CONNECTIONS

- Connections are the devices used to join elements of a structure together at a point such that forces can be transferred between them safely.
- Connection design is more critical than the design of members.
- The failure of connection usually means collapse of a greater part or whole of the structure.
- In general, relatively more factor of safety is provided in the design of connections.

- The connections shall be designed for a minimum strength of 44 kN, except for lacing, sag rods, or girts.
- The rigid connection should provide sufficient strength and ductility.
- The ductility is very useful for redistribution of stresses and dissipation of extra energy in case of earthquakes, etc.

TYPES OF CONNECTIONS

Based On Means Of Connection

A. Welded connections
B. Riveted connections
C. Bolted connections

Based On Forces To Be Transferred

A. Truss connections

B. Simple / shear connections

C. Moment connections

  – i) Fully restrained (FR) connections

  – ii) Partially restrained (Semi-rigid) connections

D. Splices

E. Brackets

F. Bearing joints

BUILDING / FRAME / BEAM CONNECTIONS

- **Fully Restrained / FR / Moment Connections**
- **Partially Restrained / PR Connections**
  a. **Simple or shear or flexible connections**
  b. **Semi-rigid connections**
  c. **Bearing joints of compression members**

Moment Connections

- Moment connections are also referred to as rigid, continuous frame or FR connections.
- Knee joints are the typical example.
- They are assumed to be sufficiently rigid keeping the original angles between members practically unchanged after application of loads.
- Greater than 90 percent moment may be transferred with respect to ideally rigid connection besides the full transfer of shear and other forces.

- These connections are particularly useful when continuity between the members of the building frame is required to provide more flexural resistance and to reduce lateral deflection due to wind loads.
- Both the flanges and web of the member are to be connected for this type of connection.

Partially Restrained Connections

- Type PR connections have rigidity less than 90 percent compared with ideally rigid connections.
- Although the relative rotation between the joining members is not freely allowed, the original angles between members may change within certain limits.
- They transfer some percentage of moment less than 90 percent and full shear between the members.
- PR connections may be further classified into simple and semi-rigid connections.

Shear Connections

- Simple or shear connections have less than 20 percent rigidity.
- They are considerably flexible and the beams become simply supported due to the possibility of the large available rotation.
- Moment may not be transferred in larger magnitudes with the requirement that the shear force is fully transferred.

- In these connections, primarily the web is to be connected because most of the shear stresses are concentrated in it.
- Connections of beams, girders, or trusses shall be designed as flexible joints to resist only the reaction shears except otherwise required.
- Flexible beam connections shall accommodate end rotations of unrestrained beams.

Semi-Rigid Connections

- Semi-rigid connections provide rigidity in-between fully restrained and simple connections.
- Approximately 20 to 90 percent moment compared with ideal rigid joint may be transferred.
- End moments may develop in the beams and the maximum beam moment may be significantly reduced.
- Usually no advantage is taken of this reduction and beams are designed as simply supported because of various reasons.

- One of the reasons is the difficulty of structural frame analysis for varying degrees of restraints at the joints and unpredicted rotations.
- Further, LRFD Specification states that a connection can only be considered as semi-rigid if proper evidence is presented to prove that it is capable of providing a certain end restraint.
- These are the commonly used types of connections in practice because their performance is exceptionally well under cyclic loads and earthquake loadings.

Bearing Joints

- There shall be sufficient connectors to hold all parts of the section securely in place when columns rest on bearing plates.
- All compression joints shall be designed to provide resistance against uplift and tension developed during the uplift load combination.

MOMENT CONNECTIONS

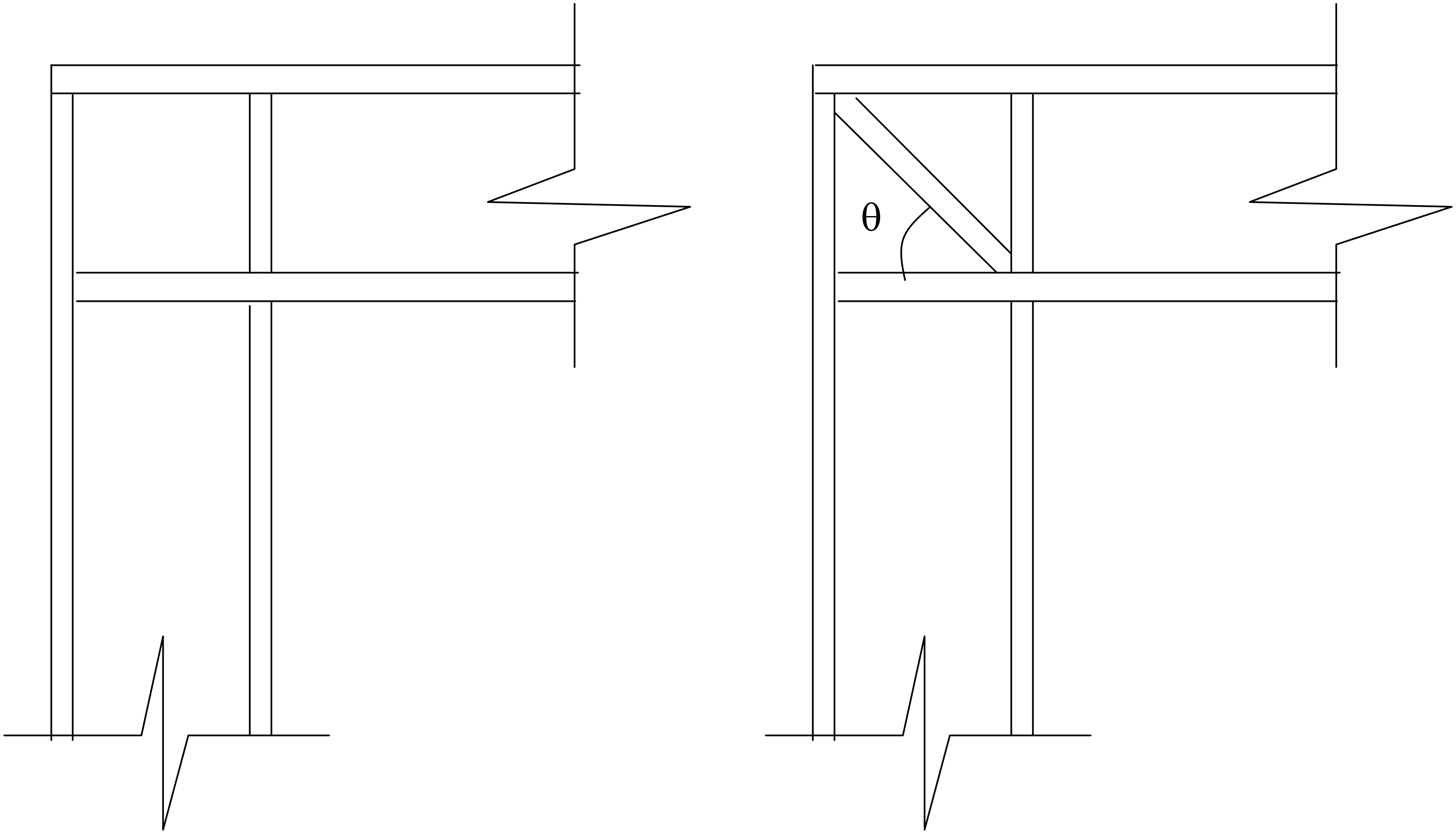
Rigid Frame Knees

- These are a type of fully restrained (FR) or moment connection.
- In the design of rigid frames the safe transmission of load at the junction of beam and column is of great importance.
- When members join with their webs lying in the plane of the frame, the junction is frequently referred to as a ***knee joint***.

Typical knee joints are:

Square Knees, with and without a diagonal stiffener, are shown in Figure 8.31.

Column or beam section may be continued through the junction.



a) Square Knee Without Stiffener

b) Square Knee With Stiffener

Figure 8.31. Square Knee Joints.

Square Knee With a Bracket is shown in Figure 8.32.

This type of joint may resist large negative moments reducing the size of the beam and the column.

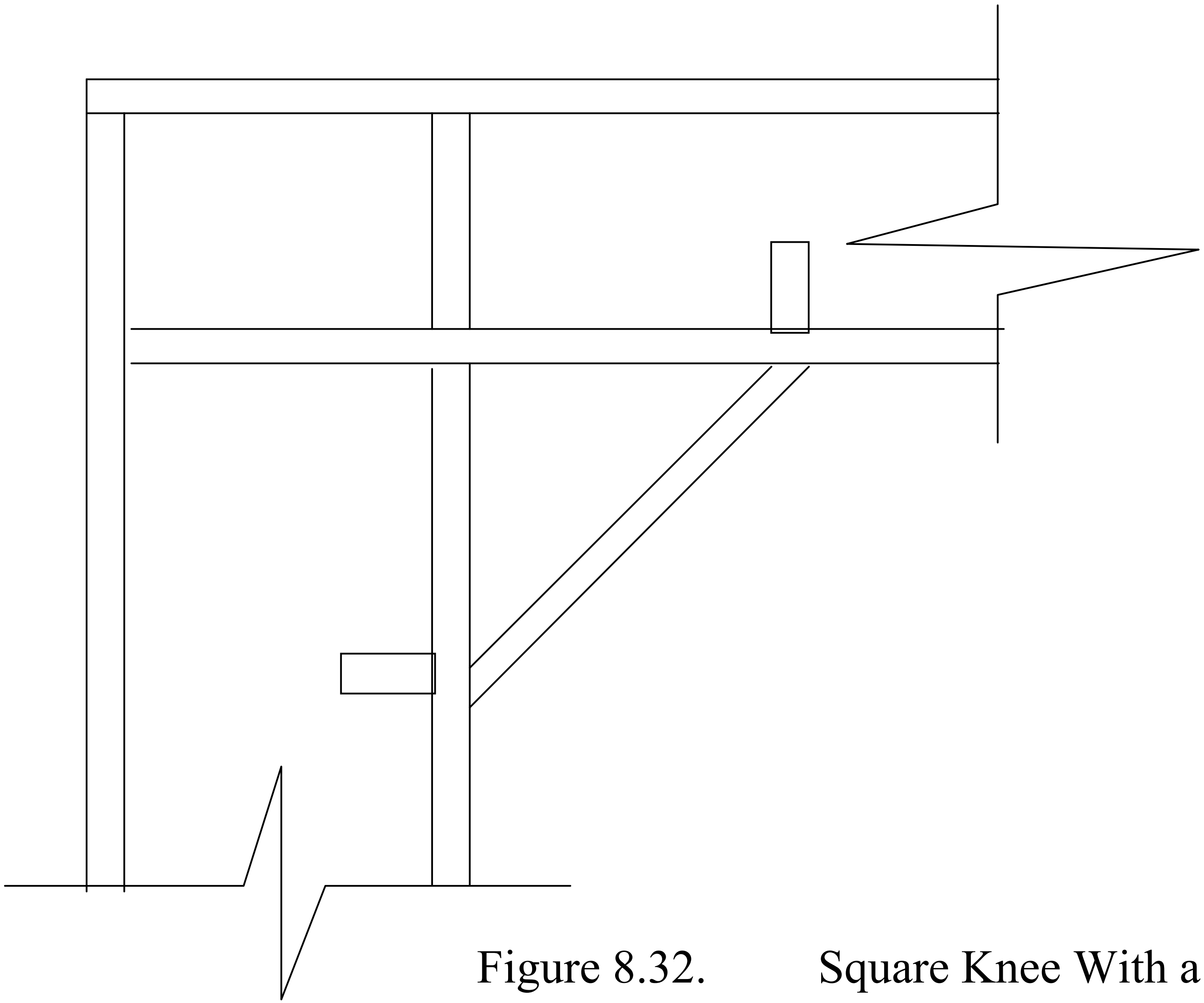


Figure 8.32. Square Knee With a Bracket.

