M.SC. STEEL STRUCTURES

LEC. #7

PLASTIC ANALYSIS AND DESIGN

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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 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.
- According to AISC J1.1, where the gravity axis of intersecting axially loaded members do not intersect at one point, the effects of eccentricity must be considered.

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 (Provided within the member; lap and butt joints)
E. Brackets (To connect different members; torque)
F. Bearing joints

BUILDING / FRAME / BEAM CONNECTIONS

- All frame connections have to transfer 100% shear force from beam to other members. The moment transferred depends on the rotational restraint at the joint. The rotational characteristics of the connection are experimentally measured by running the tests and plotting moment rotation for each type of connection.

- Fully Restrained / FR / Moment Connections
- Partially Restrained / PR Connections

  a. Simple or shear or flexible connections

  b. Semi-rigid connections

MOMENT CONNECTIONS (FR)

- 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.

MOMENT CONNECTIONS (FR)

- 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.
- This type of connection must have sufficient strength and stiffness to keep the angle between the adjoining members unchanged.
- For analysis, it is assumed that this type of connection provides no relative rotation between the adjoining members.

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 members should have sufficient strength, stiffness and deformation capacity at the strength limit states.
- PR connections may be further classified into simple and semi-rigid connections.

SHEAR CONNECTIONS

- Simple or shear connections have less than 20 percent rigidity. For e.g. connections of beams, girders and trusses. These connections are allowed to be designed for the reaction shears only.
- 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.

SHEAR CONNECTIONS

- 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.
- A simple connection should possess the rotation capacity equal to that determined during the analysis of the structure considering the a true simply support.

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.

SEMI-RIGID CONNECTIONS

- 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.

