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HSS

ARTICLE

PLATE-REINFORCED HSS CONNECTIONS

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Reinforcement of HSS connections is something that can be avoided at the design stage by judicious selection of HSS members. In fact, a User Note to Section K1 of the *Specification* (AISC, 2016) states that … "Connection strength is often governed by the size of HSS members, especially the wall thickness of truss chords, and this must be considered in the initial design. To ensure economical and dependable connections can be designed, the connections should be considered in the design of the members." Unfortunately, this is not always done. Sometimes, connection design is performed even after members are procured, leading to some connections requiring reinforcement. The *Specification* (AISC, 2016), the *Manual* (AISC, 2017) and AISC Design Guide No. 24 (Packer et al., 2010) do not cover reinforced HSS connections, but some advice and guidance is given elsewhere (Packer and Henderson, 1997; CEN, 2005, 2019; Wardenier et al., 2008; Packer et al., 2009). To strengthen HSS connections there are generally two options: (i) to fill the main through (chord) member with concrete or grout, or (ii) to weld on steel reinforcing plates. The first option (concrete filling) has been described in a previous STI Technical Article (Packer, 2020). This article deals with methods for HSS connection reinforcement by the addition of steel plates. Such reinforcement can enhance both the fatigue life and static strength of welded HSS connections, with the static strength considered herein.

PLATE-REINFORCED ROUND HSS CONNECTIONS

INTERNAL RING STIFFENERS

For round-to-round HSS connections, internal ring stiffeners have been widely used in offshore steel structures, yet quantitative guidance for the design of such connections is not given in any of the major offshore design codes. Qualitatively, experimental research has shown that the ultimate strength of ring-stiffened T-connections is typically around twice that of their unstiffened counterparts, when three annular rings are used and aligned with the crown and saddle positions of the branch (Ramachandra Murthy et al., 1992; Thandavamoorthy et al., 1999). Lee and Llewelyn-Parry (2004) found that the most efficient stiffening (for strength enhancement) was obtained by placing a single stiffener centrally under a branch, at the saddle position. The capacity of a T-connection with this single stiffener was shown to be given by the sum of the strengths of the unstiffened connection and of the stiffener. The unstiffened connection nominal strength can be obtained from AISC 360-16 Table K3.1 (AISC, 2016) and the stiffener nominal strength from (Lee and Llewelyn-Parry, 2004):

$$P_n = 0.3F_{ys}t_s d_s(\beta + 2.653)\left(0.325 - 0.022\frac{t_s}{t}\right)\left(4.385 + \frac{d_s}{D}\right)$$
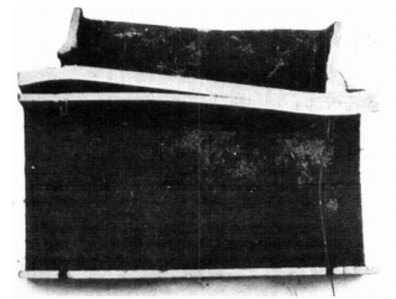
Equation 1

where F_{ys} is the yield stress of the stiffener, t_s is the thickness of the stiffener, d_s is the depth of the annular stiffener (outside radius minus the inside radius), β is branch-to-chord diameter ratio, t is the chord thickness, and D is the chord diameter. Eq. (1) is empirically derived and has the validity range of: $0.1 \le d_s/D \le 0.2$, $0.4 \le t_s/t \le 0.8$, $20 \le 2\gamma = D/t \le 60$, and $0.33 \le \beta \le 0.8$. Using an internal ring stiffener within a large-diameter chord member of an offshore structure is a feasible proposition, especially as the connection region is often composed of a short-length chord "can" which enables access inside the chord member. For most onshore structures, internal stiffening of the chord member is impractical and external stiffening would be favored.

EXTERNAL PLATE STIFFENERS

To enhance the strength of connections with thin-walled chords, reinforcement with doubler or collar plates may be used. **Doubler plates** are welded to the chord member, with the branch(es) welded to the doubler plate (Figure 1(a)). The doubler plate is normally welded only around its perimeter, to the chord. This is an efficient reinforcing method for branches in axial compression, where the capacities of the chord face and the doubler plate are additive. However, for branches loaded in axial tension or bending, one must be aware that the doubler plate separates from the chord face when loaded in tension (Figure 1(b)). In such cases the connection capacity can be conservatively taken as that of the doubler plate alone (and hence the doubler plate must be thicker than the chord member).

