

STRUCTURAL CALCULATIONS

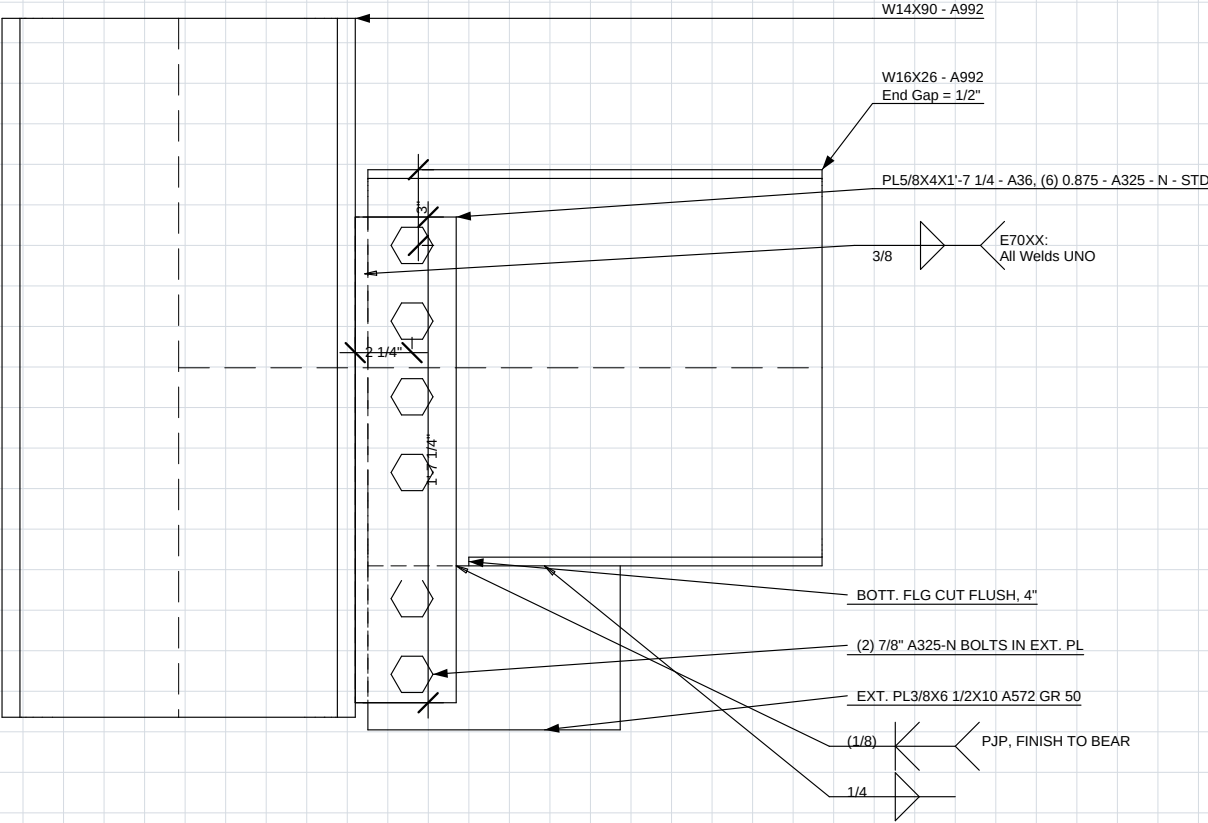
W16X26 TO W14X90 — BEAM CONNECTION TO COLUMN FLANGE

Date 2026-10-27

By Descon 8

31 checks, all pass

W16X26 TO W14X90 — BEAM CONNECTION TO COLUMN FLANGE	Description	W16X26 to W14X90 — Beam Connection to Column Flange	Date	2026-10-27
	Job no.		By	Descon 8
	Section	Design	Reviewed	



Beam Connection to Column Flange

Column: W14X90 - A992

Right Side Beam: W16X26 - A992

RSB_{Moment}	= 0 k-ft	Moment	given
RSB_{Shear}	= 120 kips	Shear	given
RSB_{Hc}	= 0 kips	Axial Force	given
RSB_{Ht}	= 0 kips	Axial Force	given

