

$$U_1 = 1.0 \quad (\text{S16.1, Clause 13.8.1(a)})$$

$$M_{r0} = 31.9 \text{ kN}\cdot\text{m} \quad \text{and} \quad M_{r0} = 86.6 \text{ kN}\cdot\text{m}$$

$$\therefore \text{combination} = (25.5/1.64) + (1.0/31.9)/86.6$$

$$= 0.016 + 0.368 = 0.38 \leq 1.0 \quad \therefore \text{OK}$$

Therefore, bottom chord will also be adequate.

Use S16.1, Clause 13.8.1 to check vertical web members as beam-columns.

Check overall member strength of first vertical from end

$$C_{r0} = 0.5P = 0.5(17) = 8.5 \text{ kN}$$

$$C_{r0} = 984 \text{ kN} \quad (L = 2500)$$

$$U_1 = \frac{\phi_1}{1 - \frac{C_{r0}}{C_e}}$$

$$C_e = \frac{\pi^2 EI}{L^2}$$

$$= 3.14^2 (200) 12.6 (10^6) / 2500^2 = 3980 \text{ kN}$$

$$\therefore U_1 = 0.4 / (1 - (8.5/3980)) = 0.401$$

$$M_{r0} = 3P = 3(17) = 51.0 \text{ kN}\cdot\text{m}$$

$$M_{r0} = \phi Z_0 F_{y0} = 61.4 \text{ kN}\cdot\text{m}, \text{ as earlier}$$

$$\text{Therefore, combination} \quad \frac{C_{r0}}{C_{r0}} + \frac{U_1 M_{r0}}{M_{r0}}$$

$$= (8.5/984) + (0.401(51.0)/61.4)$$

$$= 0.009 + 0.333 = 0.34 \leq 1.0 \quad \therefore \text{OK}$$

Check cross section strength of first vertical from end

$$C_{r0} = 8.5 \text{ kN}$$

$$C_{r0} = \phi A_0 F_{y0} = 0.9(3610) 0.350 = 1140 \text{ kN}$$

$$U_1 = 1.0$$

$$M_{r0} = 51.0 \text{ kN}\cdot\text{m} \quad \text{and} \quad M_{r0} = 61.4 \text{ kN}\cdot\text{m}$$

$$\therefore \text{combination} = (8.5/1140) + (1.0(51.0)/61.4)$$

$$= 0.007 + 0.831 = 0.84 \leq 1.0 \quad \therefore \text{OK}$$

Hence, the members would also be suitable by elastic design procedures. By either design method, the chord thickness is still enhanced to provide adequate connection strength. The end connections (at A, B, M and N) can be made by welding the vertical posts to the chords to form T connections, and then adding capping plates to the ends of the chord sections.

6.2 Knee Connections

Research on mitred square and rectangular HSS knee connections (such as those in Fig. 6.10) has been performed by Mang *et al.* (1980) at the University of Karlsruhe and subsequently incorporated into the German standard DIN 18 808 (1984). Their recommendations have also been reported by Wardenier (1982), CIDECT (1984), and Dutta and Wücker (1988). They cover both stiffened and unstiffened knee connections, and are intended for use in corner connections of rigid frames.

The original test results and moment vs. rotation diagrams are not widely available, but DIN 18 808 makes its design recommendations applicable for "flexurally rigid frame corners". However, it could be expected that the rotation capacity of some *unstiffened* connections might be low, and in structures in which reasonable rotational capacity is required, a stiffened knee connection should be used (Wardenier 1982). In the Karlsruhe tests, simple unstiffened knee connections tended to fail by excessive deformation of the lateral HSS cross-wall in compression. On the other hand, for connections with a stiffening plate, excessive deformations appeared only for very thin walled members.

For thicker hollow sections, complete plastification was reached in the course of the tests (CIDECT 1984). Table 6.1 (adapted from DIN 18 808) gives the limiting HSS member dimensions which are permitted. In view of the uncertain moment vs. rotation properties, it would seem prudent to use, for *unstiffened* connections, only compact HSS members which satisfy plastic design requirements for rigid frames (i.e., Class 1 sections).