Straight Haunched Knee is a modification of the square knee with a bracket.

The beam and column sections are discontinued short of the connection.

The haunch consists of a separate plate reinforced by perpendicular stiffeners (see Figure 8.33).

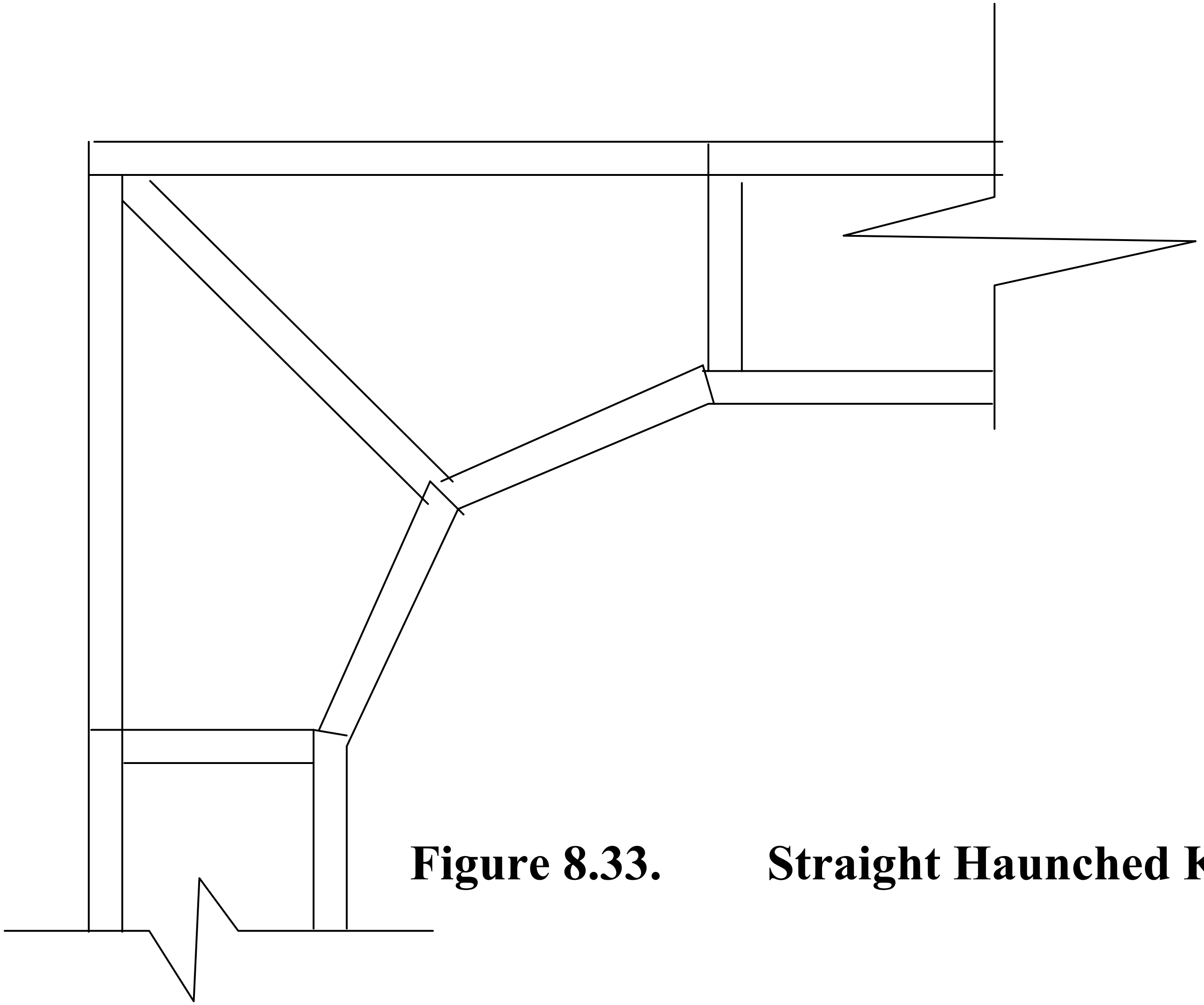


Figure 8.33. Straight Haunched Knee.

Curved Haunched Knee is similar to a straight haunched knee with the difference of having a curved inner profile, as shown in Figure 8.34.

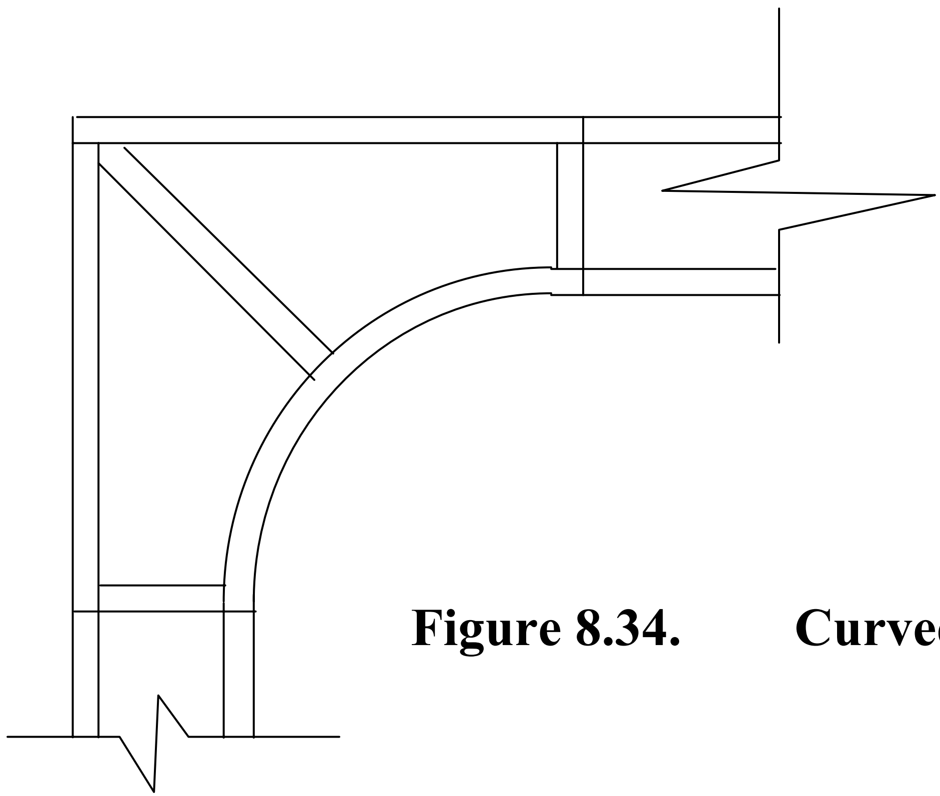


Figure 8.34. Curved Haunched Knee.

For haunched knees, variable moment of inertia has to be considered with the knees for both beams and column to perform analysis.

To be adequately designed, a knee connection must satisfy the following requirements:

1. The end moment between the beam and the column must be transferred.
2. The beam end shear must safely go to the column.
3. The shear at the top of the column should be transferred into the beam.
4. The joint must deform in a manner consistent with the analysis by which moments and shears are determined.
5. If a plastic hinge associated with the failure mechanism is expected to form at or near the knee, adequate rotation capacity must be built into the connections.

- Square knees have the greatest plastic rotation capacity but this flexibility increases the service load deflections as they deform elastically the most under the loads.
- Curved knees are the most stiff but have the least rotation capacity.
- Since straight tapered knees provide reasonable stiffness along with adequate rotation capacity, in addition to the fact that they are cheaper than curved haunches to fabricate, the straight haunched knees are more commonly used.

Shear Transfer In Square Knees

- In the design of a rigid frame having square knees, two rolled sections may come together at right angles.
- The moments, shears and axial forces (M, V and H) acting on the boundaries of the square knee region, as shown in Figure 8.35(a), may be determined by either elastic or plastic analysis.
- The forces carried by the flanges must be transmitted by shear into the web, as shown in (b) part of the same figure.

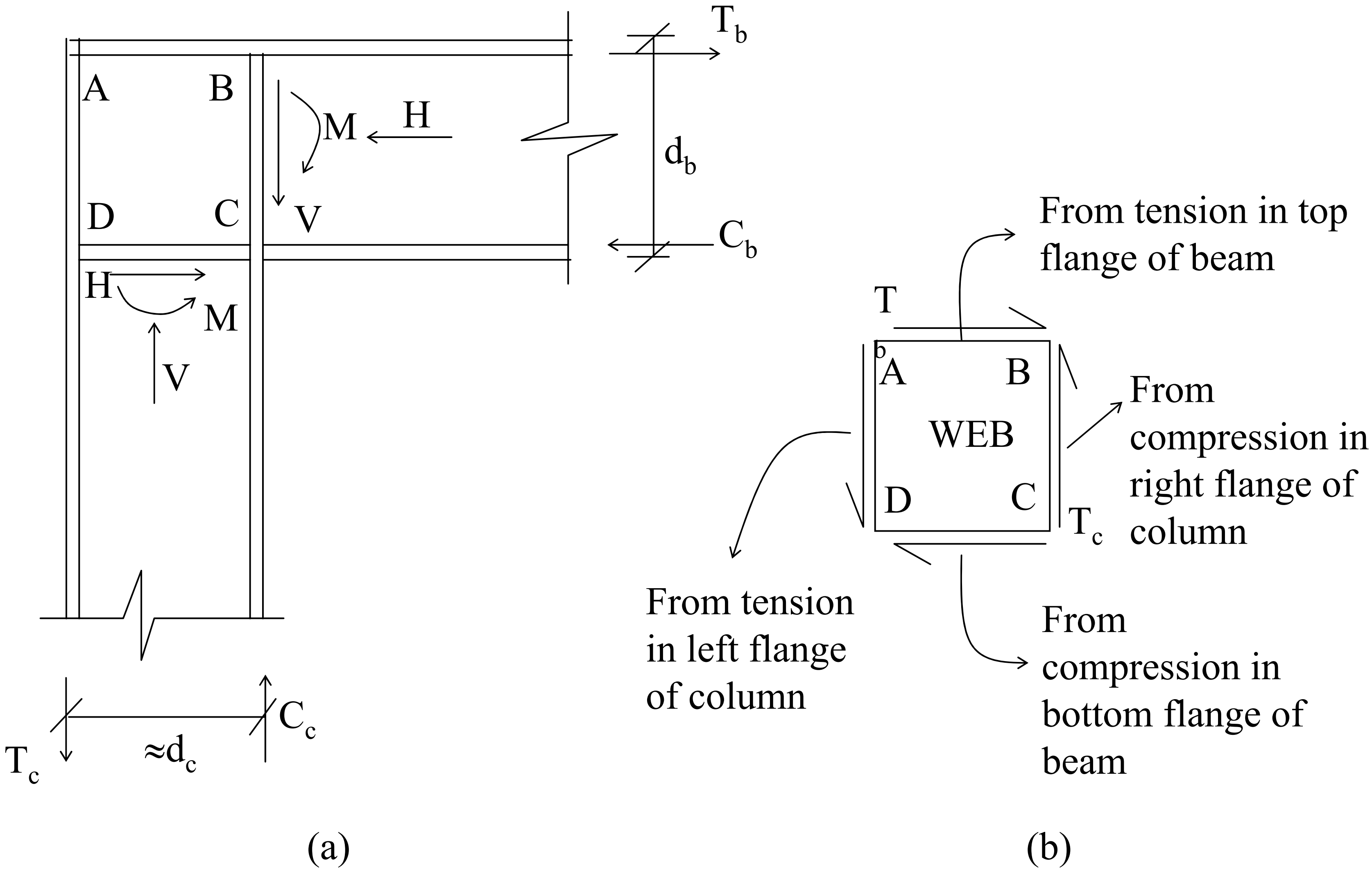


Figure 8.35. Forces Acting on Web of a Square Knee.