All Welds Are E70XX

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Right Side Beam				
1 Right Side Beam - W16X26 Shear Connection				
1.a Shear Connection Using One Plate:				
Plate (W x L x T): 19.25 in. X 4 in. X 0.625 in.				
Shear Connection Using One Plate:				
$db = 0.875$			given	
$MT = \frac{db}{2} + 0.0625 = \frac{0.875}{2} + 0.0625 = 0.5 \text{ in.}$			Max. Thickness	
$MT \geq \min(0.625, 0.25) \rightarrow 0.5 \text{ in.} \geq 0.25$			0.500	OK
Plate Material: A36				
$BS = 0.5 \text{ in}$			Beam Setback	given
Bolts: (6) (0.875 - A325 - N - STD): 4 in the beam web @ 3 in., 5 in. across the bottom flange, 2 in the web extension plate @ 3 in.				
Bolt Holes on Beam Web: 0.9375 in. Vert. X 0.9375 in. Horiz.				
Bolt Holes on Plate: 0.9375 in. Vert. X 0.9375 in. Horiz.				
Weld: 0.375 E70XX - Fillet Welds				
Equation [tmax is 6 * Mmax / (Fy * d²)] need not be checked				
Loading:				
$V = RSB_{\text{Shear}} = 120 \text{ kips}$			Vertical Shear	
$H = 0 \text{ kips}$			Axial Load	given
$R = (V^2 + H^2)^{0.5} = ((120 \text{ kips})^2 + (0 \text{ kips})^2)^{0.5} = 120 \text{ kips}$			Resultant	
$\Theta = \text{atan2}(V, H) = \text{atan2}(120 \text{ kips}, 0 \text{ kips}) = 90 \text{ degrees}$				
Check Bolt Spacing and Edge Distance:				
$s = 3 \text{ in.}$			Spacing	given
$MS = 2.625 \text{ in.}$			Minimum Spacing	given
$s \geq MS \rightarrow 3 \text{ in.} \geq 2.625 \text{ in.}$			0.875	OK
$dh = 0.9375$				
$Longitudinal_{\text{clear, distance, between, holes, parallel, to, load}} = s - dh = 3 \text{ in.} - 0.9375 = 2.0625 \text{ in.}$				
$LCDB_{\text{ok}} = 1$			Descon 8's verdict on Longitudinal_clear_distance_between_holes_parallel_to_load above 0.875 in. (bolt diameter), which it prints no numbers for	
$LCDB_{\text{ok}} \geq 1 \rightarrow 1 \geq 1$			1.000	OK
Distance to Horiz. Edge of PL (ev):				
$ev = 1.125 \text{ in.}$				given
$ev \geq 1.125 \rightarrow 1.125 \text{ in.} \geq 1.125$			1.000	OK
Minimum Distance to Vert. Edge of PL:				
$v = \max(2 \cdot db, 1.125) = \max(2 \cdot 0.875, 1.125) = 1.75 \text{ in.}$				
Distance to Vert. Edge of PL (eh):				
$eh = 1.75 \text{ in.}$				4 of 14 given