(a)

(b)

Figure 1: Connection reinforcement with doubler plates: (a) example showing doubler plates beneath specific (compression) branches; (b) longitudinal section of connection showing doubler plate separation under branch in-plane bending

In the case of **collar plates**, each branch is welded to the chord member while the collar – consisting of two or four parts – is welded to both the chord and the branch, as shown in Figure 2. Collar plates generally give a greater enhancement of the connection strength than a doubler plate. For connections loaded predominantly by branch in-plane bending, collar plates with two parts parallel to the chord axis are the preferred option, whereas for connections loaded predominantly by branch out-of-plane bending the two parts perpendicular to the chord axis are preferred (see Figure 2(a)). The 4-part collar plate is recommended for branch axial tension; all options are suitable for branch axial compression.

(a)   (b)
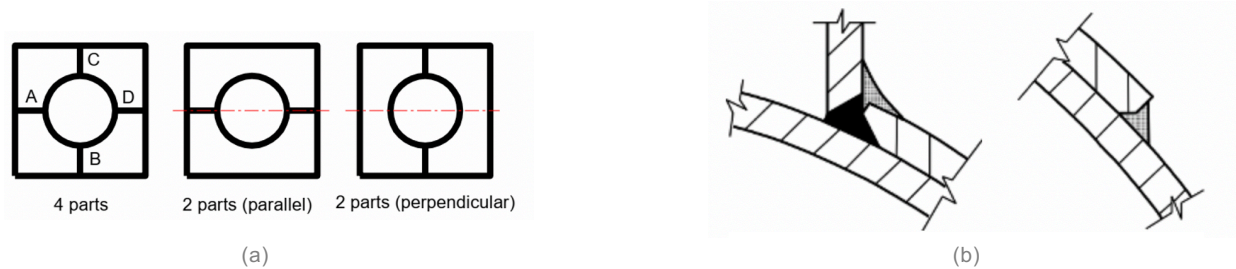
Figure 2: Connection reinforcement with collar plates: (a) 4-part and 2-part collar arrangements;
(b) welding of the collar plates to both the branch and chord members

Considerable research has been performed on plate-reinforced connections by Choo et al. (2004a, 2004b, 2005) and van der Vegte et al. (2005). As shown by their example in Figure 3, the normalized strength of a reinforced connection (vertical axis) is a function of: $2\gamma = D/t$; β; $\tau_c = t_s/t$; l_c/d_1 = length of reinforced connection region measured in the chord axis direction, relative to the branch diameter; and the type of loading.

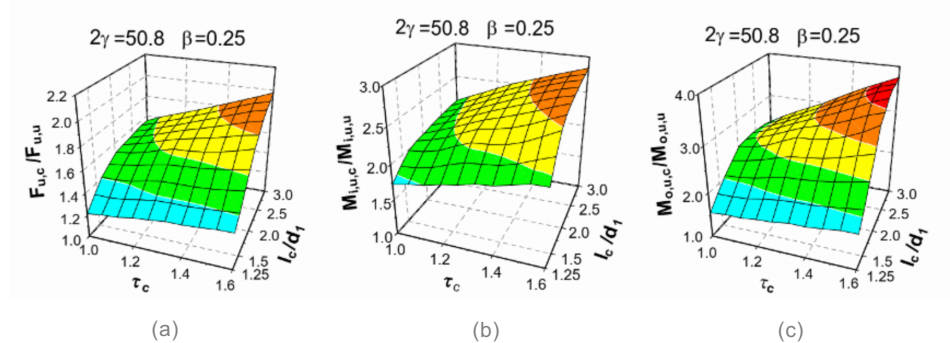(a)   (b)   (c)

Figure 3: Nominal strength of X-connections reinforced with collar plates, relative to their unreinforced counterparts, for: (a) branch axial loading; (b) branch in-plane bending; (c) branch out-of-plane bending

