Remember!

Due to the flexibility of their walls, local strength of an HSS at the connection may control the capacity of the connection.

This is very different than designing for Wide Flange (WF) supports.

Understanding this while sizing members will mean efficient and economical connections without the need for costly stiffeners or reinforcing.

Types of HSS-Wide Flange Moment Connections
- Continuous beam w/column below (and above and below)
- Directly welded
- Through-plate
- Cut out/exterior diaphragm plate
- Welded tee flange
- End plates
- Interior diaphragm plate
Continuous Beam – single story

- Don’t forget to brace the bottom of the beam!

Continuous Beam – column splice

- Beam web can be stiffened with plates or with split HSS sections to match the column section
- Beam flange should be wider than HSS
- Moment transfer limited by bolts, beam flange thickness, cap/base plate thickness

Directly Welded

- Can be used to transfer larger moments to the HSS column
- Capable of developing the available flexural strength of the column, but rarely of the WF beam due to flexibility of HSS wall
Directly Welded

• For best results, use HSS with thick wall and WF with flange width approximately equal to HSS flat dimension (B-3t)

Directly Welded Connection Example

• AISC Manual Chapter K

<table>
<thead>
<tr>
<th>TABLE K1.2A</th>
<th>Limits of Applicability of Table K1.2</th>
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</table>

Directly Welded Connection Example

• Check HSS8x8x3/8 column (A1085) and W16x36 beam (A992)
• $M_u = 66$ k-ft
• $T_u = C_u = 66/12/d = 51$ k
• Limits from Table K1.2A:
  • $B/t = 21.3 < 35$, OK
  • $0.25 < b/B = 0.87 < 1.0$, OK
Directly Welded Connection Example

• AISC Manual Chapter K

Directly Welded Connection Example

• Check beam and HSS failure modes, Transverse Plate connection (AISC table K1.2):
  • Local yielding of beam flange:
    • $\Phi R_{li} = 10(B/2h)F_{lt}b_i \leq F_{lt}b_i$
    • $\Phi R_{li} = 58.4 \, k > 51 \, k \, OK$

Directly Welded Connection Example

• HSS shear yielding/punching
  • Applicable when: $0.65b_i \leq b_i \leq B-2b_i$; therefore this needs to be checked
  • $\Phi R_{ls} = 0.6F_{lt}(2h+2B_{ml})$
    • $B_{ml} = 10b_i(B/b_i) = 3.28 \leq b_i$
  • $\Phi R_{ls} = 79.3 \, k < 51 \, k \, OK$
Directly Welded Connection Example

- Local yielding of HSS sidewalls
  - $\beta = 0.873 \neq 1.0$; therefore this condition need not be checked

Directly Welded Connection Example

- Local crippling of HSS sidewalls
  - $\beta = 0.873 \neq 1.0$; therefore this condition need not be checked

  - Controlling check:
    - $\Phi R_n = 58.4 \text{k} > R_u = 51 \text{k}$
    - Therefore connection is okay

Through-Plate Connection

- Cut column in two locations to allow top and bottom flange plates to pass through
- Must allow for combined effects of mill, fabrication, and erection tolerances (use shims in the field)
- Use PJP or CJP weld for plate to column connection instead of fillet if necessary
Through-Plate Connection

- Shop-weld with stub beams or field weld
- Strap angles

Through Plate Connection Example

- $V_u = 1.2(5) + 1.6(15) = 30\, \text{k}$
- $M_u = 1.2(35) + 1.6(60) = 138\, \text{k-ft}$

- Check using spec section J4
- Approximate maximum force in plate assuming $t_p = t_f$
  - Approx $R_u = \frac{M_u}{d+t_f}$
  - Approx $R_u = 117\, \text{k}$
  - Use approx $R_u$ to find actual plate thickness
Through Plate Connection Example

- Tensile yielding of through plate
  - \( A_g = \frac{R_u}{\phi F_y} \)
  - \( b_p = \text{col dim} + 2 = 14'' \)
  - \( t_p = 5/16'' \)
- Find actual \( R_u \)
  - \( R_u = \frac{M}{d + t_p} \)
  - \( R_u = 117 \text{ k} \)
- Tensile rupture of through plate
  - \( \phi R_n = \phi F_y A_\text{ge} \)
  - \( A_n = A_g - 2(d_p + 1/16)t_p = 3.83 \text{ in}^2 \)
  - \( A_n = A_g - 0.85A_g = 3.72 \text{ in}^2 \)
  - \( \phi R_n = 162 \text{ k} > R_u \text{ OK} \)

Through Plate Connection Example

- Compressive yielding of through plate
  - \( n = R_u/\phi F_y \)
    - \( \phi F_y \) per AISC table 7-1
  - \( n = 6.3 \rightarrow \text{use } n = 8 \text{ (two rows of four bolts)} \)
  - Use 3" bolt spacing
  - Use 1" edge distance (AISC table J3.4)
  - Use \( g = 3.5'' \) (AISC table 1-1)
Through Plate Connection Example

- Bolt bearing on through plate
  - $R_{n,end}=12.9k$
  - $R_{n,int}=32.6k$
  - Total $\Phi R_n = 166 k > R_u$ OK