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$eh \geq v \rightarrow 1.75 \text{ in.} \geq 1.75 \text{ in.}$		1.000	OK	
Minimum Distance to End of Beam:				
$v_2 = \max(2 \cdot db, 1.125) = \max(2 \cdot 0.875, 1.125) = \mathbf{1.75 \text{ in.}}$				
Distance to End of Beam (Lh):				
$Lh = 1.75 \text{ in.}$			given	
$Lh \geq v_2 \rightarrow 1.75 \text{ in.} \geq 1.75 \text{ in.}$		1.000	OK	
$h_{\text{ext}} = 6.5$			given	
$t_{\text{ext}} = 0.375$			given	
$N_{\text{ext}} = 2$			given	
$CTL_{\text{ok}} = 1$		Descon 8's verdict on Connection Top Location, which it prints no numbers for given		
$CTL_{\text{ok}} \geq 1 \rightarrow 1 \geq 1$		1.000	OK	
$Connection_{\text{Depth}} = 19.25$			given	
$y_{\text{pb}} = 1.875 + Connection_{\text{Depth}} = 1.875 + 19.25 = \mathbf{21.125}$				
$d_{\text{tot}} = 15.7 + h_{\text{ext}} = 15.7 + 6.5 = \mathbf{22.2}$		plate bottom, and bottom of the web extension, below top of beam (in.)		
$y_{\text{pb}} \leq d_{\text{tot}} \rightarrow 21.125 \leq 22.2$		0.952	OK	
$CDAL_{\text{ok}} = 1$		Descon 8's verdict on Connection_Depth at least T / 2, which it prints no numbers for given		
$CDAL_{\text{ok}} \geq 1 \rightarrow 1 \geq 1$		1.000	OK	
1.b Bolt Strength:				
Load Eccentricity for Bolts (eb):				
$a = 2.25$			given	
$eb = \frac{a}{2} = \frac{2.25}{2} = \mathbf{1.125}$				
1.c Design Bolt Shear Strength - Single Plate				
$nIN = 1$			given	
$nSP = 1$			given	
$Fv = 24.3535$			given	
$nb = 6$			given	
$s_{\parallel} = 5$			given	
$y1 = 0$			given	
$y2 = s = \mathbf{3 \text{ in.}}$				
$y3 = 2 \cdot s = 2 \cdot (3 \text{ in.}) = \mathbf{6 \text{ in.}}$				
$y4 = 3 \cdot s = 3 \cdot (3 \text{ in.}) = \mathbf{9 \text{ in.}}$				
$y5 = y4 + s_{\parallel} = 9 \text{ in.} + 5 = \mathbf{14 \text{ in.}}$				
$y6 = y5 + s = 14 \text{ in.} + 3 \text{ in.} = \mathbf{17 \text{ in.}}$		bolt positions below the top bolt		
$yc = \frac{y1 + y2 + y3 + y4 + y5 + y6}{nb} = \frac{0 + 3 \text{ in.} + 6 \text{ in.} + 9 \text{ in.} + 14 \text{ in.} + 17 \text{ in.}}{6} = \mathbf{8.16667}$				
$ly = (y1 - yc)^2 + (y2 - yc)^2 + (y3 - yc)^2 + (y4 - yc)^2 + (y5 - yc)^2 + (y6 - yc)^2$ $= (0 - 8.16667)^2 + (3 \text{ in.} - 8.16667)^2 + (6 \text{ in.} - 8.16667)^2 + (9 \text{ in.} - 8.16667)^2 + (14 \text{ in.} - 8.16667)^2 + (17 \text{ in.} - 8.16667)^2 = \mathbf{210.833}$				
$y_{\text{max}} = \max(yc - y1, y6 - yc) = \max(8.16667 - 0, 17 \text{ in.} - 8.16667) = \mathbf{8.83333}$				
$C = \frac{1}{((1/nb)^2 + (eb \cdot y_{\text{max}}/ly)^2)^{0.5}} = \frac{1}{((1/6)^2 + (1.125 \cdot 8.83333/210.833)^2)^{0.5}} = \mathbf{5.77356}$				elastic method, uneven spacing
$\Phi Rn = nIN \cdot nSP \cdot C \cdot Fv = 1 \cdot 1 \cdot 5.77356 \cdot 24.3535 = \mathbf{140.606 \text{ kips}}$				
$\Phi Rn \geq RSB_{\text{Shear}} \rightarrow 140.606 \text{ kips} \geq 120 \text{ kips}$		0.853	OK	

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1.d Design Shear Strength of the Beam:		
1.d.1 Design Shear Yield Strength:		
$dw = 15.7$		given
$tw = 0.25$		given
web extension PL3/8 A572 Gr 50 (Fy 50, Fu 65, the beam's), bottom flange cut flush; 2 of the 6 bolts in it		
$A = dw \cdot tw + h_{\text{ext}} \cdot t_{\text{ext}} = 15.7 \cdot 0.25 + 6.5 \cdot 0.375 = \mathbf{6.3625 \text{ in}^2}$		
$F_y = 50$		given
$C_v = 1$		given
$R_n = 0.6 \cdot F_y \cdot A \cdot C_v = 0.6 \cdot 50 \cdot (6.3625 \text{ in}^2) \cdot 1 = \mathbf{190.875 \text{ kips}}$		
$\Phi_{R_n} = 0.9 \cdot R_n = 0.9 \cdot (190.875 \text{ kips}) = \mathbf{171.787 \text{ kips}}$		
$\Phi_{R_n} \geq RSB_{\text{Shear}} \rightarrow 171.787 \text{ kips} \geq 120 \text{ kips}$	0.699	OK
1.d.2 Design Shear Rupture Strength:		
$N = 4$		given
$An_v = (dw - N \cdot (dh + 0.0625)) \cdot tw + (h_{\text{ext}} - N_{\text{ext}} \cdot (dh + 0.0625)) \cdot t_{\text{ext}} = (15.7 - 4 \cdot (0.9375 + 0.0625)) \cdot 0.25 + (6.5 - 2 \cdot (0.9375 + 0.0625)) \cdot 0.375 = \mathbf{4.6125 \text{ in}^2}$		
$F_u = 65$		given
$R_{n_2} = 0.6 \cdot F_u \cdot An_v = 0.6 \cdot 65 \cdot (4.6125 \text{ in}^2) = \mathbf{179.887 \text{ kips}}$		
$\Phi_{R_{n,2}} = 0.75 \cdot R_{n_2} = 0.75 \cdot (179.887 \text{ kips}) = \mathbf{134.916 \text{ kips}}$		
$\Phi_{R_{n,2}} \geq RSB_{\text{Shear}} \rightarrow 134.916 \text{ kips} \geq 120 \text{ kips}$	0.889	OK

