4.1).$$

- CONNECTION IS OK
  - Strength equals or exceeds design loads.

8A) Shear Connection – W18x_ Beam connecting to HSS5x5x_ Column using 3/8" Thick Shear plate with (5) - ¾" ϕ A325N Bolts & 1/4" thick weld - For 55 kip Shear - **REVISED**

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 8

CON - 8 [4] at X=29-11 1/2, Y=10 Elev=24-1 1/8



ELEVATION VIEW

CON - 8

Beam B_20 [20]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x35
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-9
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	17.7 in
Web thickness, t_w:	0.3 in
Flange width, b_f:	6 in
Flange thickness, t_f:	0.425 in
Design k distance, k_{des}:	0.827 in
Detail k distance, k_{det}:	1.125 in
Distance between web toes of fillets, T:	15.45 in
Moment of inertia about the major axis, I_x:	510 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Size as required
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: Automatic
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	0.5 in
Mtrl Setback:	3 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	55.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_20 [20] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_19 [19]
Section size:	HSS5x5x1/4
End 0 elevation:	0
End 1 elevation:	35-0
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.25 in
Design thickness, t_d:	0.233 in

Design loads

***Shear*:** 55.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 51.8 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Width, b:	4 in

Connection details (continued)

	Depth, d:	14.5 in
	Weld line to bolt group c.g., a:	2.5 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.25 in
	Total effective weld throat, t_e:	0.35 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A325N
	Tension control field bolts	
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	5
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 19

Results summary

Web Side Plate on left end of Beam B_20 [20]

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Bolt shear of web bolts:	3	0.982	J3
Bolt bearing on plate:	110	0.937	J3.10
Bolt bearing on beam web:	110	0.923	J3.10
Block shear rupture of plate:	6	0.857	J4.3
Shear rupture of plate:	21	0.832	J4.2
Shear yielding of plate:	38	0.702	J4.2
Shear yielding of beam web:	2	0.518	G2.1
Punching shear of support:	459	0.484	(10-7b)
Shear of support:	36	0.468	J4.2
Flexure of plate:	314	0.162	Pg 10-90

Connection strength

	Unity ratio:
Shear:	0.982

Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- Supporting HSS column slenderness check

$$\left(\frac{b}{t} = 18.4592\right) \leq \left(1.4 \cdot \sqrt{\left(\frac{E}{F_y}\right)} = 35.1518\right) \quad (AISC\ B4.1).$$

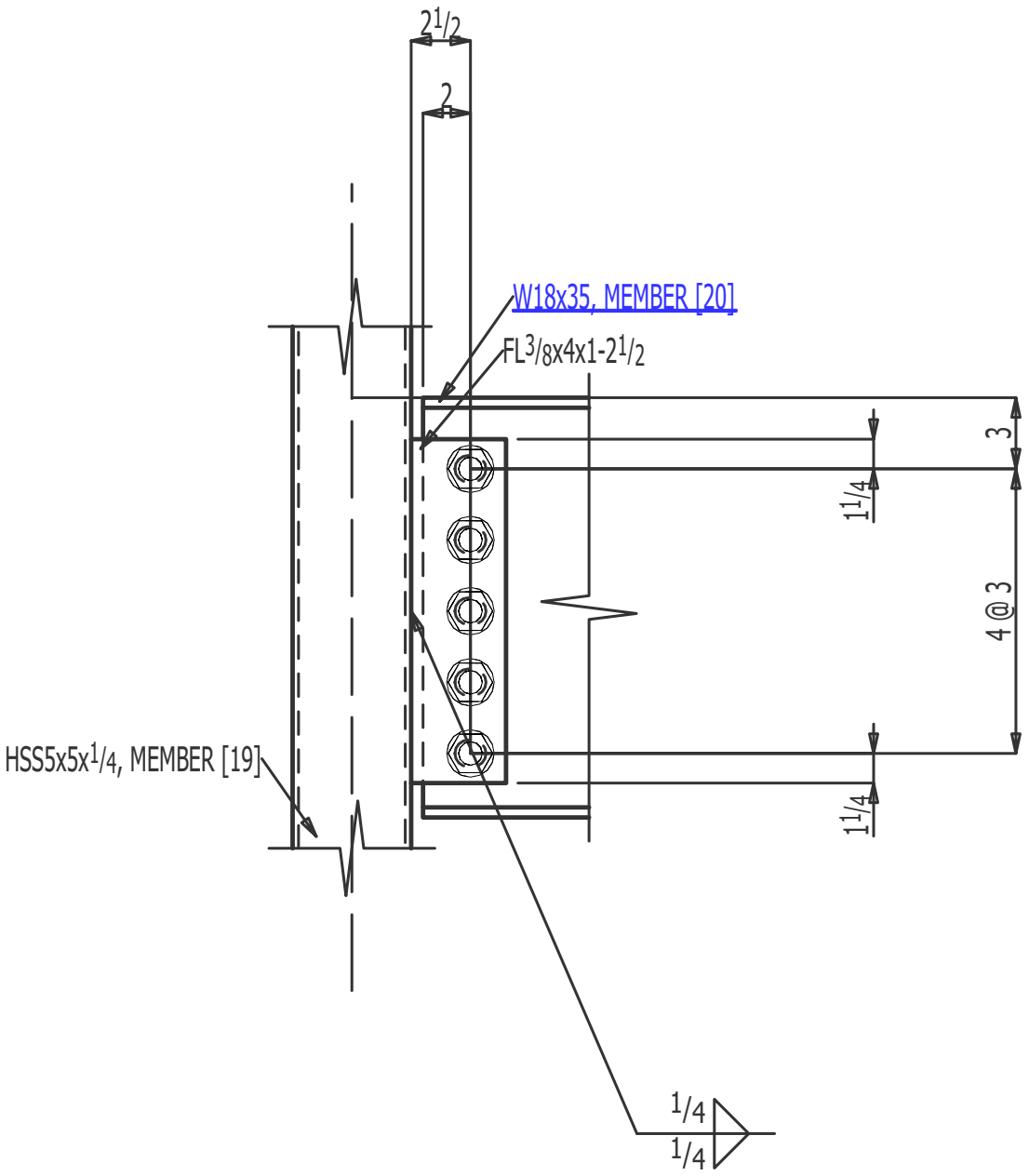
- CONNECTION IS OK
  - Strength equals or exceeds design loads.

Connection has been designed for minimum number of 5 Bolts. **Gelinas Comment**

8B) Shear Connection – W18x_ Beam connecting to HSS5x5x_ Column using 3/8" Thick Shear plate with (5) - 1" ϕ A490SC Bolts & 1/4" thick weld - For 55 kip Shear - **REVISED**

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 8

CON - 8 [4] at X=29-11 1/2, Y=10 Elev=24-1 1/8



ELEVATION VIEW
CON - 8

Beam B_20 [20]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x35
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-9
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	17.7 in
Web thickness, t_w:	0.3 in
Flange width, b_f:	6 in
Flange thickness, t_f:	0.425 in
Design k distance, k_{des}:	0.827 in
Detail k distance, k_{det}:	1.125 in
Distance between web toes of fillets, T:	15.45 in
Moment of inertia about the major axis, I_x:	510 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Size as required
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: Automatic
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	0.5 in
Mtrl Setback:	3 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	55.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_20 [20] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_19 [19]
Section size:	HSS5x5x1/4
End 0 elevation:	0
End 1 elevation:	35-0
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.25 in
Design thickness, t_d:	0.233 in

Design loads

***Shear*:** 55.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 51.8 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Width, b:	4 in

Connection details (continued)

	Depth, d:	14.5 in
	Weld line to bolt group c.g., a:	2.5 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.25 in
	Total effective weld throat, t_e:	0.35 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A490SC
	Surface:	Class A
	Mean slip coefficient:	0.30 (J3.8)
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	1
	Bolt rows, n:	5
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 19

Results summary

Web Side Plate on left end of Beam B_20 [20]

AISC manual extended configuration and design method

***Coefficient C'*:**	17.15 in
$6*M_{max}/F_y*d^2$:	0.81 in

Bolt capacity notes

- It is mandatory that SC joints also meet the strength requirements to resist the factored load as a bearing bolt, type N, joint; RCSC section 5.4 commentary

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Bolt bearing on plate:	20	0.987	J3.10
Shear rupture of plate:	21	0.984	J4.2
Bolt bearing on beam web:	20	0.914	J3.10
Bolt shear of web bolts:	3	0.914	J3
Block shear rupture of plate:	6	0.911	J4.3
Flexure of plate:	345	0.773	Pg 10-90
Shear yielding of plate:	38	0.702	J4.2
Shear yielding of beam web:	2	0.518	G2.1
Punching shear of support:	459	0.484	(10-7b)
Shear of support:	36	0.468	J4.2
Plate flexural rupture:	460	0.410	Pg 9-6
Buckling of plate:	346	0.324	Pg 9-7,8