Assuming all bending moment to be carried by the flanges, and approximating the distance between flange centroids as 0.95 d_b, the flange force is:

$$T_u = T_b = \frac{M_u}{0.95\, d_b}$$

The nominal shear strength of the web across the edge AB is:

$$V_n = V_{ab} = \tau_y\, t_w\, d_c$$

where $\tau_y = 0.6\, F_y$ and $\phi_v = 0.9$

For design, $\phi V_n = T_u$,

which gives:

$$\phi_v (0.6\, F_y)\, t_w\, d_c = \frac{M_u}{0.95\, d_b}$$

Required t_w without diagonal stiffener $= \dfrac{1.95\, M_u}{F_y\, d_b\, d_c}$

$$= \frac{1.95\, M_u}{F_y\, A_{bc}}$$

where A_{bc} = the planner area within the knee

$= d_b\, d_c$.

Diagonal Stiffeners

- In a rigid frame knee, the required web thickness usually exceeds that provided by a W-section and reinforcement is required.
- A doubler plate is sometimes used to thicken the web region, which is not a general practical solution because of the difficulty of making the attachment to the column web.
- Usually, a pair of diagonal stiffeners is the best solution, as shown in Figure 8.37.

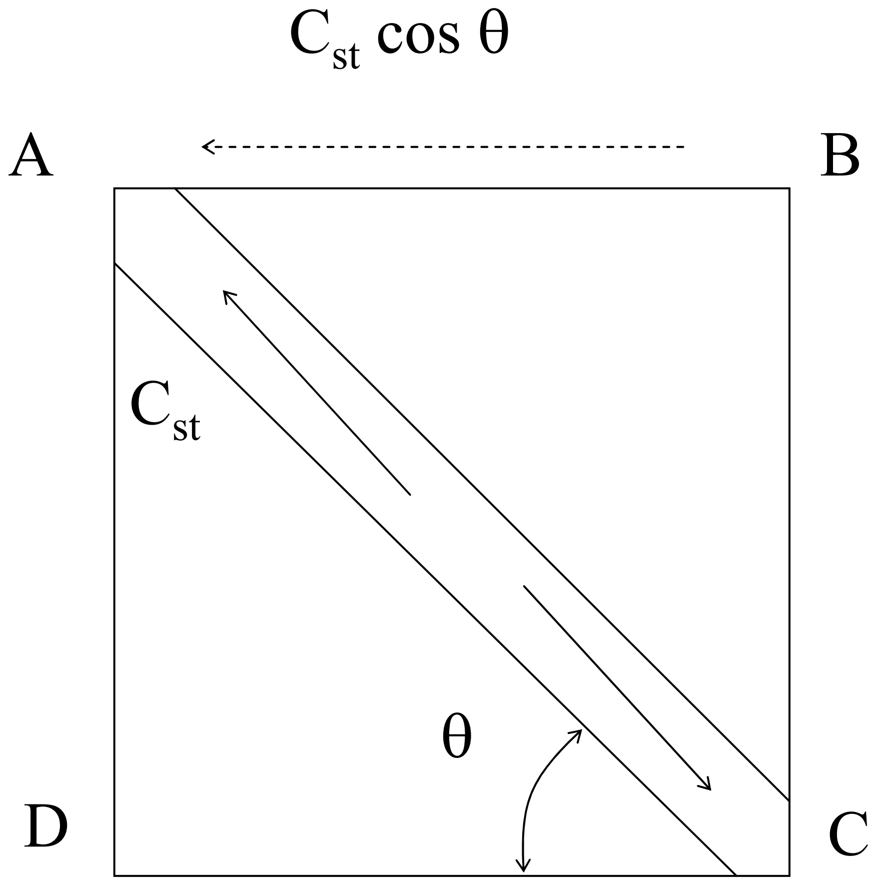


Figure 8.37. Web of a Square Knee Connection With a Stiffener.

- Stiffener resistive compressive force

$$= C_{st}$$

$$= A_{st}\ \phi_c\ F_{cr}$$

- Applied shear on the web (Figure 8.35 (b))

$$= T_u$$

- When diagonal stiffeners are used, the horizontal component $C_{st} \times \cos\theta$ of the stiffener force participates in resisting the shear with the web.

$$\Sigma F_x = 0 \Rightarrow$$

$$T_u = V_{ab} + C_{st}\, cos\ \theta$$

$$\frac{M_u}{0.95\, d_b} = \phi_v\, (_{0.60\, Fy})\ t_w\, d_c + A_{st}\, \phi_c F_{cr}\, cos\ \theta$$

$$A_{st,\, req} = \frac{1}{\phi_c F_{cr} \cos\theta}\left[\frac{M_u}{0.95\, d_b} - \phi_v \left(0.60\, F_y\right) t_w\, d_c\right]$$

where ϕ_v = 0.90 for any yield limit state like in shear

ϕ_c = 0.85 for compression elements

F_{cr} = compression limit state stress

Example 8.6

Design the square knee connection given in Figure 8.38 to join a W690 × 140 girder to a W360 × 110 column. The factored moment M_u to be carried through the joint is 510 kN-m. Use A36 steel and E70 electrodes with SMAW.

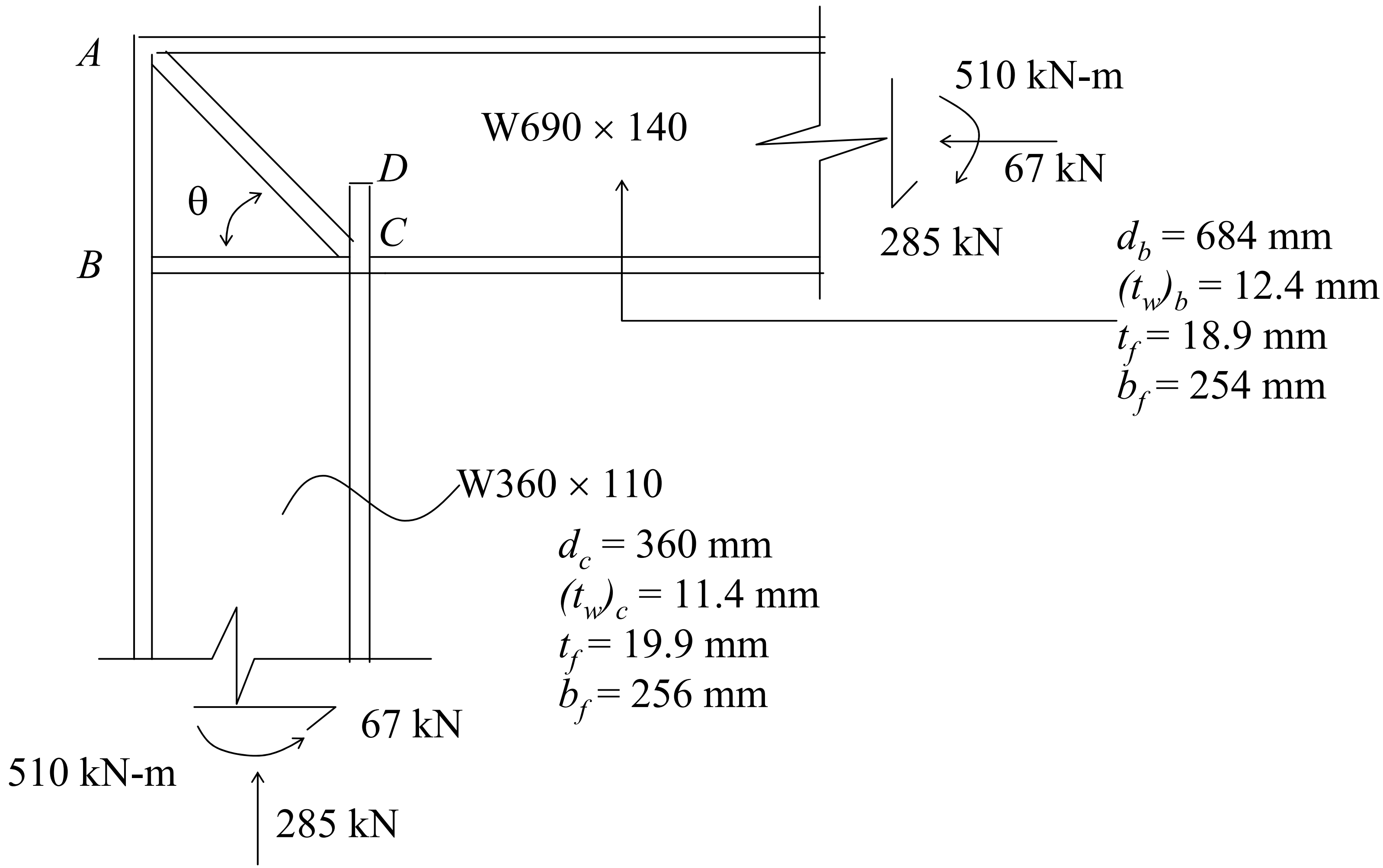


Figure 8.38. Square Knee of Example 8.6.

Solution

Check the web without diagonal stiffener:

$$\text{Required } t_w = \frac{1.95\,M_u}{F_y\,A_{bc}}$$

$$= \frac{1.95\times510\times10^6}{250\times684\times360}$$

$$= 16.15 \text{ mm}$$

Actual t_w = 12.4 mm for W690 × 140

< 16.15 mm

∴ **A diagonal stiffener is required.**

Stiffener size:

$$\tan\theta = \frac{d_b}{d_c} = \frac{684}{360}$$

$$\Rightarrow \quad \theta = 62.24^\circ \;\; and \;\; \cos\theta = 0.466$$

Assuming $F_{cr} \cong 0.95F_y$,

$$(A_{st})_{req} = \frac{1}{0.85\times238\times0.466}\left[\frac{510\times10^6}{0.95\times684} - 0.9\times0.60\times250\times12.4\times360\right]$$

$$= 1933 \text{ mm}^2$$

(half area on one side = 967 mm^2)

Using $t_{st} = 12$ mm ; $b_{st} = 81$ mm
say 85 mm

$\lambda = \frac{85}{12} = 7.08 < \lambda_r = 15.8$ ***OK***

Size of stiffener: 2 PL$_s$ 12 × 85 on both sides of the web

Strength of the stiffener acting as a column:

Overall width of stiffener = b $= 2\, b_{st} + t_w$

$= 2 \times 85 + 12.4 = 182.4$ mm

$$r = \sqrt{\frac{t_{st}(b)^3/12}{t_{st}\, b}} = \sqrt{\frac{1}{12}}(b) = 0.289\, b$$

$$= 52.65 \text{ mm}$$

$$\frac{KL}{r} = \frac{d_c/\cos\theta}{r} = \frac{360/0.466}{52.65} \cong 15$$

$$\therefore\ \boldsymbol{\phi_c F_{cr} \approx 201.88\ MPa}$$

This means that F_{cr} is equal to the assumed value of F_{cr}.

Determine the fillet weld size along length AB. The weld must transmit the factored flange force into the beam web. The maximum design flange force that can be developed is $\phi_t F_y A_f$.

Flange force $= \phi_t F_y A_f = 0.90 \times 250 \times 19.9 \times 256/1000 = 1146.24$ kN

The design strength of fillet welds along both sides of web is:

$$\phi R_{nw} = 2(0.75 \times 0.707 \times t_w \times 0.6 \times 495/1000)$$
$$= 0.315\, t_w \text{ kN/mm}$$

Available length for weld $= d_b - 2\ t_f$

$= 684 - 2 \times 18.9 = 646.2$ mm

$\therefore\ 0.315\ t_w\ (646.2) = 1146.24$

$t_w = 6$ mm

Use 6 mm thick E70 fillet weld along length AB (both sides of girder web)

Determine fillet weld size along length BC. The connection of the column web to the beam flange must carry the force resulting from flexure and axial load, combined with the shear acting simultaneously on the weld.