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1.e Design Shear Strength of the Plate:				
1.e.1 Design Shear Yield Strength:				
$t = 0.625$				given
$A_2 = Connection_{Depth} \cdot t = 19.25 \cdot 0.625 = 12.0313 \text{ in}^2$				
$Fy_2 = 36$				given
$Rn_3 = 0.6 \cdot Fy_2 \cdot A_2 = 0.6 \cdot 36 \cdot (12.03125 \text{ in}^2) = 259.875 \text{ kips}$				
$\Phi Rn_3 = 1 \cdot Rn_3 = 1 \cdot (259.875 \text{ kips}) = 259.875 \text{ kips}$				
$\Phi Vn = \Phi Rn_3 = 259.875 \text{ kips}$				
$\Phi Vn \geq RSB_{Shear} \rightarrow 259.875 \text{ kips} \geq 120 \text{ kips}$			0.462	OK
1.e.2 Design Shear Rupture Strength:				
$nL = nb = 6$				
$An = (Connection_{Depth} - nL \cdot (dh + 0.0625)) \cdot t = (19.25 - 6 \cdot (0.9375 + 0.0625)) \cdot 0.625 = 8.28125 \text{ in}^2$				Net Area
$Npl = 1$				given
$Fu_2 = 58$				given
$\Phi Rn_4 = 0.75 \cdot Npl \cdot An \cdot 0.6 \cdot Fu_2 = 0.75 \cdot 1 \cdot (8.28125 \text{ in}^2) \cdot 0.6 \cdot 58 = 216.141 \text{ kips}$				
$\Phi Rn_4 \geq RSB_{Shear} \rightarrow 216.141 \text{ kips} \geq 120 \text{ kips}$			0.555	OK
1.e.3 Block Shear Strength of the Plate:				
Gross Area with Tension Resistance (Agt)				
$et = 1.75$				given
$Nh = 1$				given
$sh = 3$				given
$v_3 = (et + (Nh - 1) \cdot sh) \cdot t = (1.75 + (1 - 1) \cdot 3) \cdot 0.625 = 1.09375 \text{ in}^2$				
Net Area with Tension Resistance (Ant)				
$v_4 = v_3 - (Nh - 0.5) \cdot (dh + 0.0625) \cdot t = 1.09375 \text{ in}^2 - (1 - 0.5) \cdot (0.9375 + 0.0625) \cdot 0.625 = 0.78125 \text{ in}^2$				
Gross Area with Shear Resistance (Agv)				
$v_5 = (Connection_{Depth} - ev) \cdot t = (19.25 - 1.125 \text{ in.}) \cdot 0.625 = 11.3281 \text{ in}^2$				
Net Area with Shear Resistance (Anv)				
$Nv = nb = 6$				
$v_6 = v_5 - (Nv - 0.5) \cdot (dh + 0.0625) \cdot t = 11.32813 \text{ in}^2 - (6 - 0.5) \cdot (0.9375 + 0.0625) \cdot 0.625 = 7.89063 \text{ in}^2$				
$Ubs = 1$				given
$\Phi Rn_5 = 0.75 \cdot \min(0.6 \cdot Fu_2 \cdot v_6 + Ubs \cdot Fu_2 \cdot v_4, 0.6 \cdot Fy_2 \cdot v_5 + Ubs \cdot Fu_2 \cdot v_4) = 0.75 \cdot \min(0.6 \cdot 58 \cdot (7.89063 \text{ in}^2) + 1 \cdot 58 \cdot (0.78125 \text{ in}^2), 0.6 \cdot 36 \cdot (11.3281 \text{ in}^2) + 1 \cdot 58 \cdot (0.78125 \text{ in}^2)) = 217.5 \text{ kips}$				
$\Phi Rn_5 \geq RSB_{Shear} \rightarrow 217.5 \text{ kips} \geq 120 \text{ kips}$			0.552	OK
1.e.4 Design Flexural Strength of the Plate:				
a is 2.25 in. (bolt line to weld line)				
$Zg = \frac{t \cdot Connection_{Depth}^2}{4} = \frac{0.625 \cdot 19.25^2}{4} = 57.9004 \text{ in}^3$				
$\Phi Rn_6 = \frac{0.9 \cdot Npl \cdot Fy_2 \cdot Zg}{a} = \frac{0.9 \cdot 1 \cdot 36 \cdot (57.9004 \text{ in}^3)}{2.25} = 833.766 \text{ kips}$				7 of 14