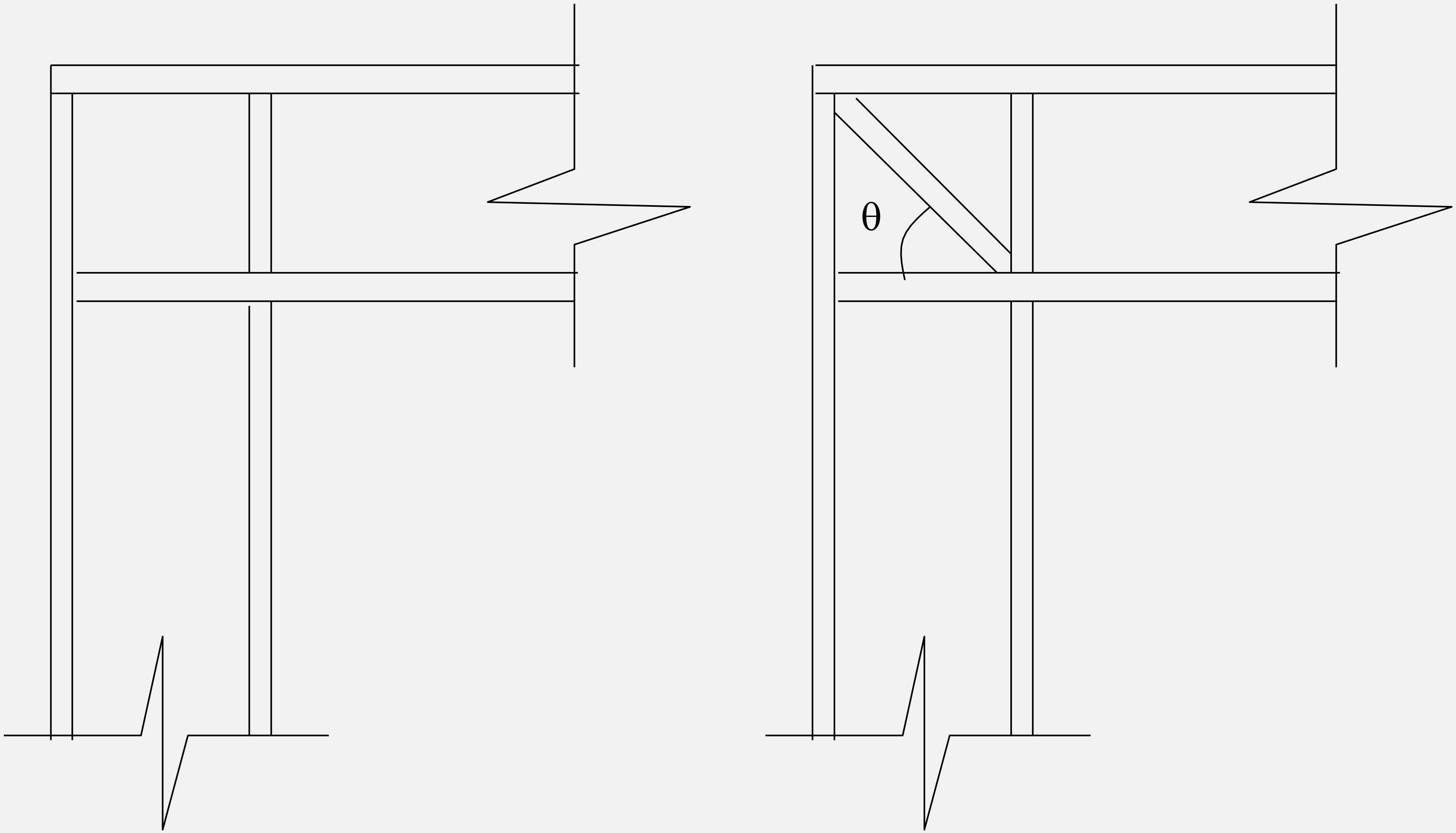
BEARING JOINTS

- 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

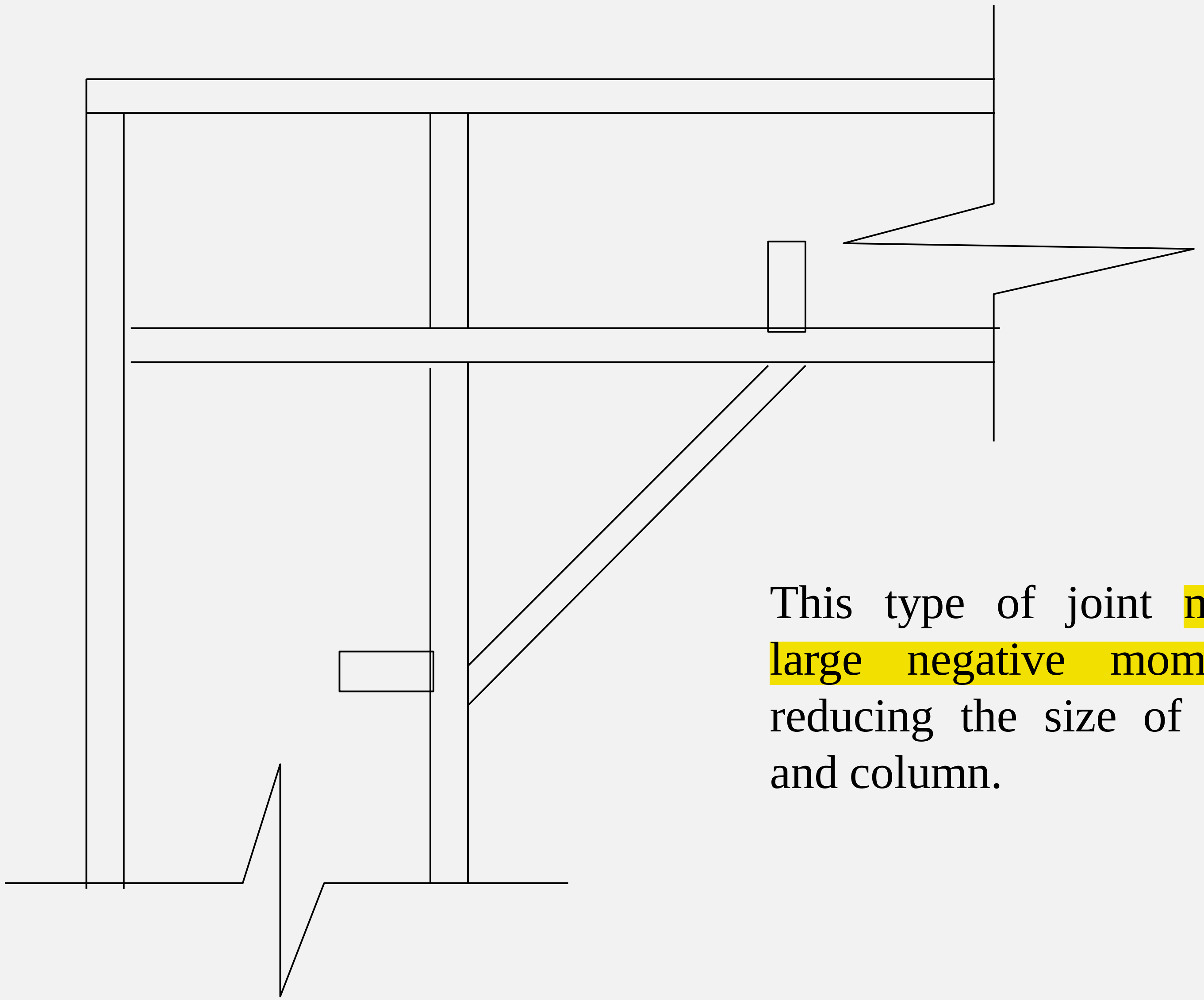
- These are a type of fully restrained (FR) or moment connection.
- In the design of rigid frames the safe transmission of load at the junction of beam and column is of great importance.
- When members join with their webs lying in the plane of the frame, the junction is frequently referred to as a *knee joint* (Figure 1).