Connection strength

	Unity ratio:
Shear:	0.987

Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- Neither exception from design check 2 on page 10-90 applies. Maximum thickness is 0.8066 in.
- Supporting HSS column slenderness check

$$\left(\frac{b}{t} = 18.4592\right) \leq \left(1.4 \cdot \sqrt{\left(\frac{E}{F_y}\right)} = 35.1518\right) \quad (AISC\ B4.1).$$

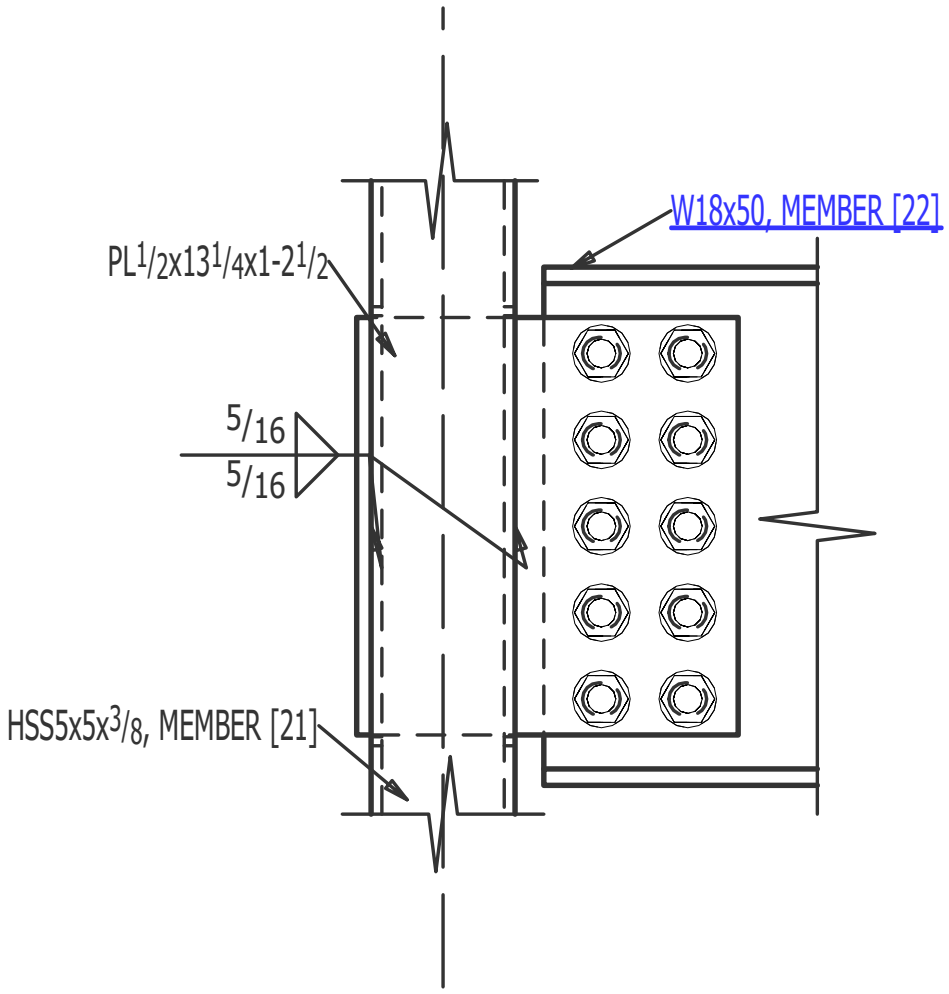
- CONNECTION IS OK
  - Strength equals or exceeds design loads.

Connection has been designed for minimum number of 5 Bolts. **Gelinas Comment**

9) Shear Connection – W18x50 Beam connecting to HSS5x5x3/8 Column using 1/2" Thick Shear through plate with (10) - 1" ϕ A490N Bolts & 5/16" thick weld - For 83 kip Shear - **NO GOOD**

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 9

CON - 9 [5] at X=17-8 3/4, Y=10-0 3/4 Elev=24-1



ELEVATION VIEW

CON - 9

Beam B_22 [22]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x50
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-8 1/2
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	18 in
Web thickness, t_w:	0.355 in
Flange width, b_f:	7.5 in
Flange thickness, t_f:	0.57 in
Design k distance, k_{des}:	0.972 in
Detail k distance, k_{det}:	1.25 in
Distance between web toes of fillets, T:	15.5 in
Moment of inertia about the major axis, I_x:	800 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Size as required, Through Plate
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	1 in
Mtrl Setback:	3.5 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	83.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_22 [22] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_21 [21]
Section size:	HSS5x5x3/8
End 0 elevation:	0
End 1 elevation:	59-10
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.375 in
Design thickness, t_d:	0.349 in

Design loads

***Shear*:** 83.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 64.9 % of the maximum web shear.

Connection summary

- WEB SIDE PL THRU TUBE COLUMN

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.5 in
	Depth, d:	14.5 in

Connection details (continued)

	Weld line to bolt group c.g., a:	4.5 in
Welds:	**Weld leg size, w:**	0.3125 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A490N
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	1
	Bolt rows, n:	5
	Bolt row spacing, s:	3 in
	Bolt columns, m:	2
	Bolt column spacing, s_{col}:	3 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 21

AISC manual extended configuration and design method

***Coefficient C'*:**	38.67 in
$6*M_{max}/F_y*d^2$:	1.82 in

Results summary

Shear Through Plate on left end of Beam B_22 [22]

Bolt capacity notes

- It is mandatory that SC joints also meet the strength requirements to resist the factored load as a bearing bolt, type N, joint; RCSC section 5.4 commentary

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Rupture of weld to supporting member:	238	1.172	J2, Table J2.5
Shear rupture of plate:	21	1.114	J4.2
Block shear rupture of plate:	6	0.960	J4.3
Flexure of plate:	345	0.908	Pg 10-90
Shear yielding of plate:	38	0.795	J4.2
Shear of support:	394	0.754	J4.2
Shear yielding of beam web:	2	0.649	G2.1
Plate flexural rupture:	460	0.557	Pg 9-6
Bolt bearing on plate:	20	0.540	J3.10
Bolt bearing on beam web:	20	0.491	J3.10
Bolt shear of web bolts:	3	0.480	J3
Buckling of plate:	346	0.440	Pg 9-7,8

Warnings

- FORCED CONNECTION
  - Engineering review required to evaluate strength and the application of SDS2 design calculations to the specific material and geometry.

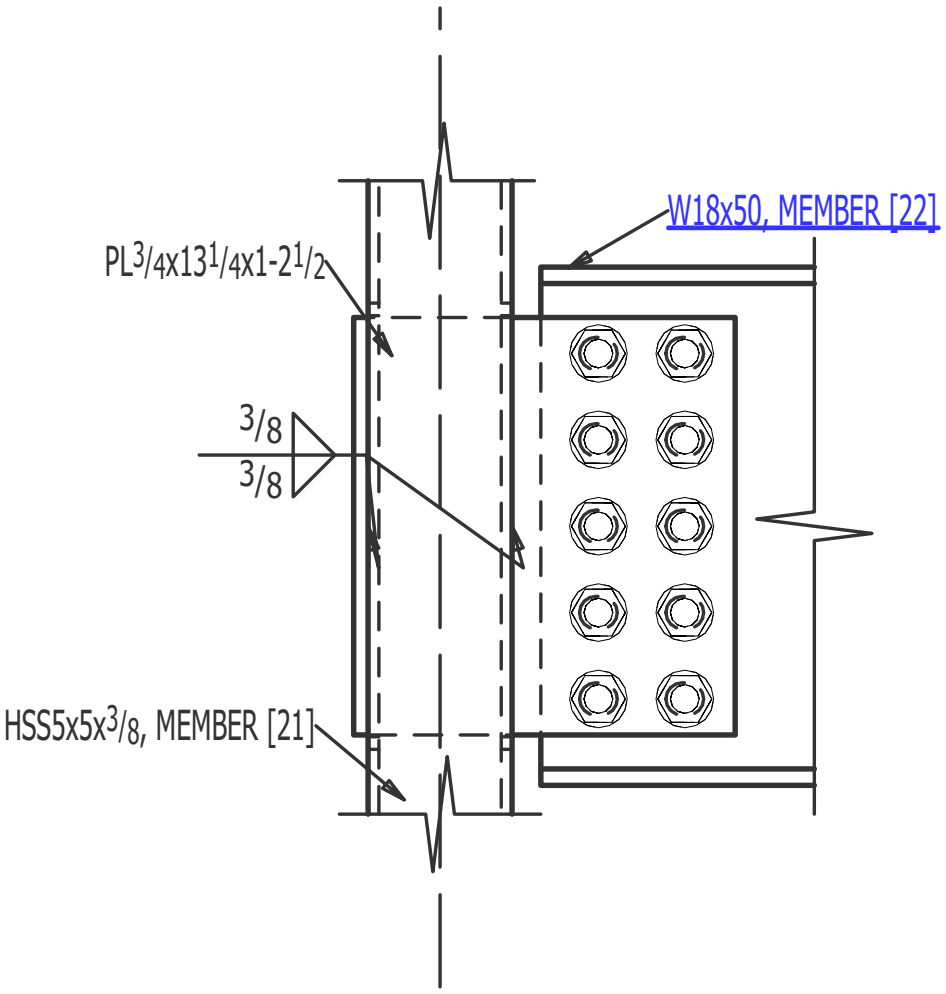
Notes and conclusions

- Maximum unity ratio for design: 0.99
- Neither exception from design check 2 on page 10-90 applies. Maximum thickness is 1.8190 in.
- CONNECTION DESIGN FAILURE
  - Connection net shear strength exceeded ⚠