PLATE-REINFORCED RECTANGULAR HSS CONNECTIONS

The type of reinforcement required depends upon the governing failure mode (limit state) that causes the inadequate connection capacity. There are various types of plate reinforcement for rectangular HSS:
(i) Plates welded to the chord connecting face (to address chord face plastification, chord face punching shear, or local yielding of a branch due to uneven load distribution);
(ii) A pair of side plates welded to the chord sidewalls (to address chord sidewall yielding, chord sidewall buckling, or shearing of the chord cross section);
(iii) In order to avoid partial overlapping of branch members in a K-connection, the branch members may be welded to a single division plate (Figure 4(a)).

Combinations of these reinforcing methods may also be used; e.g., Figure 4(b). The use of a division plate (Figures 4(a) and 4(b)) is recommended over butting two double-mitered branches together (Figure 4(c)) because of the difficulty associated with proper welding of the branch webs to each other.

(a)   (b)   (c)
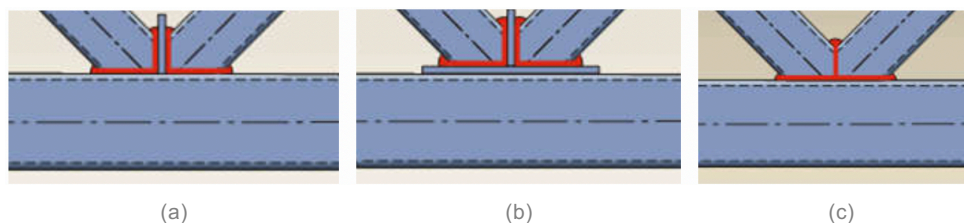
Figure 4: Overlapped K-connections, with: (a) branches welded to a division plate; (b) branches welded to both a division plate and a chord flange reinforcing plate; (c) branches welded to each other (not recommended)

REINFORCED T-, Y- AND CROSS-CONNECTIONS

Design recommendations from Eurocode 3 (CEN, 2019), adapted to AISC 360-16 for the possible limit states, are given in Tables 1, 2 and 3. A list of Notation (definition of symbols) follows the tables. The grade of steel used for the reinforcement should not be lower than that of the chord member.

<u>For T-, Y- and Cross-connections reinforced with a flange plate</u>, there is a difference in behavior of the stiffening plate depending on the sense of the load in the branch member. With a **tension load** in the branch, the plate tends to lift off the chord member (see Figure 1(b)) and behave as a plate clamped (welded) along its four edges. The strength of the connection depends primarily on the plate geometry and properties, instead of the chord connecting face, which is reflected in Table 1. With a **compression load** in the branch, Korol et al. (1982) have shown that the plate and the chord connecting face can be expected to both contribute to the connection resistance, so for the limit states of chord plastification and chord punching shear the chord-face contribution and the flange-plate contribution are added, assuming unconnected elements (see Table 2).