a) Square Knee Without Stiffener

b) Square Knee With Stiffener

Figure 1a: Square Knee Joints.



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

Figure 1b: Square Knee With a Bracket.

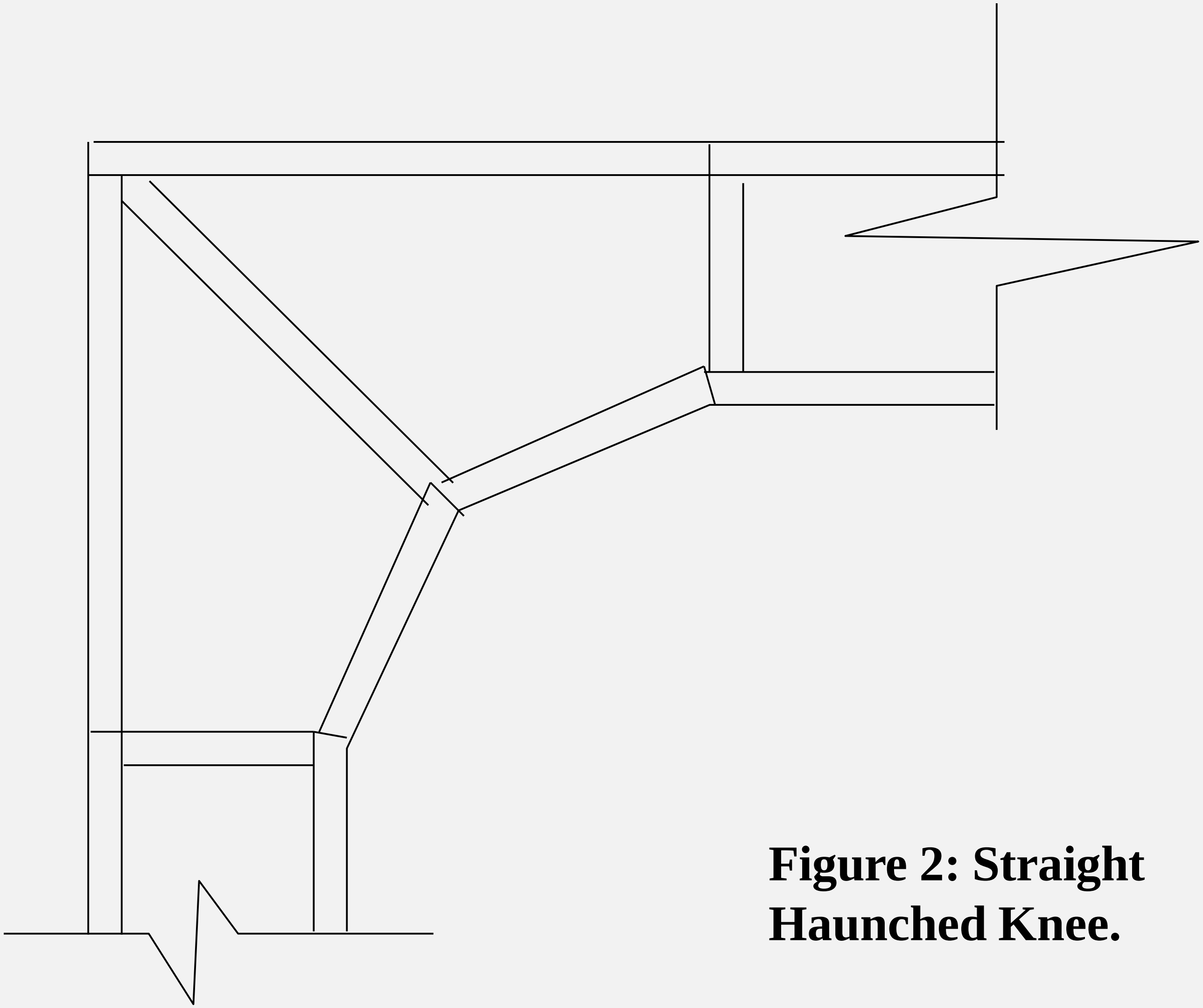


Figure 2: Straight Haunched Knee.

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

Curved Haunched knee is similar to a straight haunched knee with the difference of having a curved inner profile. For haunched knees, variable moment of inertia has to be considered with the knees for both beams and column to perform analysis.

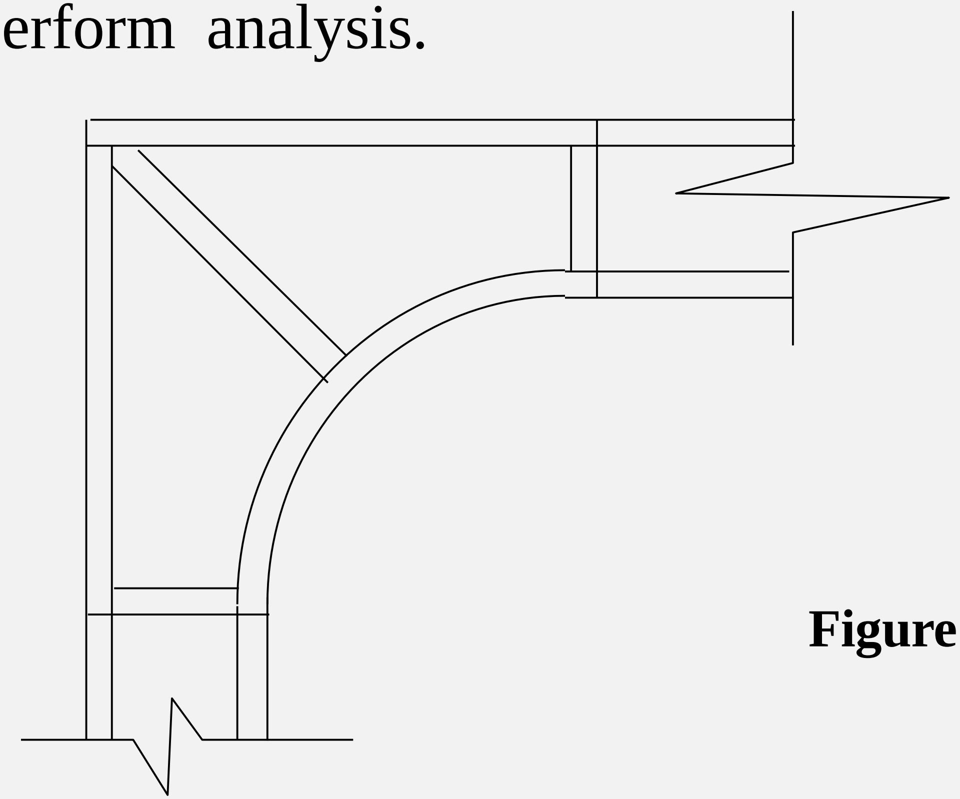


Figure 3: Curved Haunched Knee.

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 4(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 Figure 4(b).