Analysis of the test results showed that, for design purposes, it was possible to estimate the total flexural and axial load capacity of the connection by applying a reduction factor to the material yield stress. Thus, adequate connection strength will be obtained for both stiffened and unstiffened 90° mitre connections providing [6.13] and [6.14] are satisfied (DIN 18 808 1984; Eurocode Committee for Standardization (1992a)).

$$\frac{N_i}{N_{ti}} + \frac{M_i}{M_{ti}} \leq \alpha \quad \text{for } i = 1 \text{ and } 2 \quad (\text{see Fig. 6.10}) \quad [6.13]$$

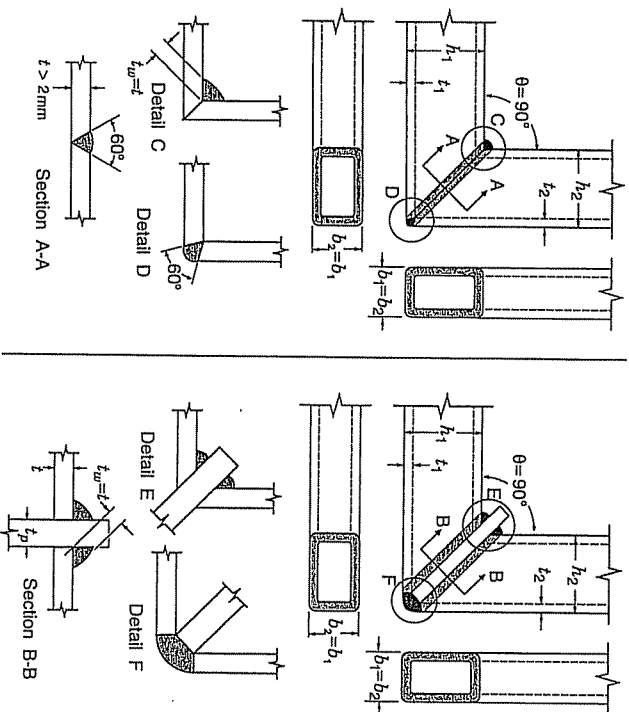


FIGURE 6.10
Details of square and rectangular HSS knee connections
(from DIN 18 808, 1984)

- $b_1, h_1 \leq 400$ mm, if stiffening plate is used (Fig. 6.10(b))
 $b_1, h_1 \leq 300$ mm, if stiffening plate is not used (Fig. 6.10(a))
 $0.33 \leq h_2/b_2 \leq 3.5$
 $2.5 \text{ mm} \leq t_2 \leq 25$ mm, for Grade 350 HSS
 $b_2/t_2, h_2/t_2 \leq 36$, for Grade 350 HSS, if stiffening plate is used
 $b_2/t_2, h_2/t_2$ to meet Class 1 requirements if stiffening plate is not used

TABLE 6.1
Ranges of validity for HSS member dimensions in 90° mitred knee connections
(adapted from DIN 18 808, 1984)

N_{Ti} is the axial yield resistance $\phi A_i F_{yi}$ of member i , and M_{Ti} is the factored moment resistance of member i . The term α is a stress reduction factor which can be taken as 1.0 for mitre connections *with stiffening plates*. For mitre connections *without stiffening plates*, α is a function of the cross-sectional dimensions and is given in Figs. 6.11 and 6.12. Also for connections *without stiffening plates*, N_i should not exceed $0.2N_{Ti}$ (European Committee for Standardization (1992)). The shear force acting at the connection V_i should also meet the requirement (European Committee for Standardization (1992)):

$$\frac{V_i}{V_p} \leq 0.5 \quad [6.14]$$

where V_p is the shear yield resistance in the member under consideration. This can be taken as $F_y/\sqrt{3}$ multiplied by the cross-sectional area of the HSS webs ($2 h_i t_i$). If [6.14] is not satisfied, the connection strength could still be deemed adequate, providing the combined stress does not produce failure according to the Von Mises failure criterion; in doing this check, the normal stresses (axial and bending) should be increased by a factor of $1/\alpha$.

For *stiffened* knee connections, the plate size should comply with the European Committee for Standardization (1992):

$$t_p \geq 1.5 t_i \quad (i = 1 \text{ or } 2), \text{ and } t_p \geq 10 \text{ mm.} \quad [6.15]$$

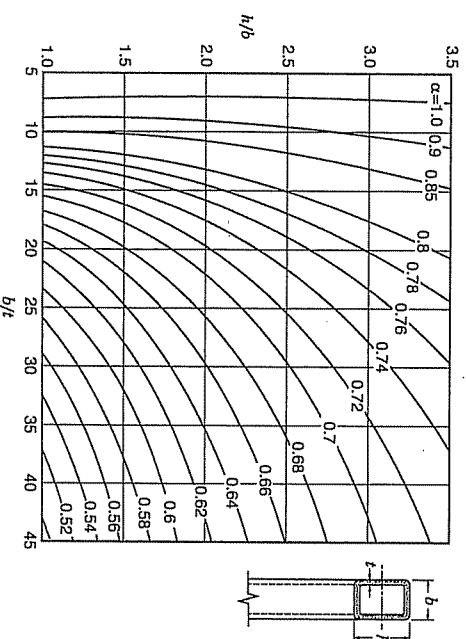


FIGURE 6.11
Stress reduction factors α for rectangular HSS subjected to bending about the Major Axis in 90° unstiffened mitred knee connections (DIN 18 808 1984)