9A) Shear Connection – W18x50 Beam connecting to HSS5x5x3/8 Column using 3/4" Thick Shear through plate with (10) - 1" ϕ A490N Bolts & 3/8" thick weld - For 83 kip Shear - **REVISED**

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 9A

CON - 9A [5] at X=17-8 3/4, Y=10-0 3/4 Elev=24-1



ELEVATION VIEW

CON - 9A

Beam B_22 [22]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x50
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-8 1/2
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	18 in
Web thickness, t_w:	0.355 in
Flange width, b_f:	7.5 in
Flange thickness, t_f:	0.57 in
Design k distance, k_{des}:	0.972 in
Detail k distance, k_{det}:	1.25 in
Distance between web toes of fillets, T:	15.5 in
Moment of inertia about the major axis, I_x:	800 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Size as required, Through Plate
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: Automatic
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	1 in
Mtrl Setback:	3.5 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	83.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_22 [22] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_21 [21]
Section size:	HSS5x5x3/8
End 0 elevation:	0
End 1 elevation:	59-10
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.375 in
Design thickness, t_d:	0.349 in

Design loads

***Shear*:** 83.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 64.9 % of the maximum web shear.

Connection summary

- WEB SIDE PL THRU TUBE COLUMN

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.75 in
	Depth, d:	14.5 in

Connection details (continued)

	Weld line to bolt group c.g., a:	4.5 in
Welds:	**Weld leg size, w:**	0.375 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A490N
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	1
	Bolt rows, n:	5
	Bolt row spacing, s:	3 in
	Bolt columns, m:	2
	Bolt column spacing, s_{col}:	3 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 21

AISC manual extended configuration and design method

***Coefficient C'*:**	38.67 in
$6*M_{max}/F_y*d^2$:	1.82 in

Results summary

Shear Through Plate on left end of Beam B_22 [22]

Bolt capacity notes

- It is mandatory that SC joints also meet the strength requirements to resist the factored load as a bearing bolt, type N, joint; RCSC section 5.4 commentary

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Rupture of weld to supporting member:	238	0.977	J2, Table J2.5
Shear of support:	394	0.754	J4.2
Shear rupture of plate:	21	0.743	J4.2
Shear yielding of beam web:	2	0.649	G2.1
Block shear rupture of plate:	6	0.640	J4.3
Flexure of plate:	345	0.606	Pg 10-90
Shear yielding of plate:	38	0.530	J4.2
Bolt bearing on plate:	20	0.514	J3.10
Bolt bearing on beam web:	20	0.491	J3.10
Bolt shear of web bolts:	3	0.480	J3
Plate flexural rupture:	460	0.371	Pg 9-6
Buckling of plate:	346	0.293	Pg 9-7,8

Warnings

- FORCED CONNECTION
  - Engineering review required to evaluate strength and the application of SDS2 design calculations to the specific material and geometry.

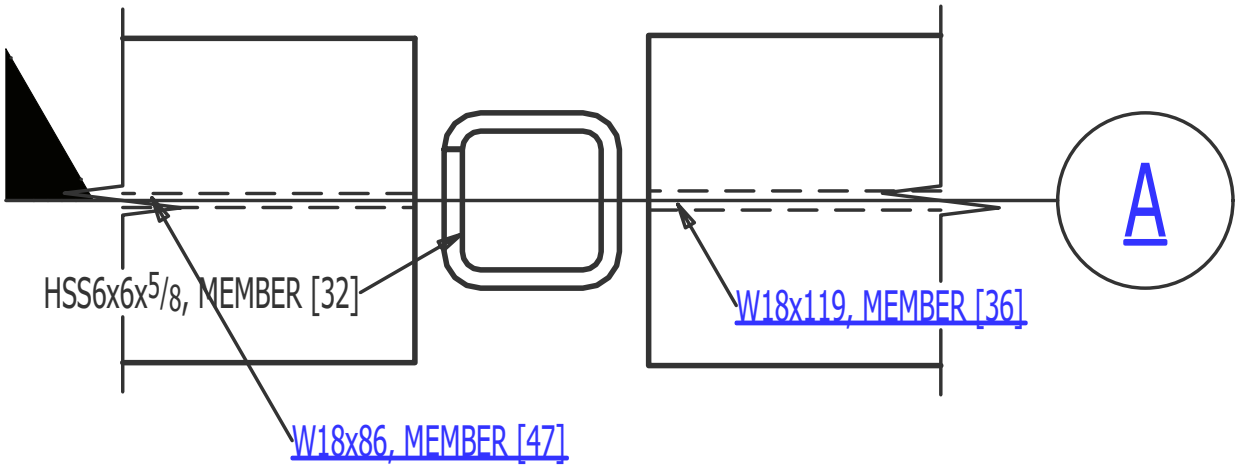
Notes and conclusions

- Maximum unity ratio for design: 0.99
- Neither exception from design check 2 on page 10-90 applies. Maximum thickness is 1.8190 in.

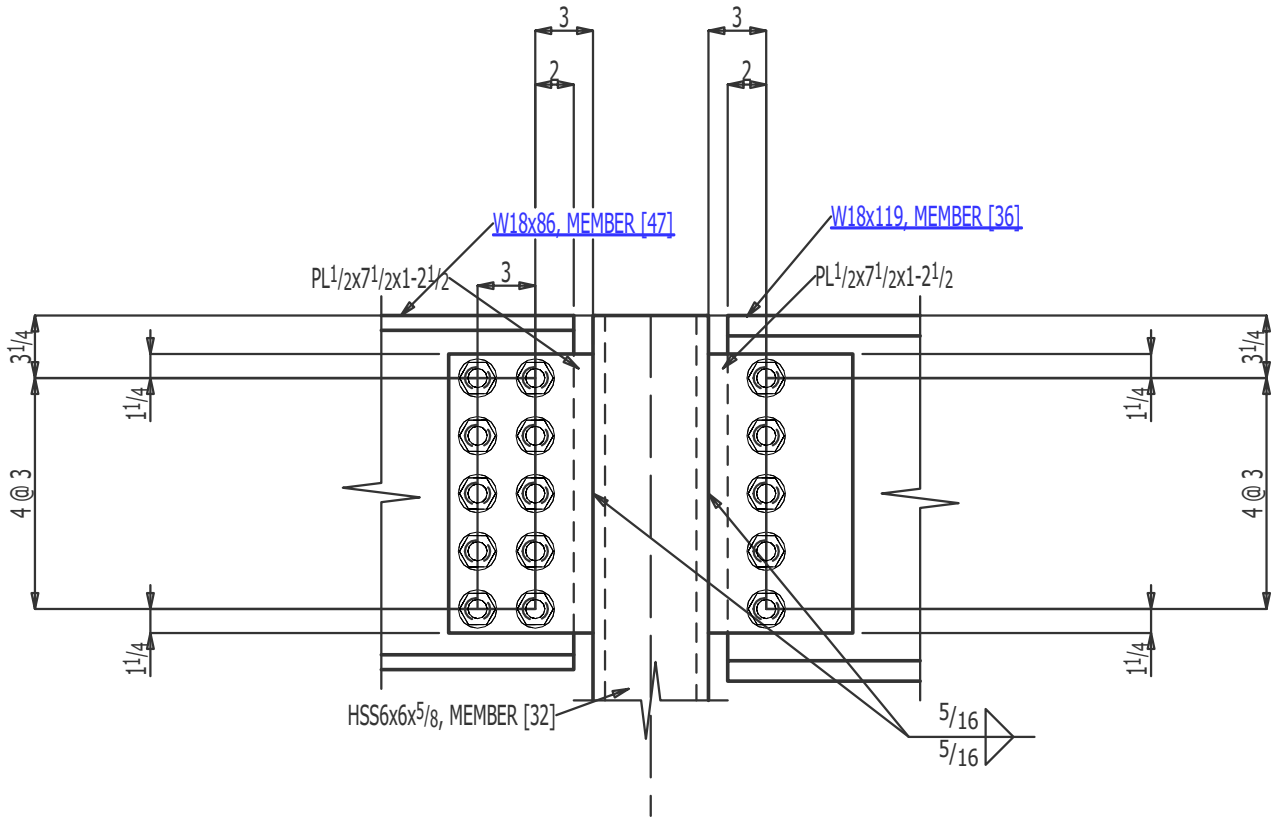
10) Shear Connection – W18x_ Beam connecting to HSS6x6x_ Column using 1/2" Thick Shear plate with (10) - 1" ϕ A490N Bolts & 5/16" thick weld - For 114 kip Shear - **NO GOOD**