The forces transferred through this weld may conservatively be estimated as follows:

Tensile component $= \phi_t F_y t_w$

$= 0.9 \times 250 \times 12.4/1000 = 2.79$ kN/mm

Shear component $= \dfrac{V_u}{d_c - 2t_f} = \dfrac{67}{360 - 2 \times 19.9}$

$= 0.21$ kN/mm

Resultant loading $= \sqrt{2.79^2 + 0.21^2} = 2.80$ kN/mm

Required $t_w = \dfrac{2.80}{0.315} \cong 9$ mm

Use 9 mm thick E70 fillet weld along length BC on both sides of girder web

Weld required along diagonal stiffeners is designed next. This weld must develop the required stiffener strength.

$$\phi C_s = \phi F_y A_{st}$$

$$= 0.9 \times 250 \times 2 \times 12 \times 80/1000 = 432 \text{ kN}$$

$$\text{Required } t_w = \frac{432/(360/0.466)}{4 \times 0.75 \times 0.707 \times 0.6 \times 0.495} = 1 \text{ mm}$$

<u>say 6 mm</u>

Use 6mm thick E70 fillet weld along diagonal stiffener on both sides of girder web

Determine the required length of the stiffener *CD*:

The design strength based on local web yielding from the inside column flange at *C* is:

$$P_{bf} = \phi(5k + t_{fb}) F_{yc} t_{wc}$$
$$= 1.0(5 \times 37 + 18.9) \times 250 \times 11.4/1000 = 581.12 \text{ kN}$$

Flange force as calculated earlier $= 1146.24$ kN

The force is greater than capacity of the web alone and diagonal stiffener is already resisting other forces. Hence, vertical stiffener is required at *C*.

Stiffener along *CD*:

$$\text{Required } A_{st} = \frac{1146.24 - 581.12}{\phi F_y} \times \frac{1}{2}$$

$$= 1256 \text{ mm}^2 \text{ per plate}$$

$$\text{Width available} = \frac{b_{fb} - t_{wb}}{2} = \frac{254 - 12.4}{2}$$

$$= 120.8 \text{ mm} \quad \underline{\text{say } 110 \text{ mm}}$$

$$\text{Required } t_{st} = \frac{1256}{110} \cong 12 \text{ mm}$$

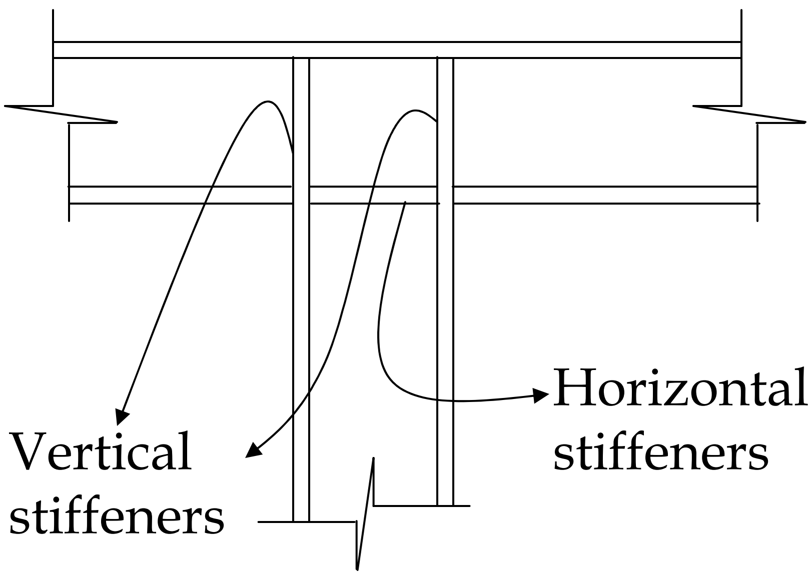
$$\lambda = \frac{110}{12} = 9.17 \quad < \quad \lambda_p = 10.8 \quad \textbf{OK}$$

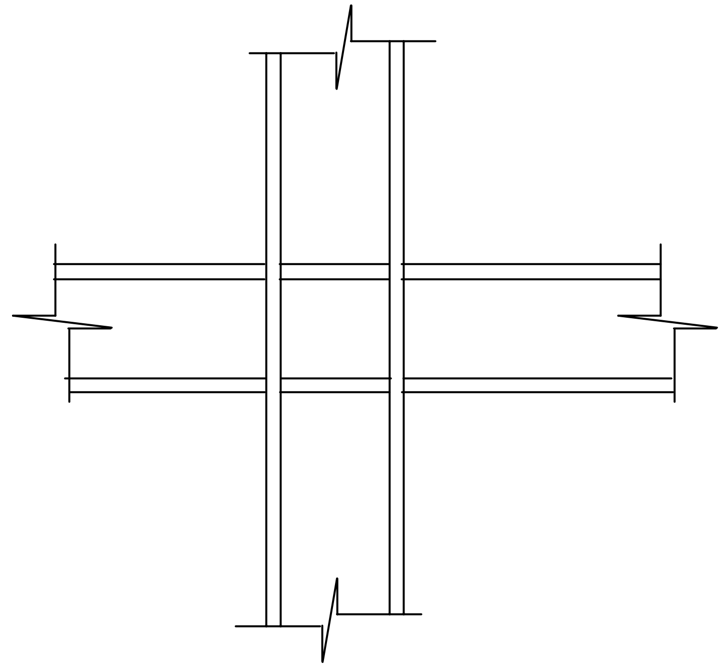
Length of stiffener $= \dfrac{d - 2t_f}{2} \cong 325$ mm

Use 2 PL_s – 12 × 110 × 325, tapered from full width at C to zero at D

Steel Structures

Central Moment Connection

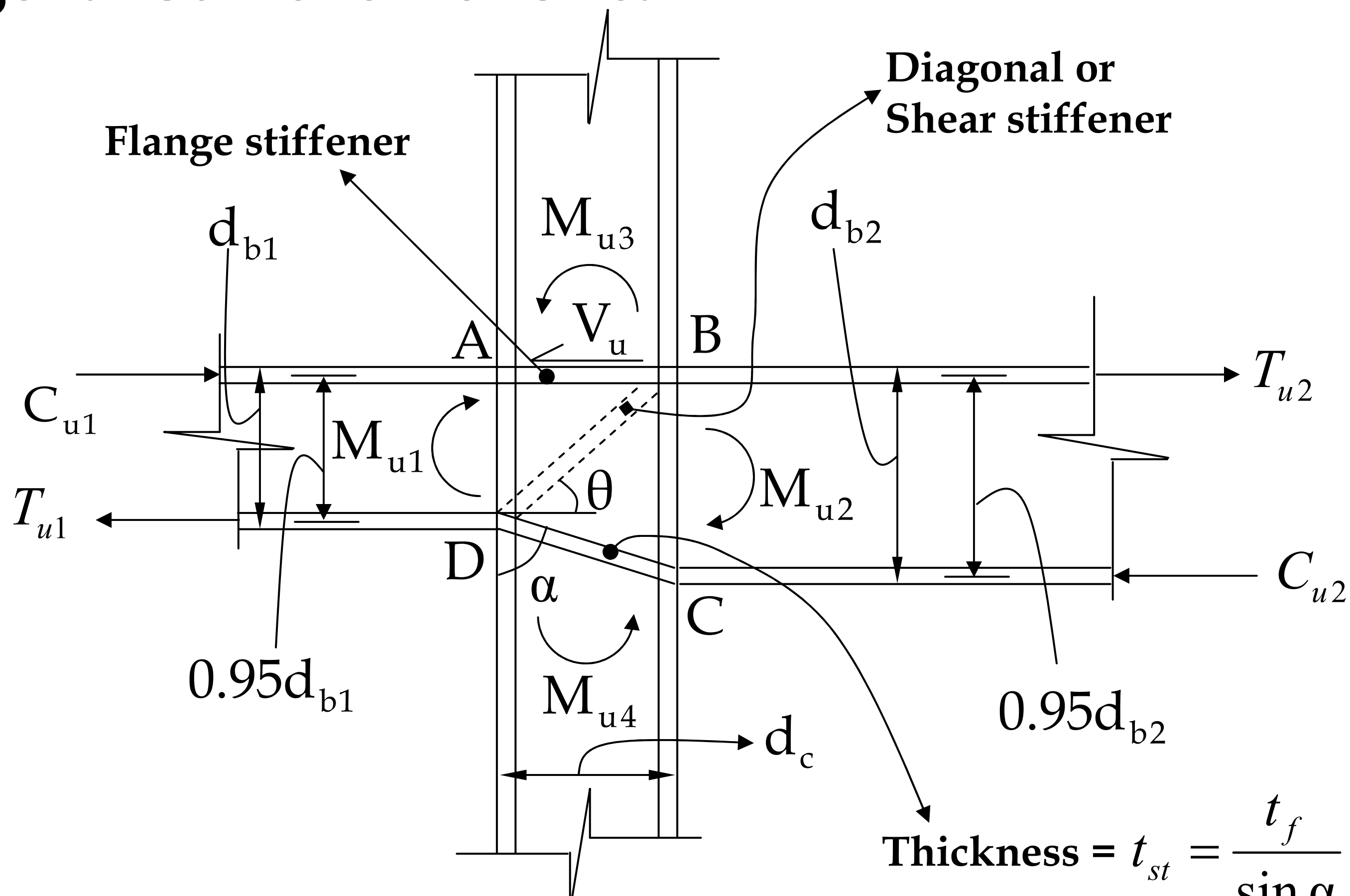




Vertical stiffeners / column flanges give extra strength to web against web crippling. Horizontal stiffener is similar to bearing stiffener. If we don't provide hz. stiffener then beam-flange force will act as point load so web crippling can occur.

Steel Structures

Diagonal Stiffener for Shear



Column moments are considered opposite to beam moments

V_u = Column shear

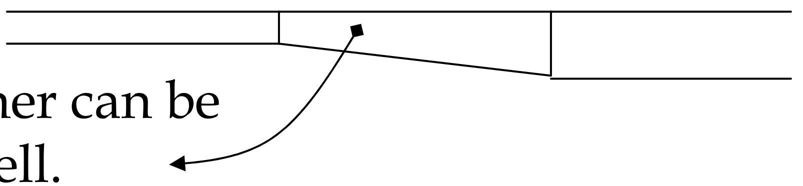
Steel Structures

$$T_{u2} = \frac{M_{u2}}{0.95 d_{b_2}} \quad C_{u2} = \frac{M_{u2}}{0.95 d_{b_2}} \quad \alpha = \tan^{-1}\left(\frac{d_c}{d_{b2} - d_{b1}}\right)$$

Design of Flange stiffeners:

Case I:

If flange stiffener is a continuation of two flanges (on both sides), the size of the stiffener is kept equal to the larger flange. This thickness of the larger flange may gradually be decreased to the size of the smaller flange on the other side.



Flange stiffener can be tapered as well.

If the stiffener is inclined, as in the bottom stiffener of the figure, the thickness of the stiffener has to be increased as follows:

$$t_{st} = \frac{t_{f2}}{\sin\alpha} \qquad \text{where} \qquad \alpha = \frac{d_c}{(d_{b2} - d_{b1})}$$

Case II:

If the flange stiffener is a continuation of flange only from one side, it is designed just like a bearing stiffener of a plate girder for the flange force.