Table 1: Available strengths for welded, flange-plate-reinforced, rectangular HSS T-, Y- and Cross-connections under branch tension
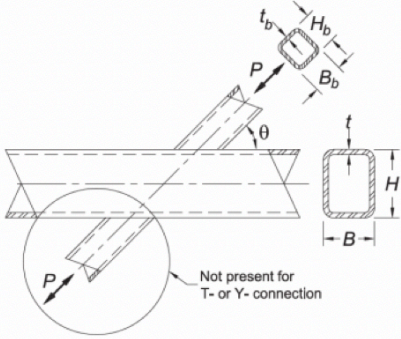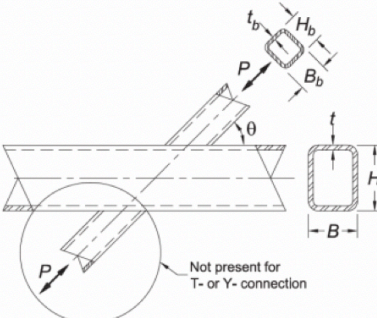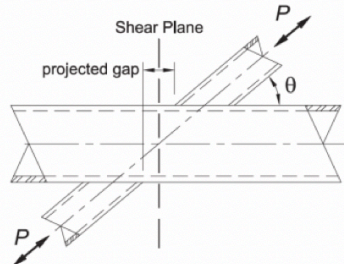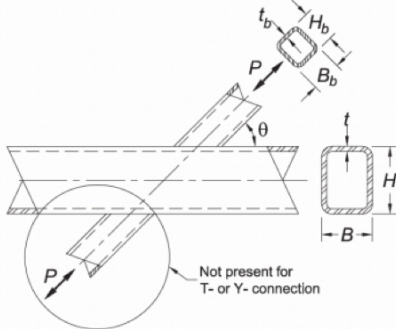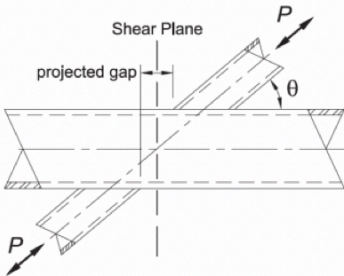
Reinforced with a Flange Plate	
Under Branch Tension Loading	**Connection Available Axial Strength**
Note: Unreinforced connection is shown. For the reinforced connection, a plate stiffener with dimensions of B_s x L_s x t_s is added under branch. $L_s \geq \dfrac{H_b}{\sin\theta} + \sqrt{B_s(B_s - B_b)}$ $L_s \geq 1.5\left(\dfrac{H_b}{\sin\theta}\right)$ and $B_s \geq B - 2t$ 	Limit State 1: Plastification of Stiffener $P_n \sin\theta = F_{ys}t_s^2\left(\dfrac{2H_b/(B_s\sin\theta)}{1-(B_b/B_s)} + \dfrac{4}{\sqrt{1-(B_b/B_s)}}\right)$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD) Limit State 2: Shear Yielding (Punching Shear) of Stiffener $P_n \sin\theta = 0.6F_{ys}t_s\left(\dfrac{2H_b}{\sin\theta} + 2B_{eps}\right)$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD) Limit State 3: Local Yielding of the Branch/Branches due to Uneven Load Distribution $P_n = F_{yb}t_b\left(2H_b + 2B_{es} - 4t_b\right)$ $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD) Limit State 4: Local Yielding of the Chord Sidewalls $P_n \sin\theta = 2F_y t\left(7.5t + 5t_s + \dfrac{H_b}{\sin\theta}\right)$ $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD) Limit State 5: Shear of the Chord Sidewalls in Gap Region $P_n \sin\theta = 0.6F_y\left[2t(H - 3t)\right]$ $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

Reinforced with a Flange Plate	
Under Branch Compression Loading	**Connection Available Axial Strength**
Note: Unreinforced connection is shown. For the reinforced connection, a plate stiffener with dimensions of B_s x L_s x t_s is added under branch. $L_s \geq \dfrac{H_b}{\sin\theta} + \sqrt{B_s(B_s - B_b)}$ $L_s \geq 1.5\left(\dfrac{H_b}{\sin\theta}\right)$ and $B_s \geq B - 2t$ Case for checking the Limit State of Shear of Chord Side Walls 	**Limit State 1: Plastification of Stiffener + Chord** $P_n \sin\theta = F_{ys}t_s^2\left(\dfrac{2H_b/(B_s\sin\theta)}{1-(B_b/B_s)} + \dfrac{4}{\sqrt{1-(B_b/B_s)}}\right)$ + $F_y t^2\left(\dfrac{2H_b/(B\sin\theta)}{1-(B_b/B)} + \dfrac{4}{\sqrt{1-(B_b/B)}}\right)Q_f$ ϕ = 1.00 (LRFD) Ω = 1.50 (ASD) **Limit State 2: Shear Yielding (Punching Shear) of Stiffener + Chord** $P_n \sin\theta = 0.6F_{ys}t_s\left(\dfrac{2H_b}{\sin\theta} + 2B_{eps}\right)$ + $0.6F_y t\left(\dfrac{2H_b}{\sin\theta} + 2B_{ep}\right)$ ϕ = 1.00 (LRFD) Ω = 1.50 (ASD) **Limit State 3: Local Yielding of the Branch/Branches due to Uneven Load Distribution** $P_n = F_{yb}t_b\left(2H_b + 2B_{es} - 4t_b\right)$ ϕ = 0.95 (LRFD) Ω = 1.58 (ASD) **Limit State 4: Local Yielding of the Chord Sidewalls** $P_n \sin\theta = 2F_y t\left(7.5t + 5t_s + \dfrac{H_b}{\sin\theta}\right)$ ϕ = 1.00 (LRFD) Ω = 1.50 (ASD) **Limit State 5: Local Crippling of the Chord Sidewalls** $P_n \sin\theta = 1.6t^2\left(1 + \dfrac{3H_b/\sin\theta}{H}\right)\sqrt{EF_y}Q_f$ ϕ = 0.75 (LRFD) Ω = 2.00 (ASD) **Limit State 6: Buckling of the Chord Sidewalls** $P_n \sin\theta = \left(\dfrac{48t^3}{H-3t}\right)\sqrt{EF_y}Q_f$ ϕ = 0.90 (LRFD) Ω = 1.67 (ASD) **Limit State 7: Shear of the Chord Sidewalls in Gap Region** $P_n \sin\theta = 0.6F_y\left[2t(H-3t)\right]$ ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)