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 10

CON - 10 [11] at X=32-11 1/2, Y=20-3 Elev=24-0 1/2



TOP SIDE VIEW
CON - 10



A

Section A
CON - 10 ELEVATION

Beam B_36 [36]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x119
Sequence:	1
ABM:	N/Assign
Plan length:	9-9
Camber:	0.00 in
Span length:	9-9
Slope:	0.00 °
Material length:	9-5
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	19 in
Web thickness, t_w:	0.655 in
Flange width, b_f:	11.3 in
Flange thickness, t_f:	1.06 in
Design k distance, k_{des}:	1.46 in
Detail k distance, k_{det}:	1.9375 in
Distance between web toes of fillets, T:	15.125 in
Moment of inertia about the major axis, I_x:	2190 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Size as required
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	1 in
Mtrl Setback:	4 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	114.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_36 [36] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_35 [32]
Section size:	HSS6x6x5/8
End 0 elevation:	0
End 1 elevation:	24-10
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.625 in
Design thickness, t_d:	0.581 in

Design loads

***Shear*:** 114.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 45.8 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.5 in
	Width, b:	7.5 in

Connection details (continued)

	Depth, d:	14.5 in
	Weld line to bolt group c.g., a:	3 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.3125 in
	Total effective weld throat, t_e:	0.44 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A490N
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	1
	Bolt rows, n:	5
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 19

Beam B_46 [47]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x86
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-7
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	18.4 in
Web thickness, t_w:	0.48 in
Flange width, b_f:	11.1 in
Flange thickness, t_f:	0.77 in
Design k distance, k_{des}:	1.17 in
Detail k distance, k_{det}:	1.625 in
Distance between web toes of fillets, T:	15.15 in
Moment of inertia about the major axis, I_x:	1530 in^4

Design summary

Right end

Connection:	Web side plate
	Plate, Size as required
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	1 in
Mtrl Setback:	4 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	114.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_46 [47] Connection strength check: RIGHT END

Member end summary

Connecting nodes

Node 1

Column:	C_35 [32]
Section size:	HSS6x6x5/8
End 0 elevation:	0
End 1 elevation:	24-10
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.625 in
Design thickness, t_d:	0.581 in

Design loads

***Shear*:** 114.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 64.6 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.5 in
	Width, b:	7.5 in

Connection details (continued)

	Depth, d:	14.5 in
	Weld line to bolt group c.g., a:	4.5 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.3125 in
	Total effective weld throat, t_e:	0.44 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A490N
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	1
	Bolt rows, n:	5
	Bolt row spacing, s:	3 in
	Bolt columns, m:	2
	Bolt column spacing, s_{col}:	3 in
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 17

AISC manual extended configuration and design method

***Coefficient C'*:**	38.67 in
$6*M_{max}/F_y*d^2$:	1.82 in

Results summary

Web Side Plate on left end of Beam B_36 [36]

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Shear rupture of plate:	21	1.530	J4.2
Shear yielding of plate:	38	1.092	J4.2
Bolt bearing on plate:	110	0.960	J3.10
Bolt shear of web bolts:	3	0.927	J3
Block shear rupture of plate:	6	0.919	J4.3
Bolt bearing on beam web:	110	0.854	J3.10
Punching shear of support:	459	0.483	(10-7b)
Shear yielding of beam web:	2	0.458	G2.1
Shear of support:	36	0.389	J4.2
Flexure of plate:	314	0.302	Pg 10-90

Warnings

- FORCED CONNECTION
  - Engineering review required to evaluate strength and the application of SDS2 design calculations to the specific material and geometry.

Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- Supporting HSS column slenderness check

$$\left(\frac{b}{t} = 7.32702\right) \leq \left(1.4 \cdot \sqrt{\left(\frac{E}{F_y}\right)} = 35.1518\right) \quad (AISC\ B4.1).$$

- CONNECTION DESIGN FAILURE
  - Connection gross shear strength exceeded ⚠

Web Side Plate on right end of Beam B_46 [47]

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Shear rupture of plate:	21	1.530	J4.2
Block shear rupture of plate:	6	1.318	J4.3
Flexure of plate:	345	1.248	Pg 10-90
Shear yielding of plate:	38	1.092	J4.2
Plate flexural rupture:	460	0.765	Pg 9-6
Bolt bearing on plate:	20	0.742	J3.10
Punching shear of support:	459	0.724	(10-7b)
Bolt bearing on beam web:	20	0.660	J3.10
Bolt shear of web bolts:	3	0.660	J3
Shear yielding of beam web:	2	0.645	G2.1
Buckling of plate:	346	0.604	Pg 9-7,8
Shear of support:	36	0.389	J4.2

Warnings

- FORCED CONNECTION
  - Engineering review required to evaluate strength and the application of SDS2 design calculations to the specific material and geometry.

Notes and conclusions

- Maximum unity ratio for design: 0.99
- Weld sized to develop the full plate strength.
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- Exception (b) from design check 2 on page 10-90 applies.
- Supporting HSS column slenderness check

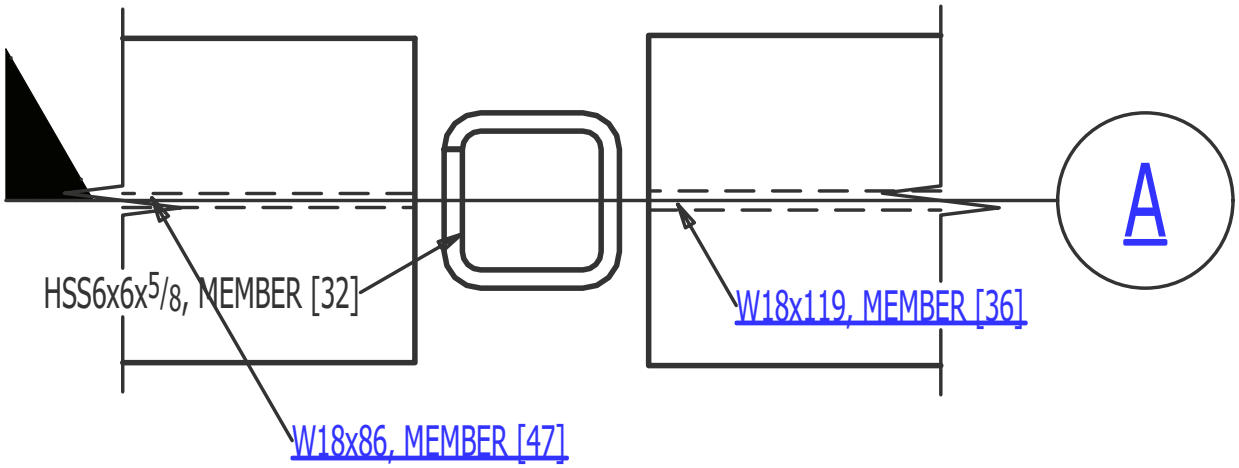
$$\left(\frac{b}{t} = 7.32702\right) \leq \left(1.4 \cdot \sqrt{\left(\frac{E}{F_y}\right)} = 35.1518\right) \quad (AISC\ B4.1).$$

- CONNECTION DESIGN FAILURE
  - Block shear strength exceeded ⚠

10A) Shear Connection – W18x_ Beam connecting to HSS6x6x_ Column using 3/4" Thick Shear plate with (10) - 1" ϕ A490N Bolts & 3/8" thick weld - For 114 kip Shear - **REVISED**

