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Project: HSS to Column Connection
Date: 07/23/15
By: Franklin Suarez

HSS to Column Weld:

Minimum Thickness per Thinner Part Joined (HSS=0.50, Column=0.75)	$w := \frac{5}{16}\ in$
Radius of Weld,	$R := 0.5 \cdot 10.75\ in = 5.38\ in$
Length of Weld,	$l := 6.283 \cdot R = 33.77\ in$
Filler Metal Strength,	$F_{E70} := 70\ ksi$
Moment of Inertia,	$I := \pi \cdot R^3 = 487.85\ \frac{in^4}{in}$
Polar Moment of Inertia,	$J := 2 \cdot I = 975.7\ \frac{in^4}{in}$
Distance to Maximum Stress,	$c := R = 5.38\ in$
Moment Demand (RISA),	$M_y := 15.708\ kip \cdot ft$
Force per inch of weld,	$f_{uby} := \frac{M_y \cdot c}{I} = 2.08\ \frac{kip}{in}$
Moment Demand (RISA),	$M_z := 35.393\ kip \cdot ft$
Force per inch of weld,	$f_{ubz} := \frac{M_z \cdot c}{I} = 4.68\ \frac{kip}{in}$
Torsion Demand (RISA),	$T := 27.827\ kip \cdot ft$
Force per inch of weld,	$f_{ut} := \frac{T \cdot c}{J} = 1.84\ \frac{kip}{in}$
Axial Demand (RISA),	$P := 5.937\ kip$
Force per inch of weld,	$f_{ua} := \frac{P}{l} = 0.18\ \frac{kip}{in}$
Shear Demand (RISA),	$V_y := 5.547\ kip$
Force per inch of weld,	$f_{uvy} := \frac{V_y}{l} = 0.16\ \frac{kip}{in}$
Shear Demand (RISA),	$V_z := 3.891\ kip$
Force per inch of weld,	$f_{uvz} := \frac{V_z}{l} = 0.12\ \frac{kip}{in}$
Peak Stress (SRSS)	$f_{u\_peak} := \sqrt{\left(f_{ua} + f_{ubz} + f_{uby}\right)^2 + \left(f_{uvy} + f_{uvz}\right)^2 + f_{ut}^{\ 2}} = 7.18\ \frac{kip}{in}$
Loading Angle,	$\theta := \text{atan}\left(\frac{f_{ua} + f_{ubz} + f_{uby}}{f_{uvy} + f_{uvz}}\right) = 87.69\ deg$

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Strength of Weld, $\phi R_n := 0.75 \cdot 0.6 \cdot 70\ ksi \cdot \left(1.0 + 0.5 \cdot \sin\left(\theta\right)^{1.5}\right) \cdot 0.707 \cdot w = 10.44\ \frac{kip}{in}$

$\phi R_n \geq f_{u\_peak}$ OK!

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Continuity Plate Check:

Demand:

Maximum Moment, $M_u := 48.868\ kip \cdot ft$

Thickness of Beam Flange, $t_b := 0.75\ in$

Depth of Beam, $d_b := 12\ in$

Flange Force, $P_{bf} := \dfrac{M_u}{d_b - t_b} = 52.13\ kip$

Due to built-up section having two webs, the flange force can be divided by two.

$$P_{web} := 0.5 \cdot P_{bf} = 26.06\ kip$$

Flange Local Buckling:

Specified Minimum Yield Stress of the Flange, $F_{yf} := 36\ ksi$

Thickness of Loaded Flange, $t_f := 0.75\ in$

$$\phi R_n := 0.9 \cdot 6.25 \cdot F_{yf} \cdot t_f^{\ 2} = 113.91\ kip$$

If the concentrated force to be resisted is applied at a distance from the member end that is less than $10 \cdot t_f = 7.5\ in$, Rn shall be reduced by 50%. Load applied at a greater distance, reduction not applicable.

$\phi R_n \geq P_{web}$ OK!

Web Local Yielding:

Specified Minimum Yield Stress of Web Material, $F_{yw} := 36\ ksi$

Thickness of Web, $t_w := 0.75\ in$

Distance from outer face of flange to the web toe of the fillet, $k := 0.75\ in$

Length of applied load = thickness of beam flange or flange plate. $l_b := 0.75\ in$

Depth of Column, $d_c := 12\ in$

When the concentrated force to be resisted is applied at a distance from the member end that is greater than the depth of the member, dc=12in.

$$\phi R_n := 1.0 \cdot F_{yw} \cdot t_w \cdot (5 \cdot k + l_b) = 121.5\ kip$$

When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal the depth of the member, dc=12in.

Governs - $\phi R_n := 1.0 \cdot F_{yw} \cdot t_w \cdot (2.5 \cdot k + l_b) = 70.88\ kip$

$\phi R_n \geq P_{web}$ OK!

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Web Local Crippling:

When the concentrated compressive force to be resisted is applied at a distance from the member end that is greater than or equal to $0.5 \cdot d_c = 6\ in$

Modulus of Steel, $E := 29000\ ksi$

$$\phi R_n := 0.75 \cdot 0.8 \cdot t_w^{\,2} \cdot \left(1 + 3 \cdot \left(\frac{l_b}{d_c}\right) \cdot \left(\frac{t_w}{t_f}\right)^{1.5}\right) \cdot \sqrt{\frac{E \cdot F_{yw} \cdot t_f}{t_w}} = 409.5\ kip$$

AISC Specifications met, continuity plates are not required

Panel Zone Check:

Column Shear, $P_r := \dfrac{M_u}{d_b - t_b} = 52.13\ kip$

Yield Strength of Column, $F_y := 36\ ksi$

Area of Column, $A_g := 30.75\ in^2$

Yield Strength of Column, $P_c := F_y \cdot A_g = 1107\ kip$

Axial Force Ratio,

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if P_r ≤ 0.4·P_c   = "J10-9"
  ‖ "J10-9"
else
  ‖ "J10-10"
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Web Nominal Shear Strength, $\phi R_n := 0.9 \cdot 0.6 \cdot F_y \cdot d_c \cdot 2 \cdot t_w = 349.92\ kip$

$\phi R_n \geq P_r$ OK!

The column web has sufficient shear strength, no doubler plates are required.