- Bolt bearing on beam flange
  - $R_{n,end}=45.1k$
  - $R_{n,int}=45.1k$
  - Total $\Phi R_n = 270 k > R_u$ OK

Through Plate Connection Example

- Block shear rupture of through plate
  - $L_y=10^\prime, A_y=6.25 \text{ in}^2, A_n=4.34 \text{ in}^2, A_{nt}=0.82 \text{ in}^2$
  - $\phi R_n = 137 k > R_u$ OK

- Block shear rupture of beam flange
  - $L_y=11^\prime, A_y=8.47 \text{ in}^2, A_n=6.11 \text{ in}^2, A_{nt}=0.91 \text{ in}^2$
  - $\phi R_n = 223 k > R_u$ OK

- Fillet weld size – design for combined axial force and moment
  - $D_{req}=4.63 \text{ 16ths} \rightarrow 5/16^\prime$ weld

Cut-Out Plate Connection

- Top and bottom plates fit around the column (column is continuous)
- Plates wider than through plate connection
- Plates can be field welded or shop welded
- Can be used to transfer moment to column instead of through (framing on one side only)
Cut-Out Plate Connection

- Perpendicular framing is ideally shallower than (or at least same depth as) moment connected beams
- Beam stubs can work well with this connection also!

Cut-Out Plate Connection Example

- Check 5/8" thick cut-out plates, 50 ksi
- Find plate dimensions
  - Controlling plate location is adjacent to column
- Find \( w_s \) dimension based on tension in the plate
  - \( T_u = M_u \frac{12}{(d + t_p)} = 183 \) k
  - \( w_s = \frac{T_u}{(\Phi^2 F_y t_p)} = 3.25 \) inches
- Total plate width = HSS dimension + 2\( w_s \) = 14.5 inches
- Use plate width = 15 inches
Cut-Out Plate Connection Example

• Check b/t ratio of plate projection
  • $w_s/t = 3.5/0.625 = 5.6$
  • Per AISC table B4-1a, case 3 limit = $0.45(E/F_y)^{0.5} = 10.8$ OK

• Plate width of 15 inches OK

Cut-Out Plate Connection Example

• Check bolt shear, bolt bearing, and block shear similar to Through Plate Connection example

Welded Tee Flange

• Convenient when connecting to only one side of the column
• Moment transfers to the column rather than through the column
Welded Tee Flange

- Ideally tees would be longer than the column width to make welding easier
- Shear can be transmitted via stiffened seat connection or shear plate

Cut-Out Plate Connection – one sided

- Stronger than welded tee connection
- Check weld transferring shear from plate to HSS wall
- Shear plate or stiffened seat

End Plate

- Flange width of beam should be as large or larger than column width
- Buckling strength of HSS side wall should be checked.
Interior Diaphragm

- Plates are shop welded inside the column to align with beam flanges
- Column only has to be cut once (as opposed to twice with through plate connection)
- The internal weld is difficult to make
- Diaphragm plates are not visible in the finished condition

Column Face Reinforcement – Doubler Plates

- Most direct approach for a moment connection to a face of an HSS may include reinforcing
- Research done by Dawe and Guravich published in Canadian Journal of Civil Engineering (1993)
- Results demonstrate 4 basic failure modes

Column Face Reinforcement – Failure Modes

(a) Rupture of beam tension flange
(b) Punching shear failure of reinforcing plate
(c) Web crippling of column side walls near compression flange
(d) Punching shear of column face
Column Face Reinforcement - Angles

- Angles welded to HSS and to each other at toes
- Assembly then drilled for field bolting

Column Face Reinforcement – Stiffening Tees

- Broaden beam flanges with tee stiffeners to deliver the flange force to column side walls

Column Face Reinforcement – Concrete Filled Tubes

- Concrete filling hollow sections greatly enhances performance under transverse compression
- In tension region, connection will pull the column face outwards, as with an unfilled column
- In some research, the yielding load in tension region increased in filled tube over unfilled tube, but not all.
- Therefore, it’s prudent to base the connection design in a tension region on an unfilled tube.
Moment Connection Research

- Lots of research into ductility of connections and their use in moment frames
- Some connections were found to exhibit brittle failure after Kobe earthquake
- Testing to improve ductility of connections
  - Improve beam copes (where used)
  - Improve welding procedures

Moment Connection Research

- Through plate connection testing showed equivalence to pre-qualified connections meeting FEMA acceptance criteria
- Bolted connections found to have less potential for brittle failure than welded

Moment Connection Research

- Dr. Jason McCormick – U of Michigan
  - “Seismic Applications of Hollow Structural Sections in Moment Resisting Frames”
  - Limits for width to thickness ratios established
  - Undergoing some FEA tests on connections
  - Presentation at NASCC – available to watch on AISC’s website
Connection Design Resources & Presentation References

- AISC Manual Part 12
- AISC Design Guide 24
- CIDECT Design Guide 9
- Hollow Structural Section Connection and Trusses by J.A. Packer & J.E. Henderson
- STI's Contact page
Software Capabilities

<table>
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<th>Program</th>
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ECA Engineer Design Forms development in 2014

Connection Software

New Design Resource!!!

- Will be available later this month after final verifications and documentation is complete on STI’s website!

http://steeltubeinstitute.org/hss/design-aids/