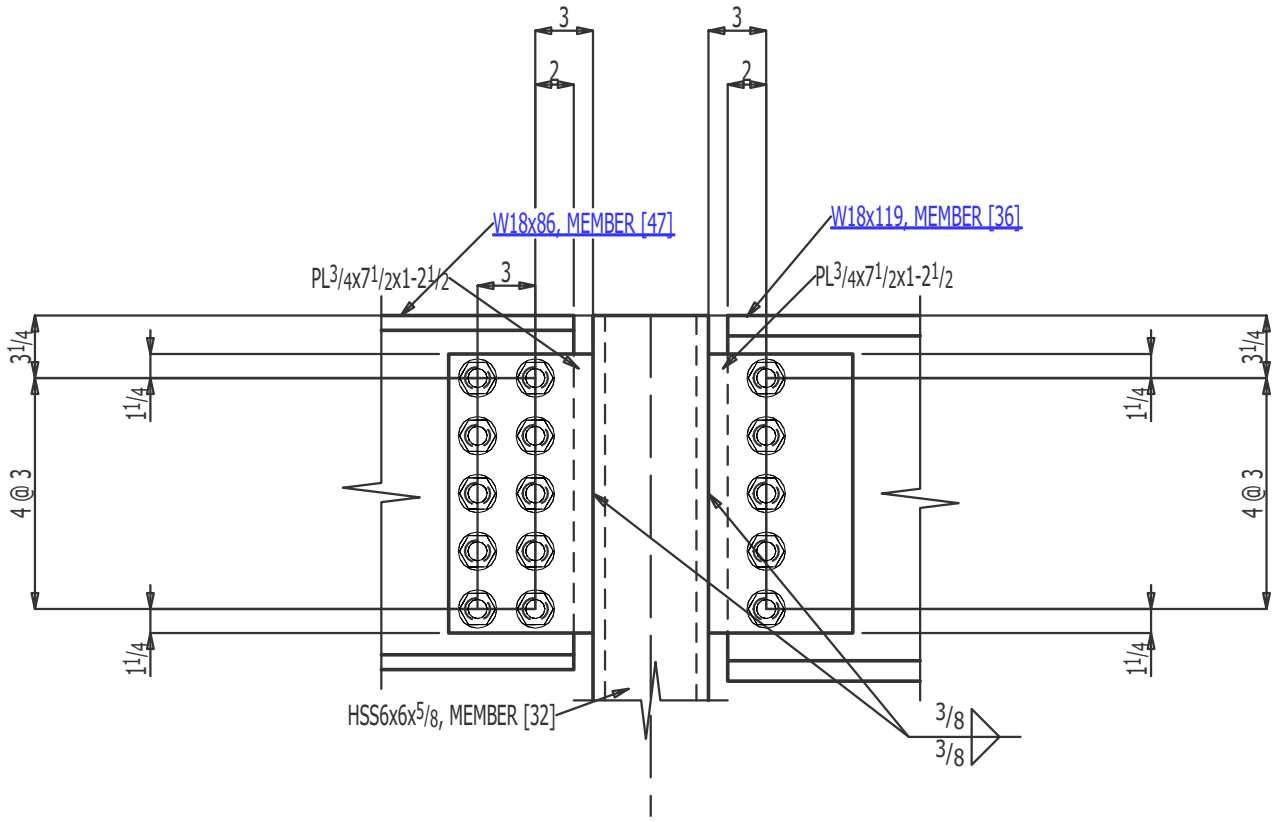
Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 10A

CON - 10A [11] at X=32-11 1/2, Y=20-3 Elev=24-0 1/2



TOP SIDE VIEW

CON - 10



A

Section A
CON - 10 ELEVATION

Beam B_36 [36]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x119
Sequence:	1
ABM:	N/Assign
Plan length:	9-9
Camber:	0.00 in
Span length:	9-9
Slope:	0.00 °
Material length:	9-5
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	19 in
Web thickness, t_w:	0.655 in
Flange width, b_f:	11.3 in
Flange thickness, t_f:	1.06 in
Design k distance, k_{des}:	1.46 in
Detail k distance, k_{det}:	1.9375 in
Distance between web toes of fillets, T:	15.125 in
Moment of inertia about the major axis, I_x:	2190 in^4

Design summary

Left end

Connection:	Web side plate
	Plate, Size as required
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	1 in
Mtrl Setback:	4 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	114.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_36 [36] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_35 [32]
Section size:	HSS6x6x5/8
End 0 elevation:	0
End 1 elevation:	24-10
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.625 in
Design thickness, t_d:	0.581 in

Design loads

***Shear*:** 114.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 45.8 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.75 in
	Width, b:	7.5 in

Connection details (continued)

	Depth, d:	14.5 in
	Weld line to bolt group c.g., a:	3 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.375 in
	Total effective weld throat, t_e:	0.53 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A490N
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	1
	Bolt rows, n:	5
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 19

Beam B_46 [47]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x86
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-7
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	18.4 in
Web thickness, t_w:	0.48 in
Flange width, b_f:	11.1 in
Flange thickness, t_f:	0.77 in
Design k distance, k_{des}:	1.17 in
Detail k distance, k_{det}:	1.625 in
Distance between web toes of fillets, T:	15.15 in
Moment of inertia about the major axis, I_x:	1530 in^4

Design summary

Right end

Connection:	Web side plate
	Plate, Size as required
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	24-10
Field Clear:	1 in
Mtrl Setback:	4 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	114.0 kips
Moment:	0.0 kip·in (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_46 [47] Connection strength check: RIGHT END

Member end summary

Connecting nodes

Node 1

Column:	C_35 [32]
Section size:	HSS6x6x5/8
End 0 elevation:	0
End 1 elevation:	24-10
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.625 in
Design thickness, t_d:	0.581 in

Design loads

***Shear*:** 114.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 64.6 % of the maximum web shear.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.75 in
	Width, b:	7.5 in

Connection details (continued)

	Depth, d:	14.5 in
	Weld line to bolt group c.g., a:	4.5 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.375 in
	Total effective weld throat, t_e:	0.53 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A490N
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	1
	Bolt rows, n:	5
	Bolt row spacing, s:	3 in
	Bolt columns, m:	2
	Bolt column spacing, s_{col}:	3 in
	Web end distance, $L_{e,w}$:	2 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 17

AISC manual extended configuration and design method

***Coefficient C'*:**	38.67 in
$6*M_{max}/F_y*d^2$:	1.82 in

Results summary

Web Side Plate on left end of Beam B_36 [36]

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Shear rupture of plate:	21	1.021	J4.2
Bolt shear of web bolts:	3	0.927	J3
Bolt bearing on plate:	110	0.913	J3.10
Bolt bearing on beam web:	110	0.854	J3.10
Shear yielding of plate:	38	0.728	J4.2
Rupture of weld to supporting member:	112	0.705	J2, Table J2.5
Block shear rupture of plate:	6	0.613	J4.3
Punching shear of support:	459	0.483	(10-7b)
Shear yielding of beam web:	2	0.458	G2.1
Shear of support:	36	0.389	J4.2
Flexure of plate:	314	0.201	Pg 10-90

Warnings

- FORCED CONNECTION
  - Engineering review required to evaluate strength and the application of SDS2 design calculations to the specific material and geometry.

Notes and conclusions

- Maximum unity ratio for design: 0.99
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- Shear tab thickness > .5 * Db + 1/16
- Supporting HSS column slenderness check

$$\left(\frac{b}{t} = 7.32702\right) \leq \left(1.4 \cdot \sqrt{\left(\frac{E}{F_y}\right)} = 35.1518\right) \quad (AISC\ B4.1).$$

- CONNECTION DESIGN FAILURE
  - Connection net shear strength exceeded ⚠

10% OVERAGE ALLOWANCE IS CONSIDERED FOR THIS CONNECTION ← **Gelinas Comment**

Web Side Plate on right end of Beam B_46 [47]

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Shear rupture of plate:	21	1.021	J4.2
Block shear rupture of plate:	6	0.879	J4.3
Rupture of weld to supporting member:	112	0.860	J2, Table J2.5
Flexure of plate:	345	0.832	Pg 10-90
Shear yielding of plate:	38	0.728	J4.2
Punching shear of support:	459	0.724	(10-7b)
Bolt bearing on plate:	20	0.706	J3.10
Bolt bearing on beam web:	20	0.660	J3.10
Bolt shear of web bolts:	3	0.660	J3
Shear yielding of beam web:	2	0.645	G2.1
Plate flexural rupture:	460	0.510	Pg 9-6
Buckling of plate:	346	0.402	Pg 9-7,8
Shear of support:	36	0.389	J4.2

Warnings

- FORCED CONNECTION
  - Engineering review required to evaluate strength and the application of SDS2 design calculations to the specific material and geometry.