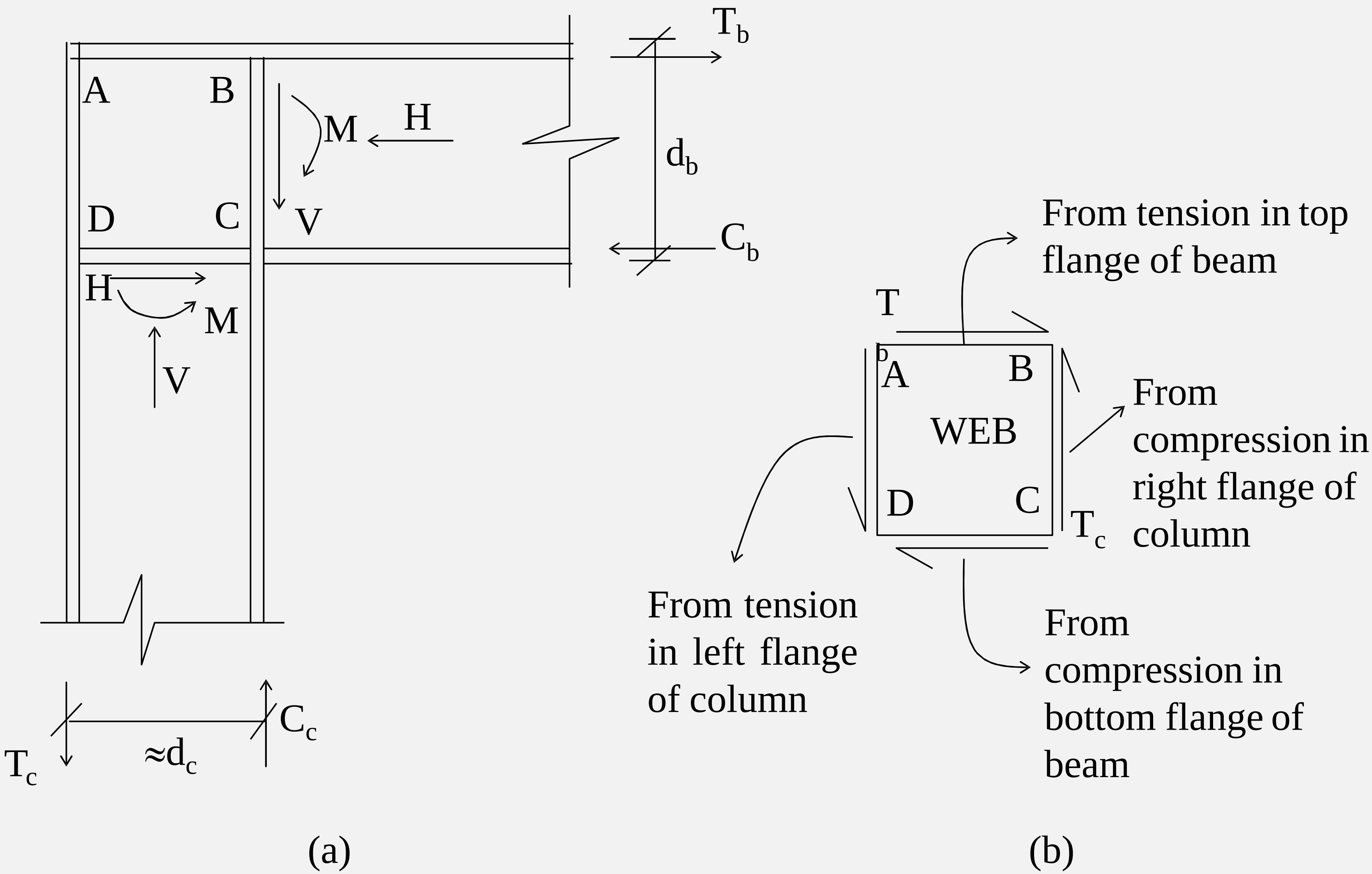


Figure 4: Forces acting on web of a Square knee

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:

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

$$\text{where Yield shear strength } \tau_y = 0.6\, F_y$$

and $\phi_v = 0.9$

$$V_N = T_u,$$

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

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

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

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 5.