For T-, Y- and Cross-connections reinforced with a pair of plates to the outside of the two sidewalls of the chord, design recommendations are given in Table 3. In this table, limit state expressions pertaining to chord sidewall failure are adjusted to take account of the contribution of the reinforcing plates (CEN, 2019).

Reinforced with 2 Sidewall Plates	
Under Branch Compression/Tension Loading	**Connection Available Axial Strength**

Note: Unreinforced connection is shown. For the reinforced connection, 2 plate stiffeners with dimensions of H x L_s x t_s are welded to each sidewall of the chord.	**Limit State 1: Chord Plastification**
	$$P_n \sin\theta = F_y t^2 \left(\frac{2H_b / (B\sin\theta)}{1-(B_b/B)} + \frac{4}{\sqrt{1-(B_b/B)}} \right) Q_f$$
	ϕ = 1.00 (LRFD) Ω = 1.50 (ASD)
	Limit State 2: Chord Shear Yielding (Punching Shear)
$L_s \geq 1.5\left(\dfrac{H_b}{\sin\theta}\right)$ for T- and Y-connections	$$P_n \sin\theta = 0.6F_y t \left(\frac{2H_b}{\sin\theta} + 2B_{ep} \right)$$
$L_s \geq 1.5\left(\dfrac{H_b}{\sin\theta} + \dfrac{H}{\tan\theta}\right)$ for Cross-connections	ϕ = 1.00 (LRFD) Ω = 1.50 (ASD)
	Limit State 3: Local Yielding of the Branch/Branches due to Uneven Load Distribution
	$$P_n = F_{yb} t_b \left(2H_b + 2B_e - 4t_b \right)$$
	ϕ = 0.95 (LRFD) Ω = 1.58 (ASD)
	Limit State 4: Local Yielding of the Chord Sidewalls + Stiffeners
	$$P_n \sin\theta = 2F_y \left(t+t_s \right)\left(7.5t + \frac{H_b}{\sin\theta} \right)$$
	ϕ = 1.00 (LRFD) Ω = 1.50 (ASD)
	Limit State 5: Local Crippling of the Chord Sidewalls + Stiffeners
	$$P_n \sin\theta = 1.6t^2\left(1+\frac{3H_b/\sin\theta}{H}\right)\sqrt{EF_y}\,Q_f \quad +$$ $$1.6t_s^2\left(1+\frac{3H_b/\sin\theta}{H}\right)\sqrt{EF_{ys}}$$
Case for checking the Limit State of Shear of Chord Side Walls	ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)
	Limit State 6: Buckling of the Chord Sidewalls + Stiffeners
	$$P_n \sin\theta = \left(\frac{48t^3}{H-3t}\right)\sqrt{EF_y}\,Q_f \quad +$$ $$\left(\frac{48t_s^3}{H-3t}\right)\sqrt{EF_{ys}}$$
	ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)
	Limit State 7: Shear of the Chord Sidewalls + Stiffeners, in Gap Region
	$$P_n \sin\theta = 0.6F_y\left[2t\left(H-3t\right) \right] \quad +$$ $$0.6F_{ys}\left[2t_s\left(H-3t\right) \right]$$
	ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)

Design recommendations from Eurocode 3 (CEN, 2019), adapted to AISC 360-16 for the possible limit states, are given in Tables 4, 5 and 6, for various types of reinforcement. As noted before, a list of Notation (definition of symbols) follows the tables. The grade of steel used for the reinforcement should not be lower than that of the chord member.