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$$\Phi_{Rn,6} \geq RSB_{\text{Shear}} \rightarrow 833.766 \text{ kips} \geq 120 \text{ kips}$$
$$yp = \frac{Connection_{\text{Depth}}}{2} - ev = \frac{19.25}{2} - 1.125 \text{ in.} = \mathbf{8.5}$$
$$Z_{net} = Zg - t \cdot (dh + 0.0625) \cdot (|y1 - yp| + |y2 - yp| + |y3 - yp| + |y4 - yp| + |y5 - yp| + |y6 - yp|)$$
$$= 57.9004 \text{ in}^3 - 0.625 \cdot (0.9375 + 0.0625) \cdot (|0 - 8.5| + |3 \text{ in.} - 8.5| + |6 \text{ in.} - 8.5| + |9 \text{ in.} - 8.5| + |14 \text{ in.} - 8.5| + |17 \text{ in.} - 8.5|) = \mathbf{38.5254 \text{ in}^3}$$

0.144

OK

plate mid-depth below the top bolt

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$$V_8 = \frac{F_u A_t \cdot l_p}{2 \cdot 0.707 \cdot F_{exx}} = \frac{58 \cdot 0.625}{2 \cdot 0.707 \cdot 70} = \mathbf{0.366236 \text{ in.}}$$

0.3662 well below 0.375 in.

Use 0.3662 in. for strength calculation.