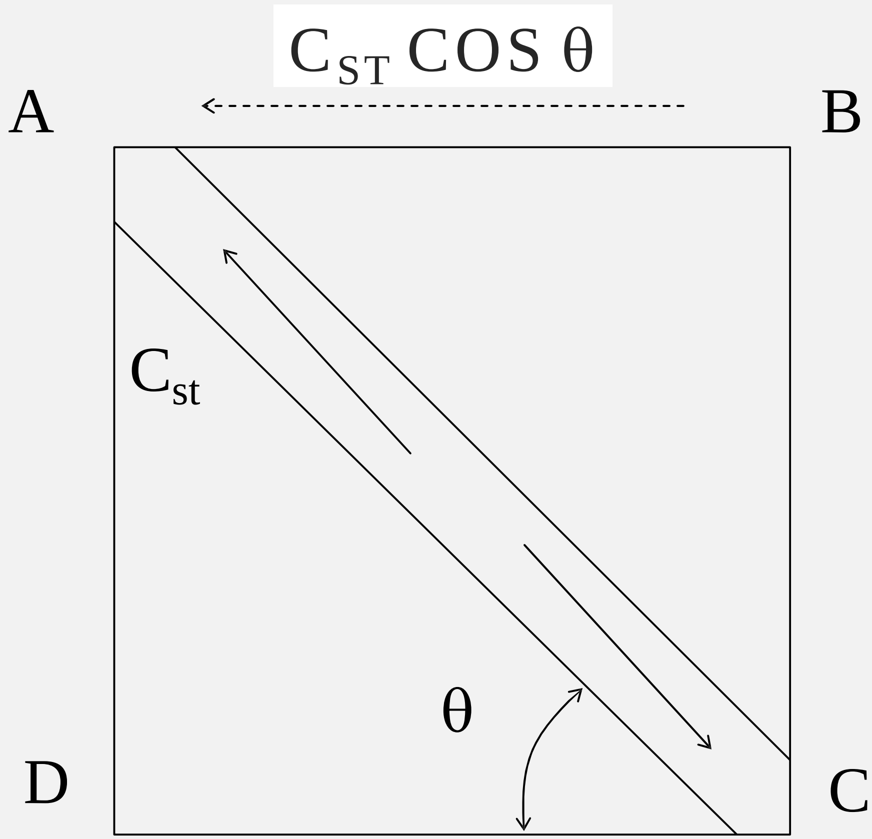


Figure 5: Web of a Square Knee Connection with a Stiffener.

- Stiffener resistive compressive force

$$= \quad C_{st}$$

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

- Applied shear on the web

$$= \quad T_u \quad \textit{(Figure 4b)}$$

- 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.

$$\Sigma F_x = 0 \quad \Rightarrow$$

$$\boxed{T_U \quad = \quad V_{AB} + C_{ST}\, COS\ \theta}$$

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

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

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 6 to join a **W690 x 140 girder** to a **W360 x 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 (Shielded metal arc welding).