Table 4: Available strengths for welded, flange-plate-reinforced, rectangular HSS gapped K-connections
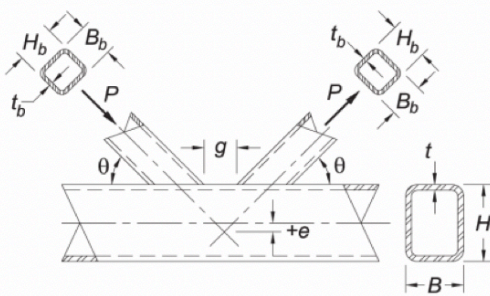
Reinforced with a Flange Plate	
Gapped K-connection	**Connection Available Axial Strength**
Note: Unreinforced connection is shown. For the reinforced connection, a plate stiffener with dimensions of B_s x L_s x t_s is added under branches. $$L_s \geq 1.5\left(\frac{H_{b1}}{\sin\theta_1} + g + \frac{H_{b2}}{\sin\theta_2}\right) \text{ and }$$ $$B_s \geq B - 2t$$	Limit State 1: Plastification of Stiffener $$P_n \sin\theta = F_{ys}t_s^2\left(9.8\beta_{effs}\left(\frac{B_s}{2t_s}\right)^{0.5}\right)$$
	ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)
	Limit State 2: Shear Yielding (Punching Shear) of Stiffener $$P_n \sin\theta = 0.6F_{ys}t_s\left(\frac{2H_b}{\sin\theta} + B_b + B_{eps}\right)$$
	ϕ = 0.95 (LRFD) Ω = 1.58 (ASD)
	Limit State 3: Local Yielding of the Branch/Branches due to Uneven Load Distribution $$P_n = F_{yb}t_b\left(2H_b + B_b + B_{es} - 4t_b\right)$$
	ϕ = 0.95 (LRFD) Ω = 1.58 (ASD)
	Limit State 4: Shear of the Chord Sidewalls in Gap Region $$P_n \sin\theta = 0.6F_y\left[2t(H - 3t)\right]$$
	ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)

Table 5: Available strength for welded, division-plate-reinforced rectangular HSS overlapped K-connections (see Figure 4(a))
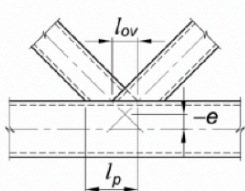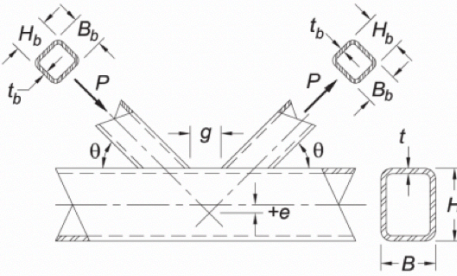
Reinforced with a Division Plate between the Branches	
Overlapped K-connection	**Connection Available Axial Strength**
Note: Stiffener thickness, t_s, to be at least twice the thickness of either branch. Overlap, O_v, is calculated, in the absence of a division plate, as follows: Overlap $= \frac{l_{ov}}{l_p} \times 100\%$	Limit State 1: Local Yielding of the Branch/Branches due to Uneven Load Distribution
	For $25\% \leq O_v < 50\%$, each branch to be checked using: $$P_n = F_{yb}t_b\left(\frac{O_v}{50}\left(2H_b - 4t_b\right) + B_e + B_{es}\right)$$ For $50\% \leq O_v < 80\%$, each branch to be checked using: $$P_n = F_{yb}t_b\left(2H_b - 4t_b + B_e + B_{es}\right)$$ For $80\% \leq O_v < 100\%$, each branch to be checked using: $$P_n = F_{yb}t_b\left(2H_b - 4t_b + B_b + B_{es}\right)$$
	ϕ = 0.95 (LRFD) Ω = 1.58 (ASD)