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1.g Web Extension Plate: PL3/8 X 6 1/2 X 10, A572 Gr 50				
Bottom flange cut flush for L_cut from the beam end: there the plate's top edge bears on the web (PJP, finish to bear); beyond, 1/4 fillets both sides to the flange.				
Fy _e = 50				given
Fu _e = 65				given
L _{ext} = 10				given
L _{cut} = 4				given
w _{ext} = 0.25				given
yt = 3			yt: top bolt below top of beam	given
Bolt spacing and edge distance in the extension plate:				
e _{et} = yt + y5 – dw = 3 + 14 in. – 15.7 = 1.3				
e _{eb} = dw + h _{ext} – (yt + y6) = 15.7 + 6.5 – (3 + 17 in.) = 2.2			top bolt to plate top edge, bottom bolt to plate bottom edge (in.)	
s _{fl} ≥ MS → 5 ≥ 2.625 in.			0.525	OK
e _{eb} ≥ 1.125 → 2.2 ≥ 1.125			0.511	OK
Bolt bearing on the extension plate (J3.11): the top bolt's clear distance taken to the plate's top edge				
Lc _{e1} = e _{et} – $\frac{dh}{2}$ = 1.3 – $\frac{0.9375}{2}$ = 0.83125				
Lc _{e2} = s – dh = 3 in. – 0.9375 = 2.0625 in.				
Fb _{lim,e} = 0.75 · 2.4 · d · Fu _e = 0.75 · 2.4 · 0.875 · 65 = 102.375 kips/in.				
Rb _{ext} = min(min(0.75 · 1.2 · Lc _{e1} · Fu _e , Fb _{lim,e}) · t _{ext} , Fv) + (N _{ext} – 1) · min(min(0.75 · 1.2 · Lc _{e2} · Fu _e , Fb _{lim,e}) · t _{ext} , Fv)				
= min(min(0.75 · 1.2 · 0.83125 · 65, 102.375 kips/in.) · 0.375, 24.3535) + (2 – 1) · min(min(0.75 · 1.2 · 2.0625 in.) · 65, 102.375 kips/in.) · 0.375, 24.3535) = 42.589 kips				
Design ^{Bearing, Strength, at, Bolt, Holes, 2} = nIN · ef · (Rb _{web} + Rb _{ext}) = 1 · 0.96226 · (97.4139 kips + 42.589 kips) = 134.719 kips			beam web and extension plate	
Design ^{Bearing, Strength, at, Bolt, Holes, 2} ≥ RSB _{Shear} → 134.719 kips ≥ 120 kips			0.891	OK
What the extension plate takes from its bolts (elastic method, as 1.c):				
V _{ext} = $\frac{V \cdot N_{ext}}{nb} = \frac{(120 \text{ kips}) \cdot 2}{6} = 40 \text{ kips}$				vertical, direct share
H _{ext} = $\frac{V \cdot eb \cdot (y5 - yc + (y6 - yc))}{I_y} = \frac{(120 \text{ kips}) \cdot 1.125 \cdot (14 \text{ in.} - 8.1666667 + (17 \text{ in.} - 8.1666667))}{210.83333} = 9.3913 \text{ kips}$				horizontal, from the eccentricity
Top edge bearing on the web, finished to bear (J7(a)):				
A _{pb} = tw · L _{cut} = 0.25 · 4 = 1 in²				
Φ _{Rn,9} = 0.75 · 1.8 · min(Fy, Fy _e) · A _{pb} = 0.75 · 1.8 · min(50, 50) · (1 in²) = 67.5 kips				
Φ _{Rn,9} ≥ V _{ext} → 67.5 kips ≥ 40 kips			0.593	OK
Fillet welds, both sides, plate to flange beyond the cut (J2.4), for H_ext; size limited by the plate's thickness:				
w _{max,e} = $\frac{Fu_e \cdot t_{ext}}{2 \cdot 0.707 \cdot F_{exx}} = \frac{65 \cdot 0.375}{2 \cdot 0.707 \cdot 70} = 0.246262$				
L _w = L _{ext} – L _{cut} = 10 – 4 = 6				
Φ _{Rn,10} = 0.75 · 0.6 · F _{exx} · 0.707 · min(w _{ext} , w _{max,e}) · 2 · L _w = 0.75 · 0.6 · 70 · 0.707 · min(0.25, 0.246262) · 2 · 6 = 65.8125 kips				
Φ _{Rn,10} ≥ H _{ext} → 65.8125 kips ≥ 9.3913 kips			0.143	OK
Flexure at the end of the flange cut: top flange and web, with the extension plate below (elastic; no holes there)				
bf = 5.5				given
tf = 0.345			W16X26	given
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	Section	Variable summary	Reviewed	

Variables									
RSB_Moment		0 k-ft					Design		
RSB_Shear		120 kips					Design		
RSB_Hc		0 kips					Design		
RSB_Ht		0 kips					Design		
db		0.875					Right Side Beam		
MT		0.5 in.					Right Side Beam		
BS		0.5 in					Right Side Beam		
V		120 kips					Right Side Beam		
H		0 kips					Right Side Beam		
R		120 kips					Right Side Beam		
Theta		90 degrees					Right Side Beam		
s		3 in.					Right Side Beam		
MS		2.625 in.					Right Side Beam		
dh		0.9375					Right Side Beam		
Longitudinal_clear_distance_between_holes_parallel_to_load		8.0625 in.					Right Side Beam		
LCDB_ok		1					Right Side Beam		
ev		1.125 in.					Right Side Beam		
v		1.75 in.					Right Side Beam		
eh		1.75 in.					Right Side Beam		
v_2		1.75 in.					Right Side Beam		
Lh		1.75 in.					Right Side Beam		
h_ext		6.5					Right Side Beam		
t_ext		0.375					Right Side Beam		
N_ext		2					Right Side Beam		
CTL_ok		1					Right Side Beam		
Connection_Depth		19.25					Right Side Beam		
y_pb		21.125					Right Side Beam		
d_tot		22.2					Right Side Beam		
CDAL_ok		1					Right Side Beam		
a		2.25					Right Side Beam		
eb		1.125					Right Side Beam		
nIN		1					Right Side Beam		
nSP		1					Right Side Beam		
Fv		24.3535					Right Side Beam		
nb		6					Right Side Beam		
s_fl		5					Right Side Beam		
y1		0					Right Side Beam		
y2		3 in.					Right Side Beam		
y3		6 in.					Right Side Beam		
y4		9 in.					Right Side Beam		
y5		14 in.					Right Side Beam		
y6		17 in.					Right Side Beam		
... and 125 more									