Notes and conclusions

- Maximum unity ratio for design: 0.99
- See 'Web side plate connection design' notes for design criteria applicable to this connection. ⚠
- Exception (b) from design check 2 on page 10-90 applies.
- Supporting HSS column slenderness check

$$\left(\frac{b}{t} = 7.32702\right) \leq \left(1.4 \cdot \sqrt{\left(\frac{E}{F_y}\right)} = 35.1518\right) \quad (AISC\ B4.1).$$

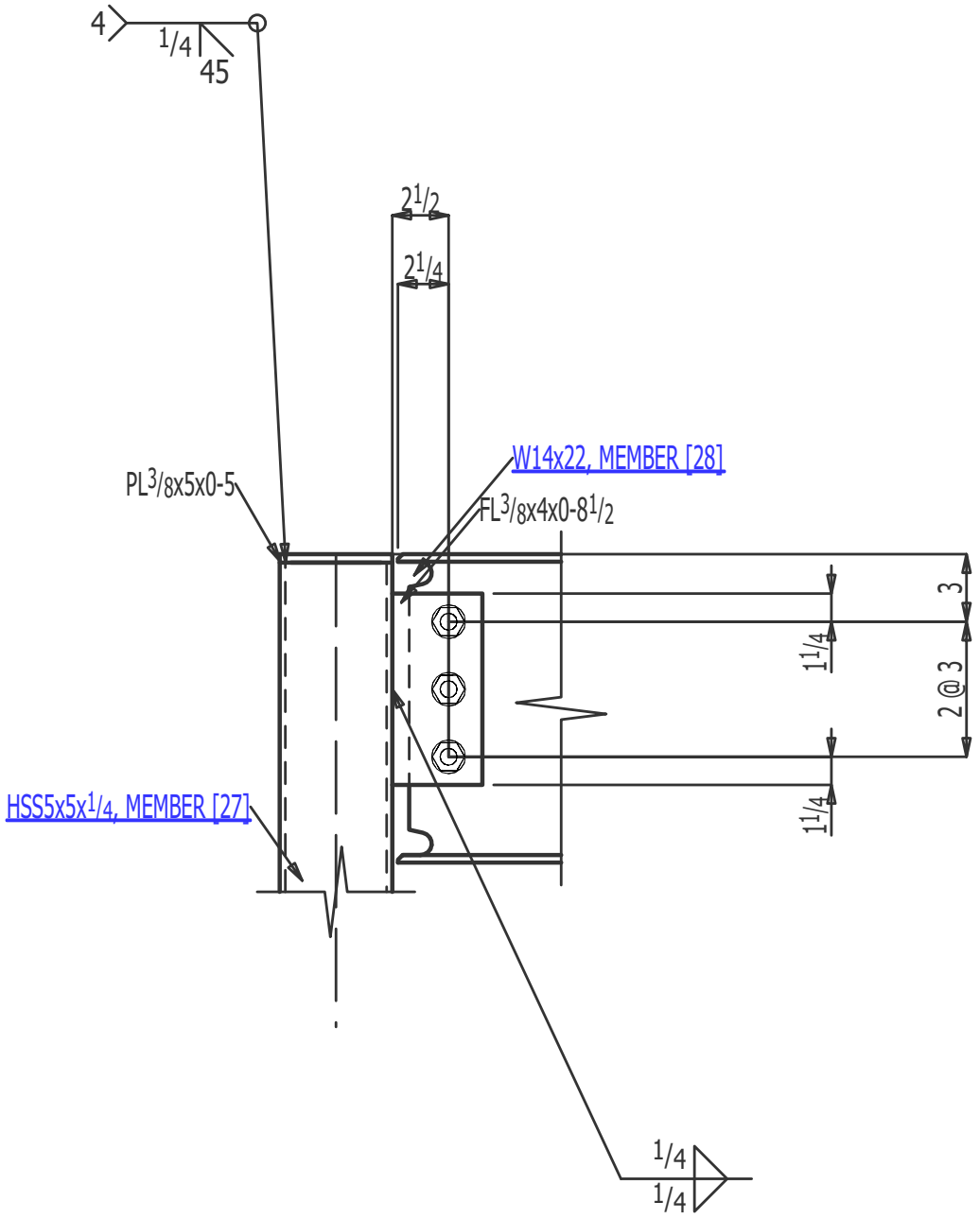
- CONNECTION DESIGN FAILURE
  - Connection net shear strength exceeded ⚠

10% OVERAGE ALLOWANCE IS CONSIDERED FOR THIS CONNECTION ← **Gelinas Comment**

11) Moment Connection – W14x22 Beam connecting to HSS5x5x1/4 Column using directly flange welded moment connection - For 18 kip-ft moment

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 11

CON - 11 [9] at X=17-8 3/4, Y=18-7 1/4 Elev=24-3 5/8



ELEVATION VIEW

CON - 11

Column C_27 [27]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	HSS5x5x1/4
Sequence:	1
ABM:	N/Assign
Length:	24-10 1/2
Rotation:	0.00 °

Section properties

Material grade:	A500B
Yield stress, F_y:	46 ksi
Tensile strength, F_u:	58 ksi
Overall width of HSS member, B:	5 in
Overall height of HSS member, H:	5 in
Nominal thickness, t_{nom}:	0.25 in
Production Standard:	ASTM A500
***Design Thickness Ratio*:**	0.93
Design wall thickness, t_{des}:	0.233 in
Gross area, A_g:	4.3 in^2
Moment of inertia about the major axis, I_x:	16 in^4

Design summary

Top end

Connection:	User column base/cap plate
	Column plate 1
	Use transverse stiffener: Never
	Weld all around: Automatic

Top end (continued)

	Web welds: Automatic
	Seal welds: Automatic
	Flange welds: Automatic
Elevation:	24-10 1/2
Minus Dim:	0 in (AUTO)
Mtrl Setback:	0.375 in
Std Detail:	None
Load:	0.0 kips
Moment:	0.0 kip·in (AUTO)
Weak axis moment:	0.0 kip·in
Uplift:	0.0 kips (AUTO)
Tying:	0.0 kips (AUTO)

C_27 [27] Connection strength check: TOP END

Member end summary

Connecting nodes

None

Design loads

***Input compression*:**	0.0 kips
***Automatic uplift*:**	0.0 kips
***Automatic moment*:**	0.0 kip·in

Connection summary

- USER DEFINED BASE PLATE

Connection details

Plate:	**Thickness, t:**	0.375 in
	Width, b:	5 in
	Length, l:	5 in
Welds to column:	**Weld metal strength, F_{exx}:**	70 ksi
	All around seal weld, size:	1/4 in
	All around seal weld, type:	Single bevel groove

Connection design lock summary

Locked Via Member Edit:	3
Locked Via User Defined:	10

Beam B_28 [28]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W14x22
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-9 1/4
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	13.7 in
Web thickness, t_w:	0.23 in
Flange width, b_f:	5 in
Flange thickness, t_f:	0.335 in
Design k distance, k_{des}:	0.735 in
Detail k distance, k_{det}:	1.0625 in
Distance between web toes of fillets, T:	11.575 in
Moment of inertia about the major axis, I_x:	199 in^4
Plastic section modulus about the major axis, Z_x:	33.2 in^3

Design summary

Left end

Connection:	Web side plate
	Plate, Size as required
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: Automatic
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
	Minimum Setup: No, Welded moment
	Re-entrant cut: FEMA
Elevation:	24-10 1/2
Field Clear:	0.75 in
Mtrl Setback:	2.75 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	18.0 kips
Story shear:	0.0 kips
Moment:	218.0 kip·in
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_28 [28] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_27 [27]
Section size:	HSS5x5x1/4
End 0 elevation:	0
End 1 elevation:	24-10 1/2
Top of bm to top of col:	0
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.25 in
Design thickness, t_d:	0.233 in

Design loads

***Shear*:**	18.0 kips
***Moment*:**	218.0 kip·in

Design load notes

- Non-composite design
- Reaction has been input
- Moment has been input
- Design reaction is 28.6 % of the maximum web shear.
- Design moment is 21.9 % of the allowable plastic moment, Mp/OMEGA.

Connection summary

- WEB SIDE PLATE WITH WELDED FLANGE MOMENT CONN.

Connection details

Web plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Depth, d:	8.5 in
Weld:	**Weld leg size, w:**	0.25 in
Web bolts:	**Bolt type:**	A325SC
	Surface:	Class A
	Mean slip coefficient:	0.30 (J3.8)
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	3
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 13

Results summary

User Column Base/Cap Plate on right end of Column C_27 [27]

Warnings

- CONNECTION STRENGTH CALCULATIONS NOT GENERATED

Shear Moment Plate on left end of Beam B_28 [28]

Beam to column flange moment connection

***Minimum column web thickness*:**	0.197 in
	J10.6, Fig. C-J10.4
d_1**:**	13.70 in
M_1**:**	218.00 kip·in
d_2**:**	17.70 in
M_2**:**	0.00 kip·in
***Story shear*:**	0.00 kips
σ_F**:**	16.31 ksi
Column resisting moment, F_y*****Z_x**/**ω**:**	209.62 kip·in

Bolt capacity notes

- It is mandatory that SC joints also meet the strength requirements to resist the factored load as a bearing bolt, type N, joint; RCSC section 5.4 commentary

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Strength of HSS column wall:	283	0.983	K1
Bolt bearing on beam web:	110	0.947	J3.10
Bolt shear of web bolts:	1	0.947	J3
Bolt bearing on plate:	110	0.947	J3.10
Shear rupture of plate:	21	0.470	J4.2
Punching shear of support:	459	0.461	(10-7b)

Limit state summary (continued)