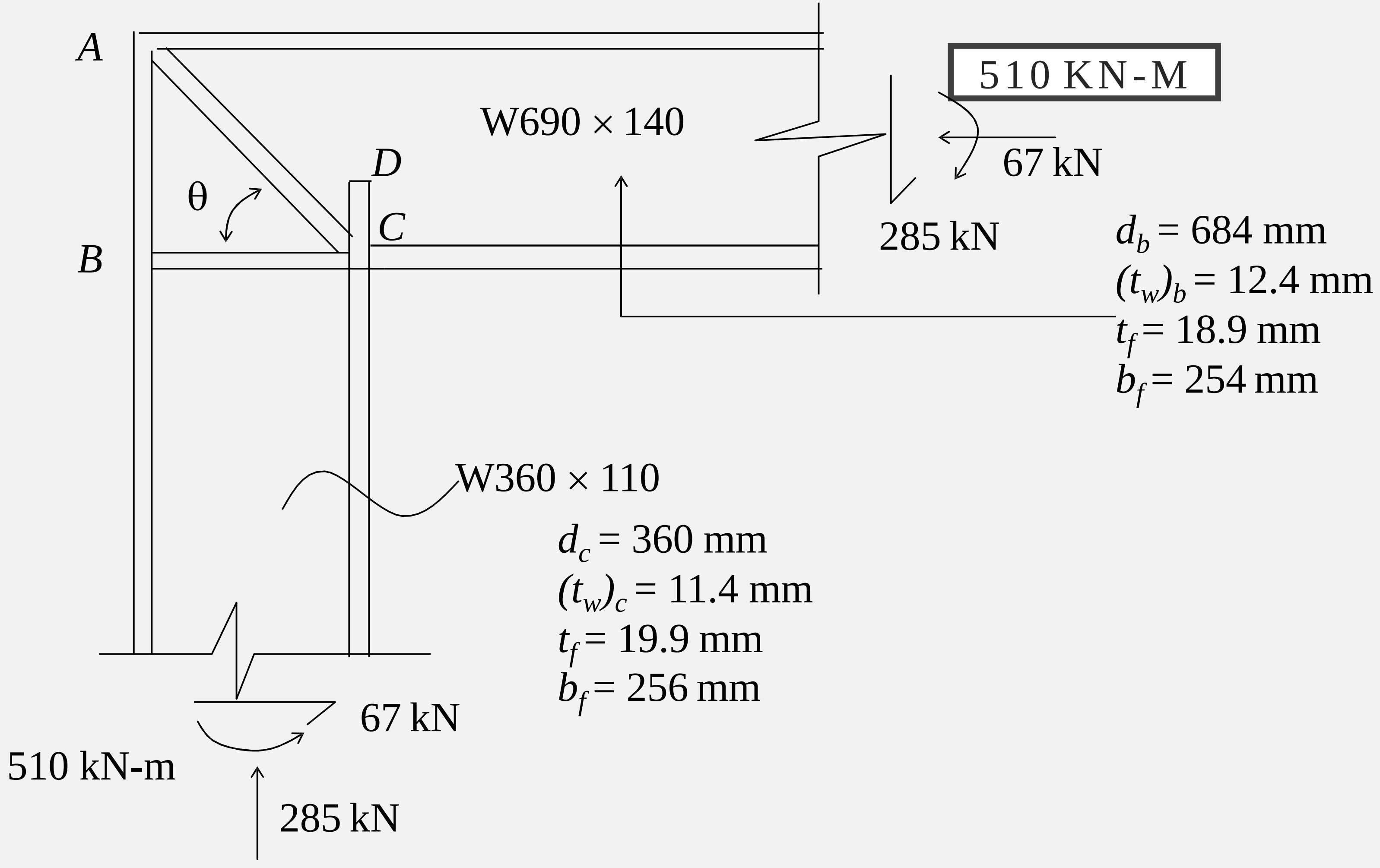


Figure 6 Square Knee of Example 8.6.

SOLUTION

1. Check the web without diagonal stiffener:

$$\text{Required } t_w = \frac{1.95 M_u}{F_y A_{bc}(d_b d_c)}$$

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

$$= 16.15 \text{ mm}$$

$$\text{Actual } t_w = 12.4 \text{ mm for W690} \times 140$$

$$< 16.15 \text{ mm}$$

$\therefore$ **A diagonal stiffener is required.**

2. Stiffener size:

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

$$\Rightarrow \quad \theta = 62.24° \ \ and \ \ \cos\theta = 0.466$$

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

$$(A_{st})_{req} = \frac{1}{0.85\times 238\times 0.466}\left[\frac{510\times 10^6}{0.95\times 684} - 0.9\times 0.60\times 250\times 12.4\times 360\right]$$

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

(half area on one side = 967 mm²)

USING $T_{ST} =$ 12 mm ; $b_{st} = 81$ mm say 85 mm

$$\lambda = \frac{85}{12} = 7.08 < \lambda_r = 15.8 \; \textbf{\textit{OK}}$$

Size of stiffener**:2 PL $_s$ 12 x 85** on both sides of the web of column

3. **Strength of the stiffener acting as a column:**

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

$$= 2 \times 85 + 12.4 = 182.4 \text{ mm}$$

$$r = \sqrt{\frac{\boxed{\frac{T_{ST}(B)^3}{12}}}{t_{st}\, b}} = \sqrt{\frac{1}{12}(b)} = 0.289\, b$$

$$= 52.65 \text{ mm}$$

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

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

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

4. 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 $\phi_T F_Y A_F$. *(WEB DIMENSIONS)*

Flange force $= \phi_t F_y A_w = 0.90 \times 250 \times 19.9 \times 256/1000 = 1146.24$ 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 $= d_b - 2\,t_f$

$$= 684 - 2 \times 18.9 = 646.2 \text{ mm}$$

$$\therefore \quad 0.315\, t_w (646.2) = 1146.24 \text{ (Flange force)}$$

$$t_w = 6 \text{ mm}$$

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

5. 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. (FLANGE DIMENSIONS)

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

$$\text{Tensile component} \ = \ \phi_t F_y t_f$$

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

$\boxed{\text{SHEAR COMPONENT}} = \dfrac{V_u}{d_c - 2t_f} = \dfrac{67}{360 - 2 \times 19.9}$

$= 0.21 \text{ kN/mm}$

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

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

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

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

6.WELD REQUIRED along diagonal stiffeners is designed next. This weld must develop the required stiffener strength.

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

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

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

<u>say 6 mm</u>

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

7. DETERMINE THE REQUIRED LENGTH OF THE STIFFENER *CD*:

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

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

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*.

8. STIFFENER ALONG *CD*:

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

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

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

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

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

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

Length of stiffener $= \frac{\boxed{D - 2T_F}}{2} \cong 325$ mm

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