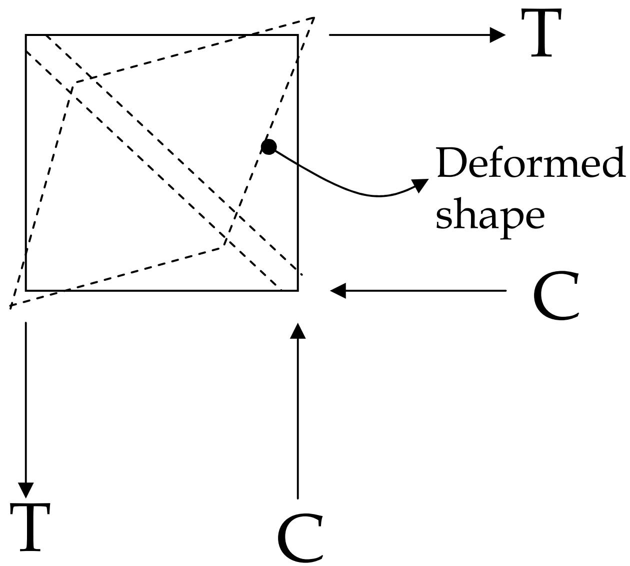
Area of stiffener $$A_{st} = A_{fb} - t_{wc}\left(5k_t t_{fb}\right)$$

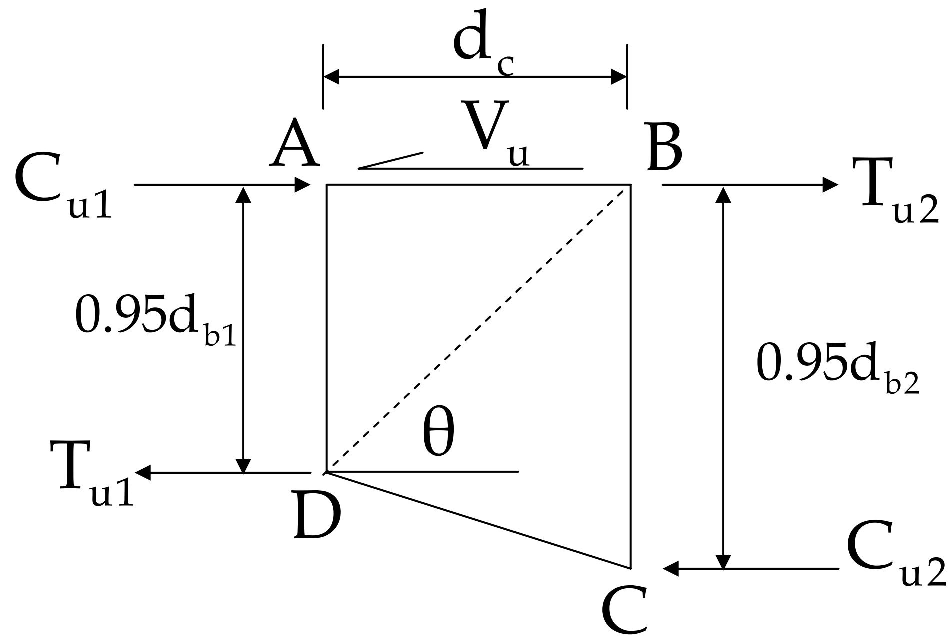
If A_{st} is negative, no stiffener is required.

When this stiffener is required at outer end of a knee joint, size equal to the flange is preferred.

Steel Structures

Check For Web Without Shear Stiffener





V_u = factored shear in the column, it will be opposite to T_{u2} because the column moment will be opposite to larger beam moment for the joint equilibrium.

Steel Structures

Strength of ABCD Without Diagonal Stiffener

Considering the shear acting on edge AB of the web

Total applied factored shear $= C_{u1} + T_{u2} - V_u$

Shear capacity of web, $\phi_v V_n$ $= \phi_v 0.6 F_y t_w d_c$

t_w = Thickness of web within the joint.

$$\phi_v 0.6 F_y t_w d_c = C_{u1} + T_{u2} - V_u$$
$$= \frac{M_{u1}}{0.95 b_{d1}} + \frac{M_{u2}}{0.95 b_{d2}} - V_u$$

If d_{b2} is used in place of b_{d1} the difference is small and it is one safer side as well, because usually M_{u1} will be of opposite sign and lesser value of the corresponding term will give more resultant answer.

Steel Structures

Strength of ABCD Without Diagonal Stiffener

$$t_{w\,req} = \frac{M_{u1} + M_{u2}}{b_{d2}} \times \frac{1}{0.95 \times 0.9 \times 0.6 F_y d_c} - \frac{V_u}{0.9 \times 0.6 F_y d_c}$$

$$t_{w\,req} = \frac{1.95 (M_{u1} + M_{u2})}{F_y b_{d2} d_c} - \frac{1.85 V_u}{F_y d_c}$$

If no load is acting within the column, its inflection point will be almost at the mid-height.

$$V_u \cong \frac{M_{u3}}{h/2}$$

Further assume

$$M_{u4} \cong M_{u3}$$

$$V_u = \frac{2M_{u3}}{h} = \frac{M_{u3} + M_{u4}}{h} = \frac{M_{u1} + M_{u2}}{h}$$

Steel Structures

$$V_u = \frac{M_{u1} + M_{u2}}{13d_b}$$

Assuming $h \approx 13b_d$

$$t_{wreq} = \frac{1.95(M_{u1} + M_{u2})}{F_y d_{b2} d_c} - \frac{1.85(M_{u1} + M_{u2})}{13F_y d_{b2} d_c}$$

$$t_{wreq} = \frac{1.81(M_{u1} + M_{u2})}{F_y d_{b2} d_c}$$

1.95 if $V_u = 0$

If $(t_w)_{available} \geq (t_w)_{req}$, no diagonal stiffener is required otherwise provide diagonal stiffener.

Steel Structures

Strength of Connection With Diagonal Stiffener

$$\phi_v 0.6F_y t_w d_c + A_{st}\phi_c F_{cr}\cos\theta = \frac{M_{u1}+M_{u2}}{0.95d_{b2}} - V_u$$

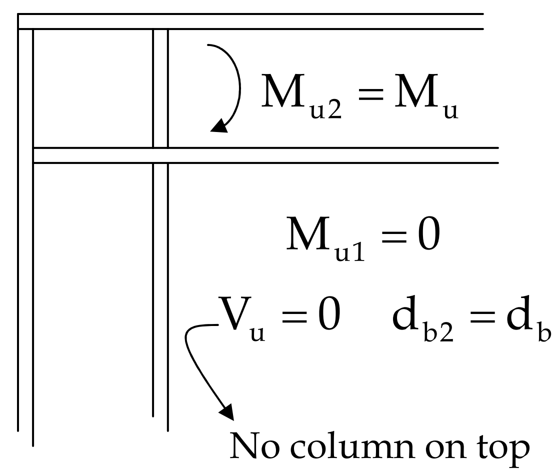
$$\left(A_{st}\right)_{req} = \frac{1}{\phi_c F_{cr}\cos\theta}\left[\frac{M_{u1}+M_{u2}}{1.025d_{b2}} - 0.9\times 0.6F_y t_w d_c\right]$$

0.95 if $V_u = 0$

<u>Other Cases</u>

$$\left(t_w\right)_{req} = \frac{1.95M_u}{F_y d_b d_c}$$

$$\left(A_{st}\right)_{req} = \frac{1}{\phi_c F_{cr}\cos\theta}\left[\frac{M_u}{0.95d_b} - 0.9\times 0.6F_y t_w d_c\right]$$

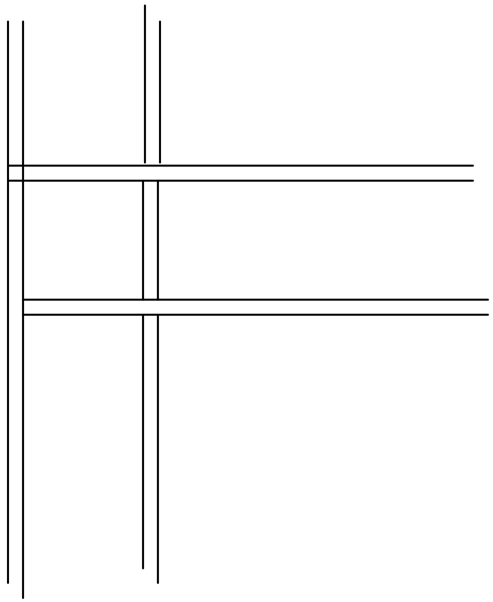


Steel Structures

Other Cases: (contd…)

$$\left(t_w\right)_{req} = \frac{1.81M_u}{F_y d_b d_c}$$

$$\left(A_{st}\right)_{req} = \frac{1}{\phi_c F_{cr}\cos\theta}\left[\frac{M_u}{1.025d_b} - 0.9\times 0.6F_y t_w d_c\right]$$



HAUNCHED CONNECTIONS

Haunched knees may exhibit poor rotation capacity.

Either due to insufficient baring or due to inadequate proportions of the haunch itself, the knee may buckle laterally before the desired design conditions will reach.

Therefore, the design of a haunched connection for use in a plastically designed structure must embody both strength and stability considerations.

The haunch is proportioned with sufficient strength so that the plastic hinge is formed at the end of the haunch.

The haunch is proportioned and is braced in a manner that will provide adequate resistance to lateral buckling.

Basic Assumptions

Following assumptions are made for the design of tapered haunches:

1. The moment diagram is linear from the point of inflection (O) in the beam to the haunch point (H).
2. Plastic hinge is formed at the end of the rolled shape (section R).
3. The length O-R is approximately taken equal to $3d_b$ and represents about as severe a condition as might be encountered in practice.
4. Lateral support will at least be provided at the extremities and at the common intersection points of the haunch.
5. The width of the haunch flange is considered equal to that of the adjoining rolled section.

6. If the angle β is greater than about 12, the critical section will be at section R and no increase in the flange thicknesses of the haunch may be needed. If the angle β is less than 12, flange thicknesses are to be increased to provide moment at section 1 greater than M_1.

7. Within the dimensions d_1 and d_2, the strength of the members in shear must also be checked.

Lateral Stability Without Bracing

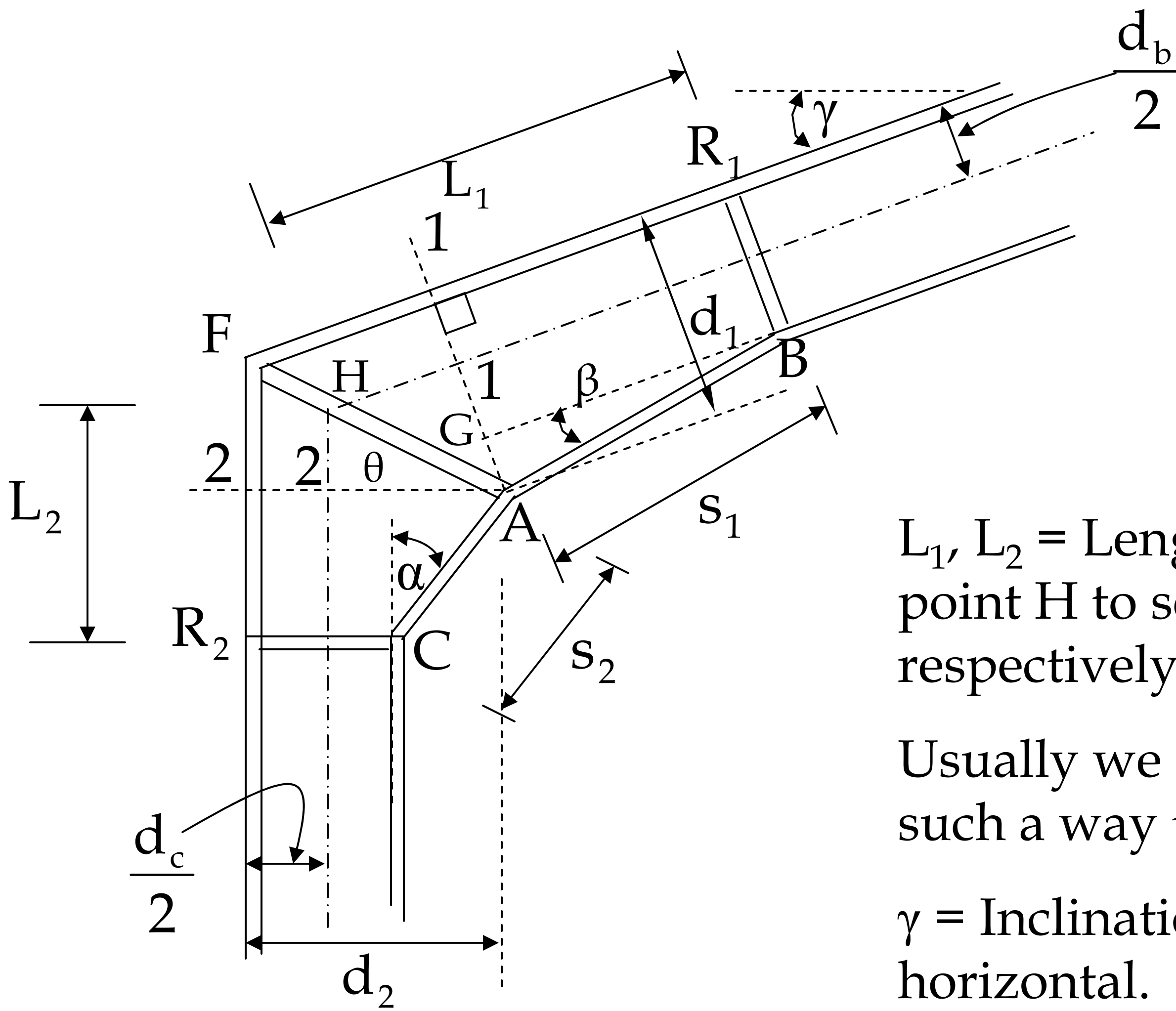
If we want to avoid lateral bracing in-between the haunch, the flange thickness of the haunch is to be increased by the amount Δt.