Block shear rupture of plate:	6	0.451	J4.3
Shear yielding of plate:	15	0.392	J4.2
Shear yielding of beam web:	2	0.286	G2.1
Rupture of beam flange weld:	125	0.283	J2, Table J2.5
Shear of support:	36	0.261	J4.2
Flexural yielding of beam:	59	0.219	F

Connection strength

	Unity ratio:
Shear:	0.947
Moment:	0.983

Notes and conclusions

- Maximum unity ratio for design: 0.99
- Web side plate weld sized to develop the full plate strength.
- Eccentricity is neglected in the shear connection, misc note 33.
- Matching weld metal is required for the flange weld.
- Supporting HSS column slenderness check

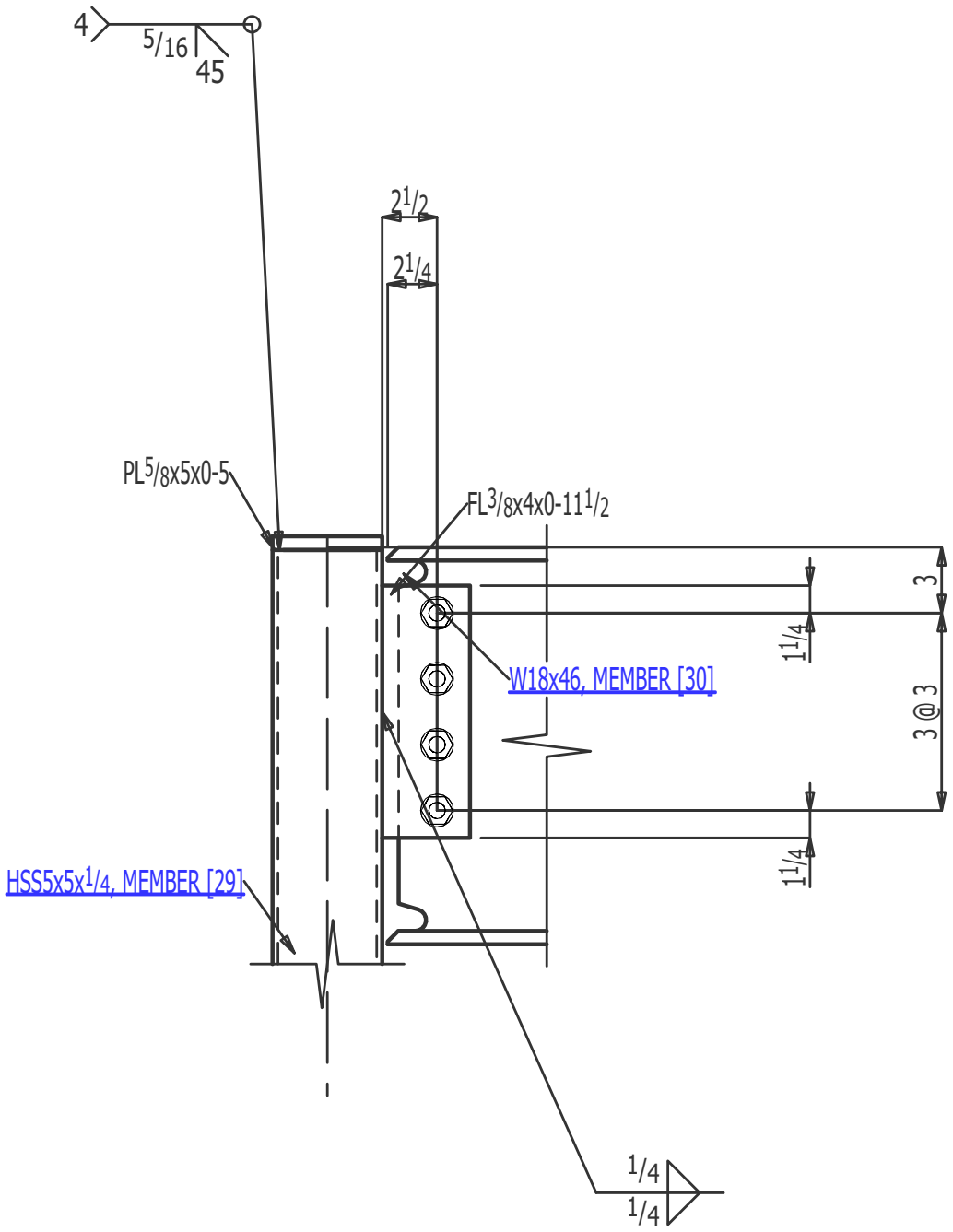
$$\left(\frac{b}{t} = 18.4592\right) \leq \left(1.4 \cdot \sqrt{\left(\frac{E}{F_y}\right)} = 35.1518\right) \quad (AISC\ B4.1).$$

- Computed beam flange force = 16.31 kips
- CONNECTION IS OK
  - Strength equals or exceeds design loads.

12) Moment Connection – W18x46 Beam connecting to HSS5x5x1/4 Column using directly flange welded moment connection - For 23 kip-ft moment

Project: 38 OLD COLONY AVENUE
Fabricator: D CRONINS WELDING SERVICE INC
Report: Connection Cube Report for CON - 12

CON - 12 [10] at X=17-8 3/4, Y=20-3 Elev=24-1 3/16



ELEVATION VIEW

Column C_29 [29]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	HSS5x5x1/4
Sequence:	1
ABM:	N/Assign
Length:	24-10 1/2
Rotation:	0.00 °

Section properties

Material grade:	A500B
Yield stress, F_y:	46 ksi
Tensile strength, F_u:	58 ksi
Overall width of HSS member, B:	5 in
Overall height of HSS member, H:	5 in
Nominal thickness, t_{nom}:	0.25 in
Production Standard:	ASTM A500
***Design Thickness Ratio*:**	0.93
Design wall thickness, t_{des}:	0.233 in
Gross area, A_g:	4.3 in^2
Moment of inertia about the major axis, I_x:	16 in^4

Design summary

Top end

Connection:	User column base/cap plate
	Column plate 1
	Use transverse stiffener: Never
	Weld all around: Automatic

Top end (continued)

	Web welds: Automatic
	Seal welds: Automatic
	Flange welds: Automatic
Elevation:	24-10 1/2
Minus Dim:	0 in (AUTO)
Mtrl Setback:	0.625 in
Std Detail:	None
Load:	0.0 kips
Moment:	0.0 kip·in (AUTO)
Weak axis moment:	0.0 kip·in
Uplift:	0.0 kips (AUTO)
Tying:	0.0 kips (AUTO)

C_29 [29] Connection strength check: TOP END

Member end summary

Connecting nodes

None

Design loads

***Input compression*:**	0.0 kips
***Automatic uplift*:**	0.0 kips
***Automatic moment*:**	0.0 kip·in

Connection summary

- USER DEFINED BASE PLATE

Connection details

Plate:	**Thickness, t:**	0.625 in
	Width, b:	5 in
	Length, l:	5 in
Welds to column:	**Weld metal strength, F_{exx}:**	70 ksi
	All around seal weld, size:	5/16 in
	All around seal weld, type:	Single bevel groove

Connection design lock summary

Locked Via Member Edit:	4
Locked Via User Defined:	10

Beam B_30 [30]

Design method

- AISC Steel Construction Manual, Fifteenth Edition (ASD)
- AISC 360-16

Overview

Section size:	W18x46
Sequence:	1
ABM:	N/Assign
Plan length:	3-0
Camber:	0.00 in
Span length:	3-0
Slope:	0.00 °
Material length:	2-9 1/4
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	18.1 in
Web thickness, t_w:	0.36 in
Flange width, b_f:	6.06 in
Flange thickness, t_f:	0.605 in
Design k distance, k_{des}:	1.01 in
Detail k distance, k_{det}:	1.25 in
Distance between web toes of fillets, T:	15.6 in
Moment of inertia about the major axis, I_x:	712 in^4
Plastic section modulus about the major axis, Z_x:	90.7 in^3

Design summary

Left end

Connection:	Web side plate
	Plate, Size as required
	No Stiffener Opposite
	Web side plate on NS
	Skew holes in beam
	Combine web side plates: Automatic
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
	Minimum Setup: No, Welded moment
	Re-entrant cut: FEMA
Elevation:	24-10
Field Clear:	0.75 in
Mtrl Setback:	2.75 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	25.0 kips
Story shear:	0.0 kips
Moment:	285.0 kip·in
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_30 [30] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_29 [29]
Section size:	HSS5x5x1/4
End 0 elevation:	0
End 1 elevation:	24-10 1/2
Top of bm to top of col:	1/2
Material grade:	A500B
Supporting member thickness, t_{sup}:	0.25 in
Design thickness, t_d:	0.233 in

Design loads

***Shear*:**	25.0 kips
***Moment*:**	285.0 kip·in

Design load notes

- Non-composite design
- Reaction has been input
- Moment has been input
- Design reaction is 19.2 % of the maximum web shear.
- Design moment is 10.5 % of the allowable plastic moment, Mp/OMEGA.