The fabrication details shown in Fig. 6.10 should be followed, with t_w being the weld throat thickness. According to DIN 18 808, the welds can be considered a "pre-approved" size when the throat thickness is equal to the connected wall thickness, plus the factor α (for unstiffened knee connections) is ≤ 0.71 for Grade 350 HSS. (According to CAN/CSA-S16.1-94, the pre-approved fillet weld throat size would be 1.10 times the HSS wall thickness, for Grade 350 HSS, as discussed in Section 6.1.2.)

Since obtuse-angle (i.e., $\theta > 90^\circ$ in Fig. 6.10) knee connections behave more favourably than right-angle connections, the same design checks can be undertaken for them as for right-angle knees (CIDECT 1984). For *unstiffened* knee connections with $90^\circ < \theta < 180^\circ$ this strength enhancement can be used to advantage in [6.13] by increasing the value of α as follows:

$$\alpha = 1 - \left(\sqrt{2} \cos \frac{\theta}{2} \right) (1 - \alpha_{\theta=90}) \quad [6.16]$$

$\alpha_{\theta=90}$ is the value obtained from Figs. 6.11 and 6.12.

An alternative form of connection reinforcement (other than a transverse stiffening plate) is a haunch on the inside of the knee. This haunch piece needs to be the same width as the two main members, and can easily be provided by taking a cutting from one of the HSS sections. Provided the haunch length is sufficient to ensure that the bending moment does not exceed the section yield moment ($S_x F_y$) in either main member, the connection resistance will be adequate and does not require checking (CIDECT 1984).

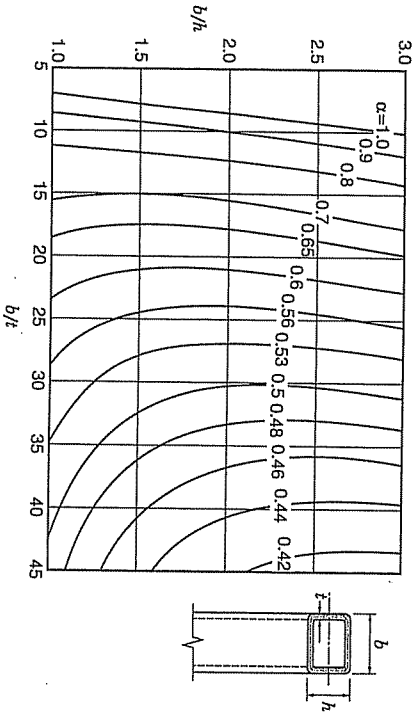


FIGURE 6.12

Stress reduction factors α for rectangular HSS subjected to bending about the Minor Axis in 90° unstiffened mitred knee connections (DIN 18 808 1984)

6.3 In-Plane and Out-of-Plane Moments for T and X Connections

Bending moments in the plane of the structure, and out of the plane of the structure are considered in the following sections.

6.3.1 In-Plane Bending Moments for Rectangular HSS

T Connections

The design criteria for square and rectangular HSS *T connections* with the branch member subjected to an in-plane bending moment M_{T1} are described in Section 6.1.2 under the topic of Vierendeel connections. To summarize, the moment resistances can be calculated as follows:

- (a) For $\beta \leq 0.85$, design is governed by chord face yielding, with the connection moment of resistance given by:

$$M_{T1}^* = F_{y0} t_0^2 h_1 \left(\frac{1}{2h_1/b_0} + \frac{2}{\sqrt{1-\beta}} + \frac{h_1/b_0}{(1-\beta)} \right) f_2(r) \quad (6.21)$$

where $f_2(r)$ is given by [6.1a].

- (b) For $0.85 < \beta \leq 1.0$, design is governed by the more critical failure mode between 1) the reduced branch member capacity (an "effective width" failure mode) and 2) the chord side wall bearing or buckling capacity.

For "effective width" failure:

$$M_{T1}^* = F_{y1} \left[Z_1 - \left(1 - \frac{b_e}{b_1} \right) b_1 t_1 (h_1 - t_1) \right] \quad (6.31)$$

In [6.3], Z_1 is the plastic section modulus about the correct axis of bending. The term b_e is defined by [6.4].

For chord side wall failure:

$$M_{T1}^* = 0.5 F_k t_0 (h_1 + 5t_0)^2 \quad (6.5)$$

where $F_k = F_{y0}$

X Connections

The design criteria for square and rectangular HSS *X connections* subjected to equal and opposite (self-equilibrating) in-plane bending mo-