Reinforced with 2 Sidewall Plates

Gapped K-connection	Connection Available Axial Strength
Note: Unreinforced connection is shown. For the reinforced connection, 2 plate stiffeners with dimensions of H x L_s x t_s are welded to each sidewall of the chord. $L_s \geq 1.5\left(\dfrac{H_{b1}}{\sin\theta_1} + g + \dfrac{H_{b2}}{\sin\theta_2}\right)$ 	**Limit State 1: Plastification of Chord** $P_n\sin\theta = F_y t^2\left[9.8\beta_{eff}\left(\dfrac{B}{2t}\right)^{0.5}\right]Q_f$ $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
	Limit State 2: Shear Yielding (Punching Shear) of Chord $P_n\sin\theta = 0.6F_y t\left(\dfrac{2H_b}{\sin\theta} + B_b + B_{ep}\right)$ $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State 3: Local Yielding of the Branch/Branches due to Uneven Load Distribution $P_n = F_{yb}t_b\left(2H_b + B_b + B_e - 4t_b\right)$ $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State 4: Shear of the Chord Sidewalls + Stiffeners, in Gap Region $P_n\sin\theta = 0.6F_y\left[2t\left(H-3t\right)\right]$ $+$ $0.6F_{ys}\left[2t_s\left(H-3t\right)\right]$ $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

Notation

B = overall width of rectangular HSS chord, 90° to the plane of the connection

B_b = overall width of rectangular HSS branch, 90° to the plane of the connection

B_e = effective width of HSS branch, 90° to the plane of the connection $=\left(\dfrac{10t}{B}\right)\left(\dfrac{F_y t}{F_{yb}t_b}\right)B_b \leq B_b$

B_{ep} = effective width for punching shear of chord, 90° to the plane of the connection $=\left(\dfrac{10t}{B}\right)B_b \leq B_b$

B_{eps} = effective width for punching shear of plate, 90° to the plane of the connection $=\left(\dfrac{10t_s}{B_s}\right)B_b \leq B_b$

B_{es} = effective width of HSS branch, 90° to the plane of the connection $=\left(\dfrac{10t_s}{B_s}\right)\left(\dfrac{F_{ys}t_s}{F_{yb}t_b}\right)B_b \leq B_b$

B_s = width of stiffening plate on HSS chord connecting face, 90° to the plane of the connection

d_s = depth of annular stiffener (outside radius minus the inside radius)

D = outside diameter of round HSS chord member

E = modulus of elasticity of steel = 29,000 ksi

F_y = specified minimum yield stress of HSS chord member

F_{yb} = specified minimum yield stress of HSS branch member

F_{ys} = specified minimum yield stress of stiffening plate

H_b = overall height of rectangular HSS chord member, in the plane of the connection

L_s = length of stiffening plate, in the plane of the connection

Q_f = chord member stress function

   = 1.0 for chord connecting face in tension

   $= 1.3 - 0.4(U/\beta) \leq 1.0$ for chord connecting face in compression, for T-, Y- and Cross-connections

   $= 1.3 - 0.4(U/\beta_{eff}) \leq 1.0$ for chord connecting face in compression, for gapped K-connections

t = design wall thickness of HSS chord member

t_b = design wall thickness of HSS branch member

t_s = thickness of stiffening plate

U = highest normal stress in the chord connecting face next to connection, due to axial load + bending

β = width ratio between branch and chord = B_b/B; diameter ratio between branch and chord = D_b/D

$\beta_{eff} = \left[\left(B_b + H_b\right)_{compressionbranch} + \left(B_b + H_b\right)_{tensionbranch}\right]/4B$

$\beta_{effs} = \left[\left(B_b + H_b\right)_{compressionbranch} + \left(B_b + H_b\right)_{tensionbranch}\right]/4B_s$

θ = acute angle between the branch and chord

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