Connection summary

- WEB SIDE PLATE WITH WELDED FLANGE MOMENT CONN.

Connection details

Web plate:	**Grade:**	A36
	Tensile strength, F_u:	58 ksi
	Yield stress, F_y:	36 ksi
	Thickness, t:	0.375 in
	Depth, d:	11.5 in
Weld:	**Weld leg size, w:**	0.25 in
Web bolts:	**Bolt type:**	A325SC
	Surface:	Class A
	Mean slip coefficient:	0.30 (J3.8)
	Hole type in connection:	Short slot
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	4
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit: 14

Results summary

User Column Base/Cap Plate on right end of Column C_29 [29]

Warnings

- CONNECTION STRENGTH CALCULATIONS NOT GENERATED

Shear Moment Plate on left end of Beam B_30 [30]

Beam to column flange moment connection

***Minimum column web thickness*:**	0.197 in
	J10.6, Fig. C-J10.4
d_1**:**	18.10 in
M_1**:**	285.00 kip·in
d_2**:**	0.00 in
M_2**:**	0.00 kip·in
***Story shear*:**	0.00 kips
σ_F**:**	16.29 ksi
Column resisting moment, F_y***** Z_x**/**ω**:**	209.62 kip·in

Bolt capacity notes

- It is mandatory that SC joints also meet the strength requirements to resist the factored load as a bearing bolt, type N, joint; RCSC section 5.4 commentary

Limit state summary

	Calc. Num.	Unity ratio	AISC Ref
Bolt bearing on beam web:	110	0.988	J3.10
Bolt shear of web bolts:	1	0.988	J3
Bolt bearing on plate:	110	0.988	J3.10
Strength of HSS column wall:	283	0.981	K1
Block shear rupture of plate:	6	0.481	J4.3
Shear rupture of plate:	21	0.479	J4.2

Limit state summary (continued)

Shear yielding of plate:	15	0.403	J4.2
Punching shear of support:	459	0.350	(10-7b)
Shear of support:	36	0.268	J4.2
Shear yielding of beam web:	2	0.192	G2.1
Rupture of beam flange weld:	125	0.164	J2, Table J2.5
Flexural yielding of beam:	59	0.105	F

Connection strength

	Unity ratio:
Shear:	0.988
Moment:	0.981

Notes and conclusions

- Maximum unity ratio for design: 0.99
- Web side plate weld sized to develop the full plate strength.
- Eccentricity is neglected in the shear connection, misc note 33.
- Matching weld metal is required for the flange weld.
- Supporting HSS column slenderness check

$$\left(\frac{b}{t} = 18.4592\right) \leq \left(1.4 \cdot \sqrt{\left(\frac{E}{F_y}\right)} = 35.1518\right) \quad (AISC\ B4.1).$$

- Computed beam flange force = 16.29 kips
- CONNECTION IS OK
  - Strength equals or exceeds design loads.

20) Bearing Connection – W14x43 Beam braced with HSS5x5x_ Column using 3/4" Thick plate with (4) - ¾" ϕ A325N Bolts & 5/16" thick weld - For 19 kip Shear.

RAM Connection

Units system: English

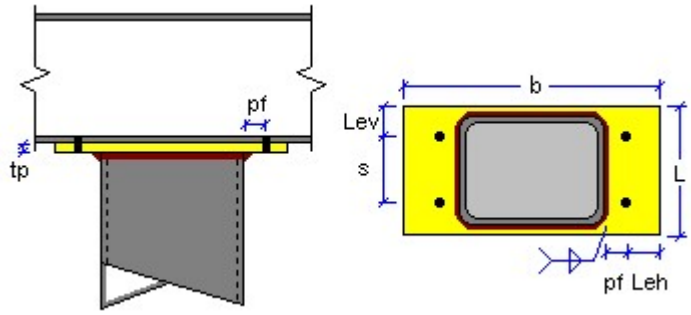
Steel Connections Data

Connection: 20

Family: Column cap (CC)
Type: Cap Plate
Description: Column cap

General information

Connector



Members

Beam

General

Beam section	:	W 14X43
Beam material	:	A992 Gr50
Built-up	:	No

Column

General

Support section	:	HSS_SQR 5-1_2X5-1_2X3_8
Support material	:	A500 GrB rectangular
Column orientation	:	Longitudinal

Cap Plate

Connector

tp: Plate thickness	:	0.75 in
Plate material	:	A36
Bolts	:	3/4" A325 N
Lev: Transverse distance to edge	:	1.75 in
Leh: Longitudinal distance to edge	:	1.5 in
pf: distance bolt centerline-tension flange	:	2 in
s: Transverse bolt spacing	:	3.5 in
Hole type on plate	:	Standard (STD)

Beam side

Hole type on beam	:	Standard (STD)

Column side

Weld to support	:	E70XX
D1: Weld size to support (1/16in)	:	5

Stiffeners

Ns: Transverse stiffeners	:	None

RAM Connection

Units system: English

Steel Connections
Results

Connection: 20

Family: Column cap (CC)
Type: Cap Plate
Description: Column cap

Design code: AISC 360-16 ASD

Demands

Description	Ru [kip]	Pu [kip]	Mu [kip*ft]	PufTop [kip]	PufBot [kip]	Load type
DL	0.00	20.00	0.00	10.00	10.00	Design

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Cap Plate						
Bolt diameter	[in]	0.75	--	1.50	✔	DG4 Sec. 1.1
Clear distance between holes (transversal)	[in]	8.69	0.75	--	✔	Sec. J3.3
Transverse center-to-center spacing (gage)	[in]	3.50	2.00	12.00	✔	Sec. J3.5
Transverse edge distance	[in]	1.75	1.00	--	✔	Tables J3.4, J3.5
Longitudinal edge distance	[in]	1.50	1.00	--	✔	Tables J3.4, J3.5
Beam						
Transverse edge distance	[in]	2.25	1.00	--	✔	Tables J3.4, J3.5
Plate (support side)						
Distance from centerline of bolt to nearer surface o...	[in]	2.00	1.25	--	✔	DG4 Sec. 2.1
Weld size	[1/16in]	5	3	--	✔	table J2.4

Design Check

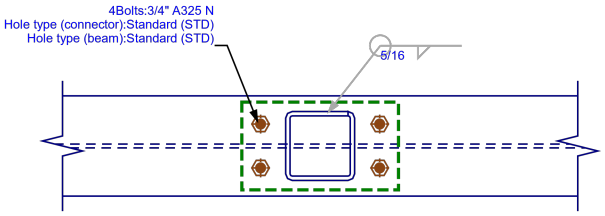
Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Cap Plate						
Resulting tension capacity due prying action	[Kip]	38.82	10.00	DL	0.26	p. 9-13

Bolt bearing under shear load	[Kip]	78.30	0.00	DL	0.00	Tables (7-1..14)
Bolt tearout under shear load	[Kip]	57.09	0.00	DL	0.00	Tables (7-1..14)
Bolts shear	[Kip]	47.74	0.00	DL	0.00	Tables (7-1..14)
Beam						
Bending	[Kip*ft]	173.65	0.00	DL	0.00	Sec. F13.1
Resulting tension capacity due prying action	[Kip]	34.75	10.00	DL	0.29	p. 9-13
Local web yielding	[Kip]	98.61	0.00	DL	0.00	Eq. J10-2
Web crippling	[Kip]	82.21	0.00	DL	0.00	Eq. J10-4
Compression buckling of the web	[Kip]	42.84	0.00	DL	0.00	Eq. J10-8
Support						
Weld capacity	[Kip]	25.52	10.00	DL	0.39	Eq. J2-3
Side wall local crippling	[Kip]	861.28	0.00	DL	0.00	Eq. J10-4
Side wall local yielding	[Kip]	210.99	20.00	DL	0.09	Eq. J4-1

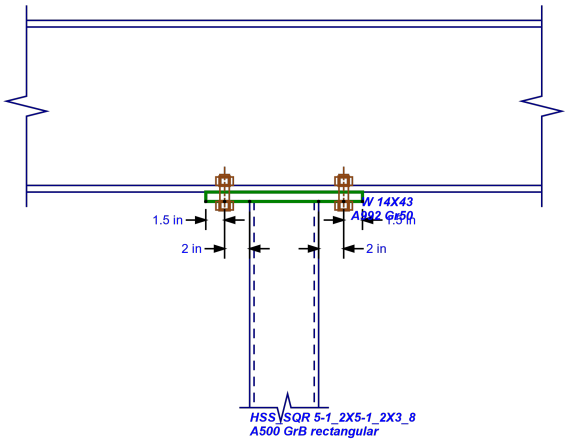
Global critical strength ratio	**0.39**

RAM Connection Standalone
Units system: s
Current Date: s

Top view



Lateral view



Front view