$$\Delta t = 0.1\,(s / b - 4)\,\mathrm{t_t} \qquad \text{for } \beta \leq 24° \qquad s / b \leq 17$$

$$t_t = [1 + 0.1\,(s / b - 4)]\,t_f \qquad \text{for } s / b > 4$$

Steel Structures

Haunch of a Gable Frame



L_1, L_2 = Length from haunch point H to section R_1 and R_2 respectively.

Usually we design haunch in such a way that $d_1 = d_2 = d_h$

γ = Inclination of girder w.r.t horizontal.

Steel Structures

Haunch of a Gable Frame

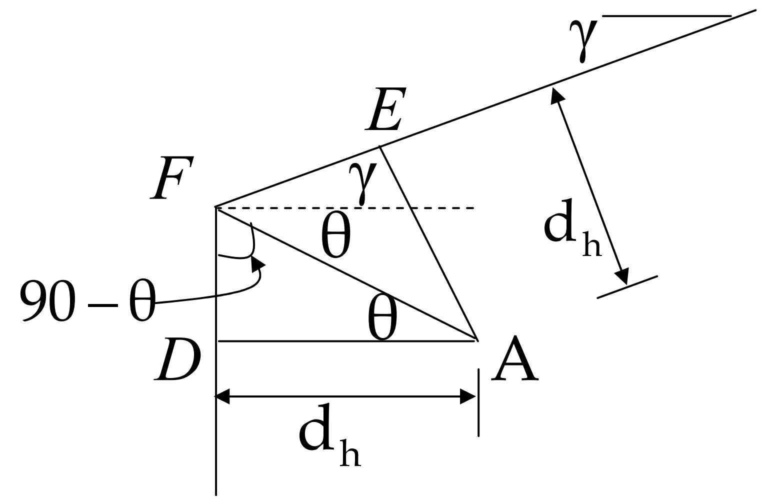
Relationship Between θ & γ

Total angle at F = 90 + γ

$$\text{Angle AFE} = \gamma + \theta$$

$$\text{Angle EAF} = 90 - (\gamma + \theta)$$

Two sides of the triangles ADF and AEF are equal and one angle is 90°, hence, these are similar.



$$\text{Angle DFA} = \text{Angle EFA}$$

$$90 - \theta = \gamma + \theta$$

$$\boxed{\theta = 45 - \frac{\gamma}{2}}$$

Steel Structures

$$H1 = \left(d_h - \frac{d_b}{2} \right) \tan\theta$$

$$H2 = \left(d_h - \frac{d_c}{2} \right) \tan\theta$$

$$BG = L_1 - H1$$

$$\beta = \tan^{-1}\left(\frac{d_h - d_b}{L_1 - H1} \right)$$

$$\alpha = \tan^{-1}\left(\frac{d_h - d_c}{L_2 - H2} \right)$$

$$s_1 = \frac{d_h - d_b}{\sin\beta}$$

$$s_2 = \frac{d_h - d_c}{\sin\alpha}$$

Design of Diagonal Stiffener AF

i) Design For Shear Requirement

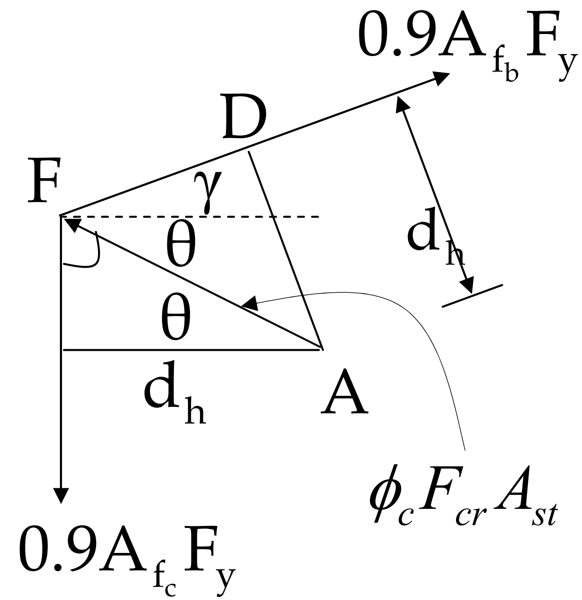
Consider horizontal equilibrium of forces in portion AF

$$0.9A_{f_b}F_y\cos\gamma = \underbrace{V_w\cos\gamma}_{} + A_{st}\phi_c F_{cr}\cos\theta$$

Resistance of web of dimension FD

$$A_{st} = \frac{\cos\gamma\left(A_{f_b}0.9F_y - V_w\right)}{\phi_c F_{cr}\cos\theta}$$

We can take $\phi_c F_{cr} = 0.9\times 0.95F_y$



$$V_w = 0.9\times 0.6F_y t_w FD$$

$$FD = d_h\tan\theta\,, A_t = A_{f_b}$$

Putting all values we get

$$A_{st} = 1.05\frac{\cos\gamma}{\cos\theta}\left(A_t - 0.6t_w d_h\tan\theta\right) \quad \textbf{(1)}$$

ii) Design For Axial Thrust Requirement

A_{c1} = area of lower beam haunch flange

A_{c2} = area of inner column haunch

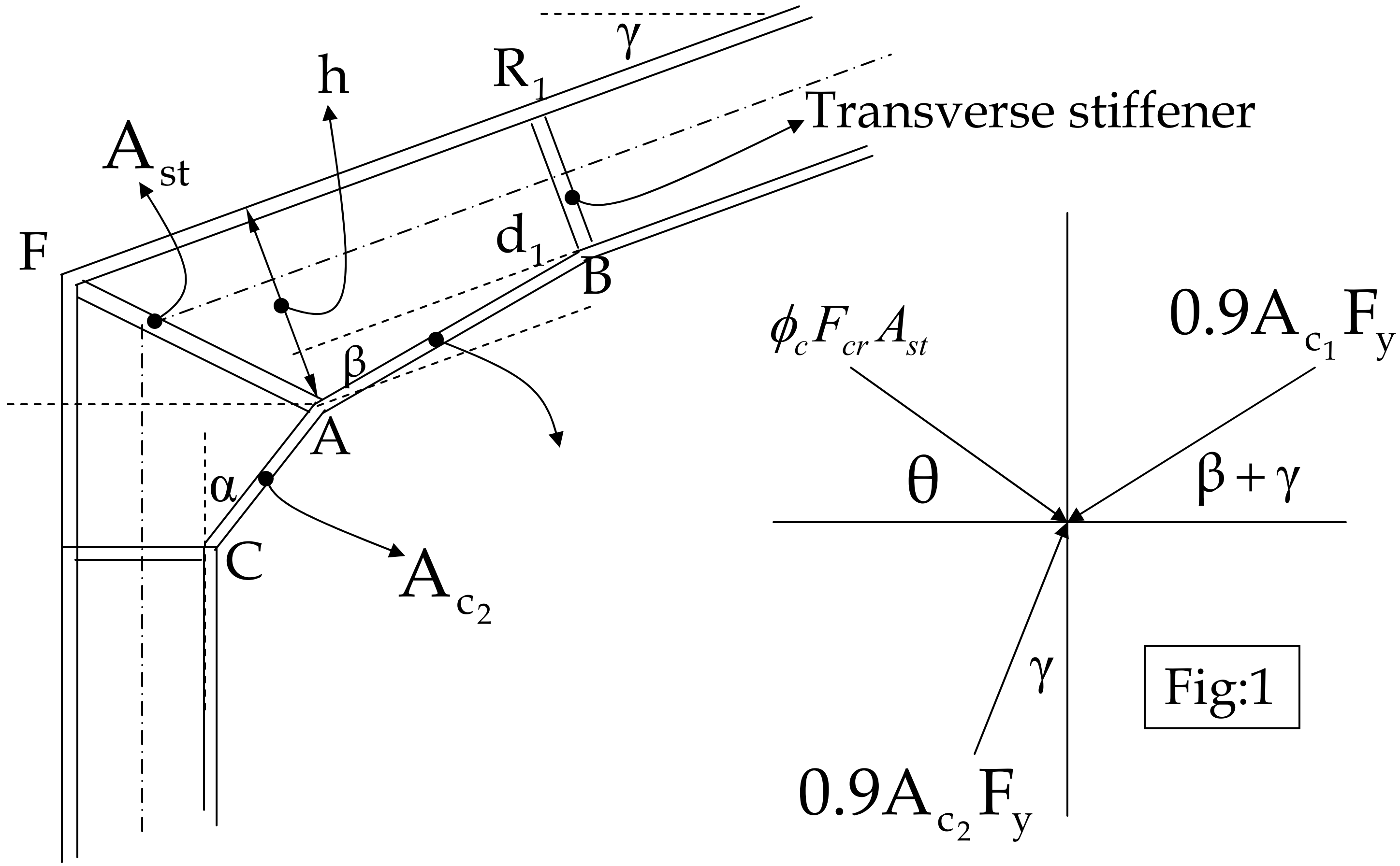


Fig:1

$$A_{c_1} = \frac{A_{tb}}{\cos\beta}$$

$$A_{c_2} = \frac{A_{tc}}{\cos\alpha}$$

ii) Design For Axial Thrust Requirement

From Fig:1

$$\sum F_x = 0$$

$$A_{st}\phi_c F_{cr}\cos\theta = 0.9A_{c_1}F_y\cos(\beta+\gamma) - 0.9A_{c_2}F_y\sin\alpha$$

$$A_{st} = \frac{0.9F_y\left\lfloor A_{c_1}\cos(\beta+\gamma) - A_{c_2}\sin\alpha\right\rfloor}{\phi_c F_{cr}\cos\theta}$$

$$\boxed{A_{st} \cong 1.05\frac{A_{c_1}\cos(\beta+\gamma) - A_{c_2}\sin\alpha}{\cos\theta}} \quad \textbf{(2)}$$

$$\boxed{\frac{0.9F_y}{\phi_c F_{cr}} = 1.05}$$

A_{st} is calculated from Shear effect (1) and Axial effect (2) and greater value is selected.

Transverse Stiffener At End of Haunch

At the ends of the haunch (section R), transverse stiffeners are required for transmitting the compressive thrust.

$$\phi_c F_{cr} A_{st} = 0.9 A_{c_1} F_y \sin\beta$$

$$A_{st} = \frac{0.9 F_y}{\phi_c F_{cr}} A_{c_1} \sin\beta$$

If width is taken equal to other stiffeners / flange width

$$\boxed{t_{st} = 1.05 t_{c_1} \sin\beta}$$ For R_1

$$\boxed{t_{st} = 1.05 t_{c_2} \sin\alpha}$$ For R_2

Design Procedure

1. Select the general proportions of the haunch, like its length along the girder and column and horizontal depth of haunch (d_h).

2. Calculate α, β, & γ and web thickness (mostly equal to larger girder and column web thicknesses).

3. Calculate moments due to loads at sections 1 and 2 and also at the junction of haunch with rolled section.

4. Check the plastic modulus furnished at the common intersection point (section 1). If it is considerably less than the value required from the moment diagram (M_1), the depth d_h must be increased.

   The increased value may be evaluated from the following Equation.

$$(b - t_w)^2\, t_t - d_h\, (b - t_w)\, t_t - t_w /4\, d_h^{\,2} + Z_1 \;=\; 0$$

5. Check s / b ratio and if it is greater than 4, increase the flange thickness.

6. Calculate the thickness of compression flange using the equation:

$$t_c \;=\; t_t / \cos \beta$$

7. Repeat steps 4 to 6 for column portion of the haunch.

8. A diagonal stiffener is then proportioned, but its thickness must not be less than b / 17.

9. Finally transverse stiffeners are proportioned, but again their thicknesses must not be less than b / 17.

Steel Structures

Example: Design Haunch Connection

Beam Section: W 840 x 176

Column Section: W 840 x 251

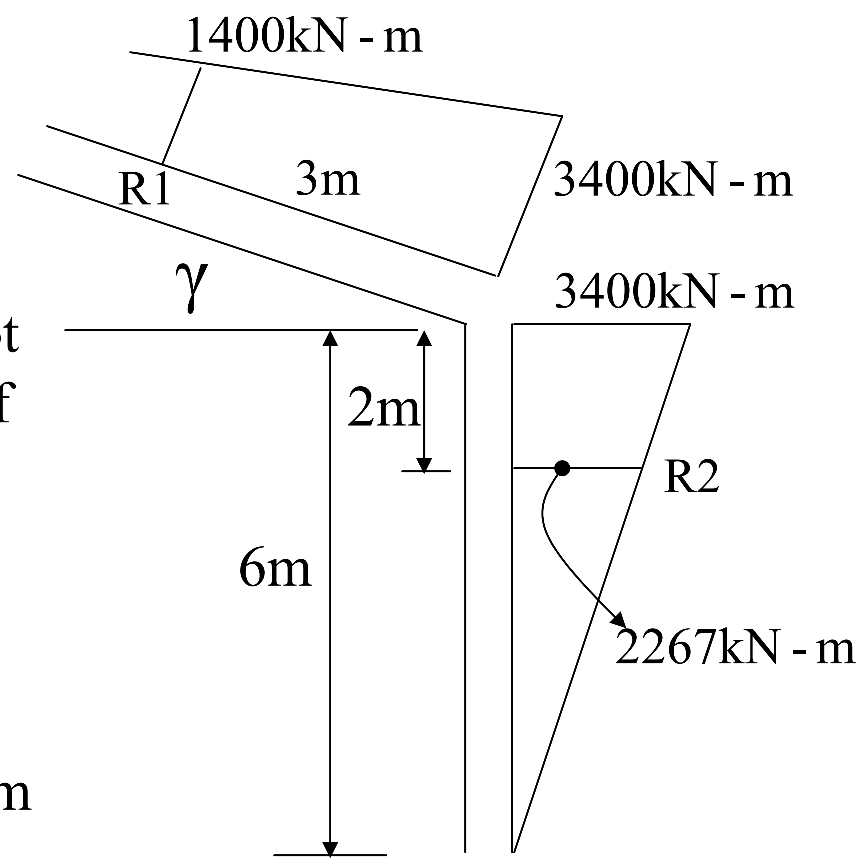
The W360 condition for columns is not considered here because of presence of relatively larger bending moments.

$\gamma = 25^o$

$d_h = 1200mm$

Length of haunch along the girder = 3m

Length of haunch along the column= 2m



A_{c1} = area of lower beam haunch flange

A_{c2} = area of inner column haunch

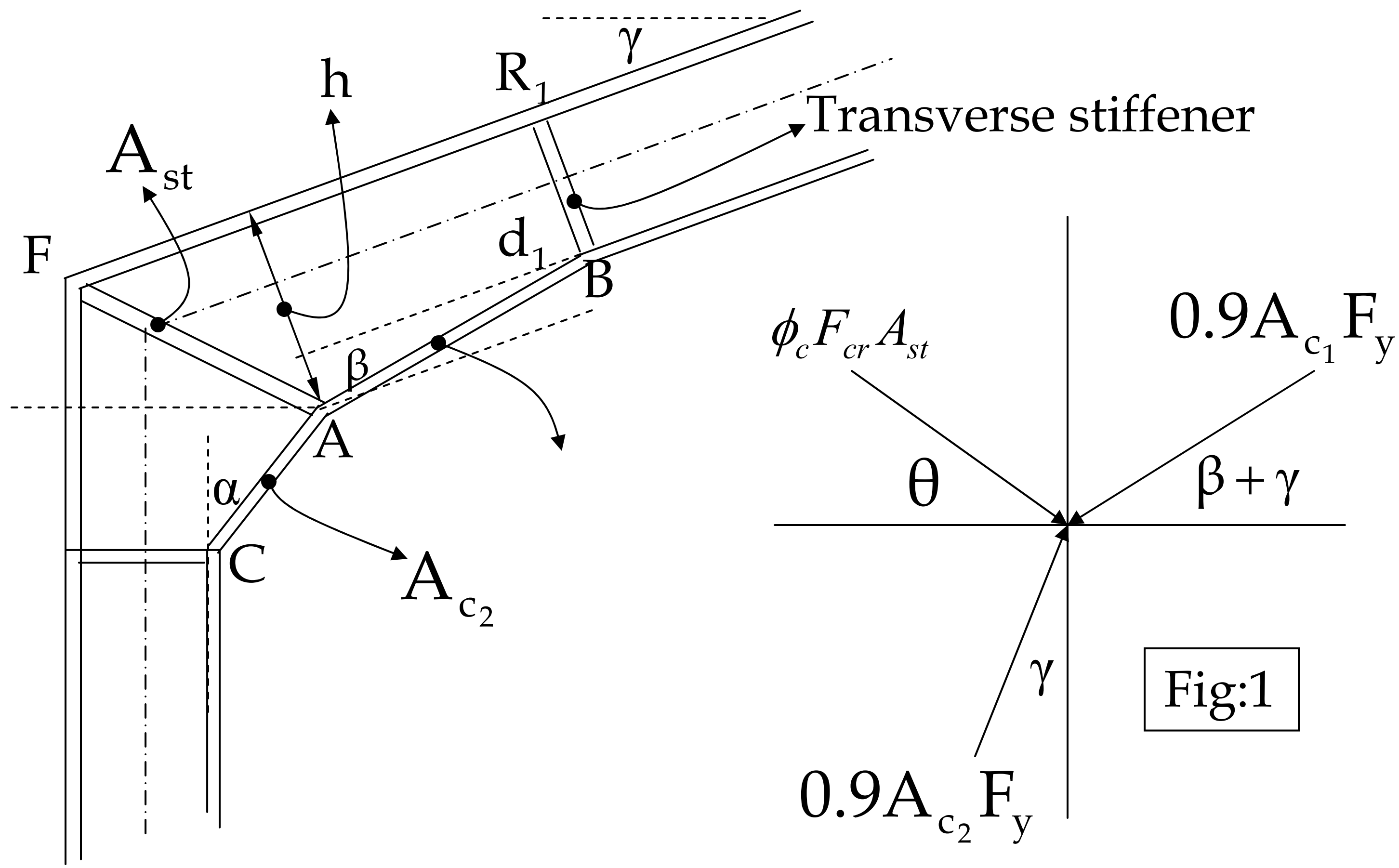


Fig:1

$$A_{c_1} = \frac{A_{tb}}{\cos\beta}$$

$$A_{c_2} = \frac{A_{tc}}{\cos\alpha}$$

Steel Structures

Solution:

$$\gamma = 25^{o}, d_{h} = 1200\,mm, L_{1} = 3m, L_{2} = 2m$$

Column $d = 859mm, b_{f} = 292mm, t_{f} = 31mm, t_{w} = 17mm$

Girder $d = 835mm, b_{f} = 292mm, t_{f} = 18.8mm, t_{w} = 14mm$

$$\theta = 45 - \frac{\gamma}{2} = 32.5^{o}$$

$$H1 = \left(d_{h} - \frac{d_{b}}{2}\right)\tan\theta = \left(1200 - \frac{835}{2}\right)\tan 32.5 = 499mm$$

$$H2 = \left(d_{h} - \frac{d_{c}}{2}\right)\tan\theta = 491mm$$

$$\alpha = \tan^{-1}\frac{d_{h} - d_{c}}{L_{2} - H2} = \tan^{-1}\frac{1200 - 859}{2000 - 491} = 12.7^{o}$$

Steel Structures

$$\beta = \tan^{-1}\frac{d_h - d_b}{L_1 - H1} = \tan^{-1}\frac{1200 - 835}{2000 - 499} = 8.3^o$$

$$s_1 = \frac{d_h - d_b}{\sin\beta} = \frac{1200 - 835}{\sin 8.3} = 2528 \text{ mm}$$

$$s_2 = \frac{d_h - d_c}{\sin\alpha} = \frac{1200 - 859}{\sin 12.7} = 1551 \text{ mm}$$

Let t_w for the haunch web $\approx$ web thickness of column

$= 18$ mm

Steel Structures

Solution: **(contd…)**

$M_1 = 3067 kN - m$

$M_2 = 3122 kN - m$

t_t = thickness of stiffener on tension side

Let $t_t = t_f$ of beam ≈ 20mm

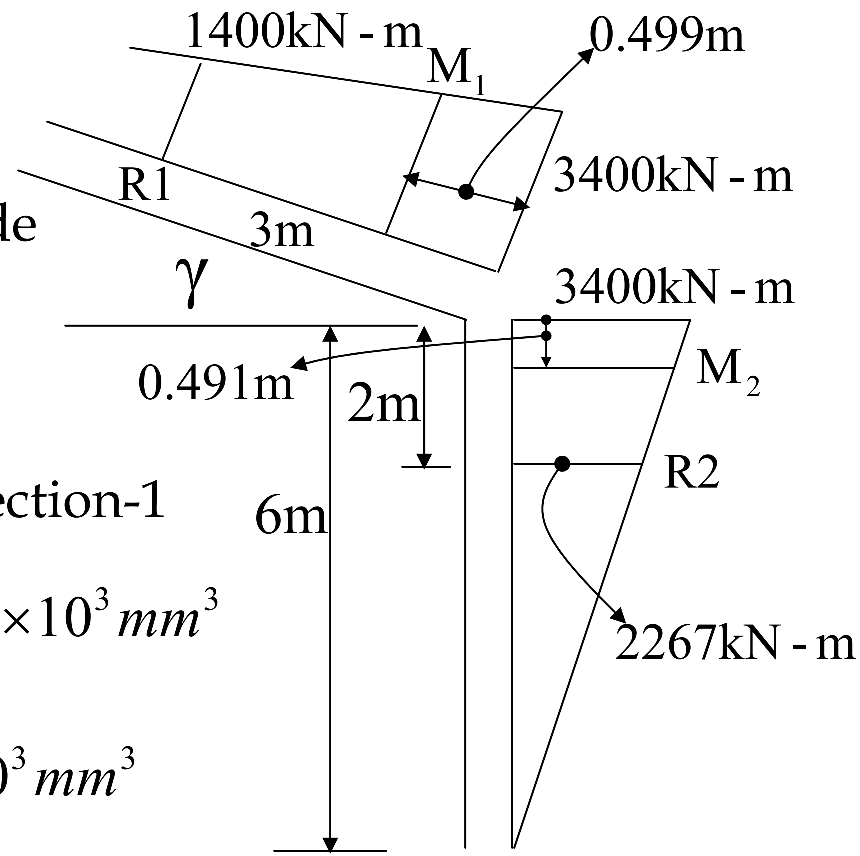
$b_t = b_f$ of beam ≈ 292mm

Plastic Section modulus available at section-1

$$Z_1 = bt_t(d_h - t_t) + \frac{t_w}{4}(d_h - 2t_t)^2 = 12946 \times 10^3 mm^3$$

$$(Z_1)_{req} = \frac{M_1}{0.9F_y} = \frac{3067 \times 10^6}{0.9 \times 250} = 13631 \times 10^3 mm^3$$

$(Z_1)_{avail} < (Z_1)_{req}$ 5% less



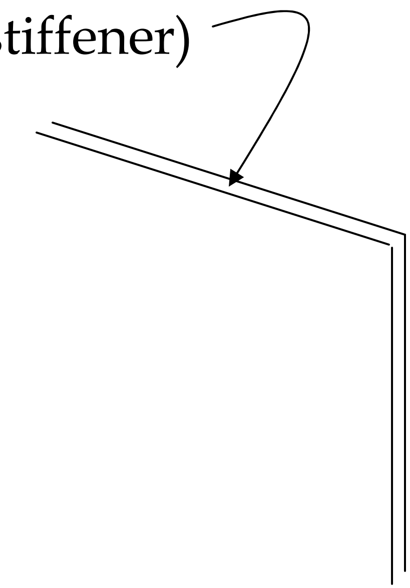
If difference is larger increase architectural dimensions e.g. d_h. If difference is smaller increase tension flange thickness

Steel Structures

Solution: **(contd…)**

Put $(Z_1)_{avail} = (Z_1)_{req}$ and get t_t from the resulting quadratic equation or do trials.

$\Rightarrow t_t = 22.2\text{mm}$ (Provide 25 mm thick outer stiffener)



<u>For lateral stability of haunch</u>

$$t_t = \left[1 + 0.1\left(\frac{s_1}{b} - 4\right)\right] t_t$$

$$\frac{s_1}{b} = \frac{2528}{292} = 8.66 > 4 \quad \textbf{OK}$$

$$t_t = [1 + 0.1(8.66 - 4)]\,22.2 = 32.54mm$$

Larger of 22.2 mm and 32.54mm, rounded up to available size.

$$\boxed{t_t = 35mm}$$

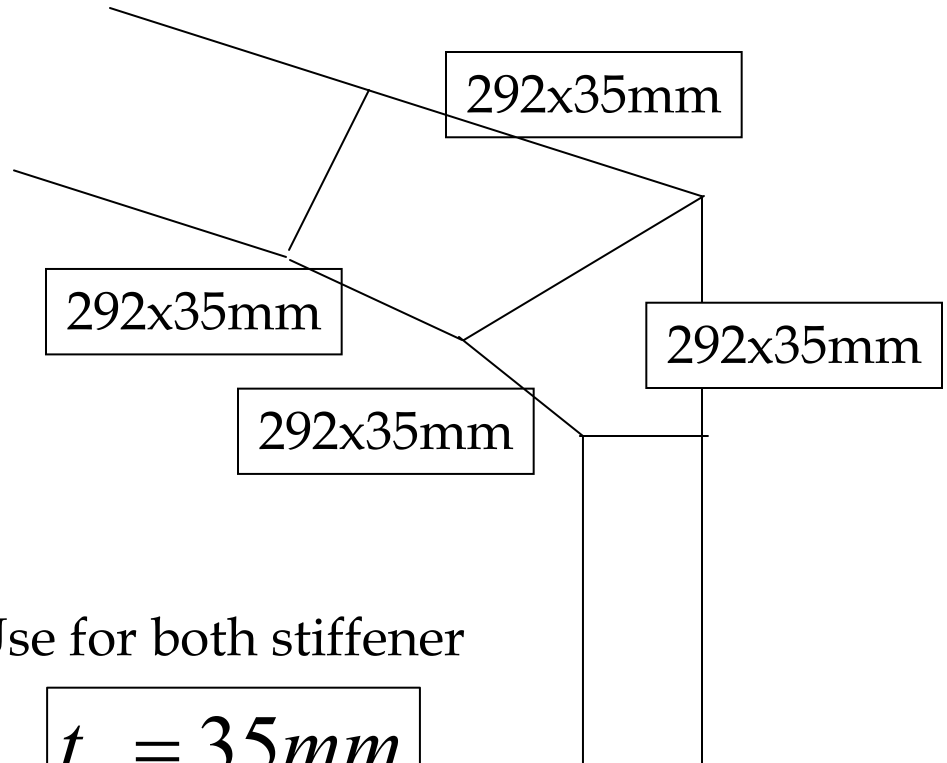
Steel Structures

Compression Flange

$$t_c = \frac{t_t \text{ (req. not considering stability)}}{\cos\beta} \geq 35 \text{ mm}$$

$$= \frac{22.2}{\cos 8.3} = 22.44mm \geq 35 \text{ mm}$$

$$\boxed{t_c = 35mm}$$



Perform same calculations for column side

t_t = 32 mm is required for Z

b = 292 mm, t_t = 35.06mm

$$t_c = \frac{32}{\cos 12.7} \geq 35.06 \quad = 35mm$$

Use for both stiffener

$$\boxed{t_c = 35mm}$$

Steel Structures

Diagonal Stiffener:

<u>For Shear:</u>

For column as this is greater than beam

$$A_{st_1} = 1.05 \frac{\cos\gamma}{\cos\theta}\left(A_t(\text{req}) - 0.6 t_w d_h \tan\theta\right)$$

$$= 1.05 \frac{\cos 25}{\cos 32.5}\left(292\times 32 - 0.6\times 18\times 1200 \tan 32.5\right)$$

$$= 1228 mm^2$$

<u>For Axial:</u>

$$A_{st_2} = 1.05 \frac{A_{c_1}(\text{req})\cos(\beta+\gamma) - A_{c_2}(\text{req})\sin\alpha}{\cos\theta}$$

$$= 1.05 \frac{292\times 22.44\cos(8.3+25) - 292\times 32.8\sin 12.7}{\cos 32.5} = 4197 mm^2$$

$$\boxed{A_{st_2} = 4197 mm^2}$$

Steel Structures

Solution: (contd…)

Diagonal Stiffener:

$$b_{st_2} = \frac{292-18}{2} = 137mm \approx 135mm$$

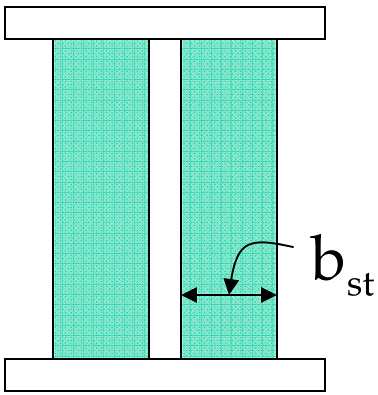
$$2b_{st}t_{st} = A_{st_2}$$

$$t_{st} = \frac{A_{st_2}}{2b_{st}} = \frac{4197}{2\times135} = 15.55mm$$

For local stability

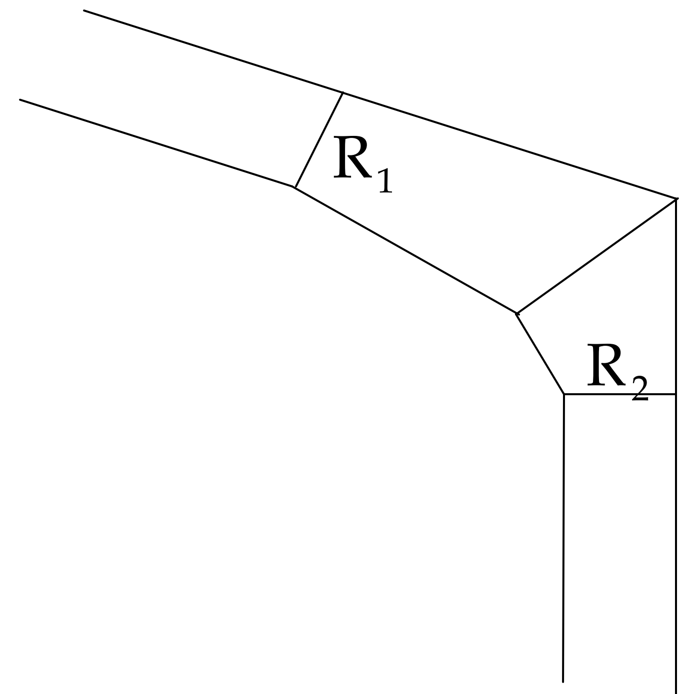
$$(t_{st})_{\min} = \frac{b_{st}}{17} = 7.94mm$$

So $t_{st} = 15.55mm$ say $\boxed{t_{st} = 18mm}$



Steel Structures

Transverse Stiffener:



At R_1

$$b = 292mm$$ From girder

$$t_w = 14mm, 18mm$$ From haunch

$$b_{st} = \frac{292-18}{2} \cong 137mm \quad \text{say } 135 \text{ mm}$$

$$(t_{st})_{min} = \frac{135}{10.8} \cong 12.5mm \quad \text{Say 15 mm}$$

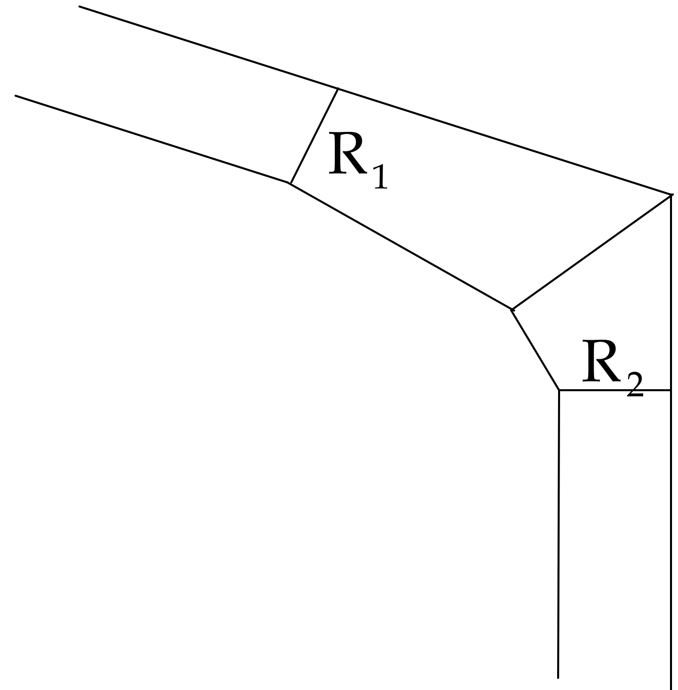
$$t_{st_1} = t_{c_1} \times \sin\beta \times 1.05 = 35 \times \sin 8.3 \times 1.05 = 5.31mm$$

$$\boxed{t_{st} = 15mm}$$

Steel Structures

Transverse Stiffener:

At R_2



$$b = 292mm$$ From column

$$t_w = 17mm, 18mm$$ From haunch

$$b_{st} = \frac{292-18}{2} \cong 137mm \quad \text{say } 135 \text{ mm}$$

$$(t_{st})_{\min} = \frac{135}{10.8} \cong 12.5mm \quad \text{Say 15 mm}$$

$$t_{st_2} = t_{c_2} \times \sin\alpha \times 1.05 = 35 \times \sin 12.7 \times 1.05 = 8.08mm$$

$$\boxed{t_{st} = 15mm}$$