1.2 Connection configurations:

1.2.1 Simple connections:

Simple connections are assumed to transfer shear only shear at some nominal eccentricity. Therefore such connections can be used only in non-sway frames where the lateral loads are resisted by some alternative arrangement such as bracings or shear walls. Simple connections are typically used in frames up to about five storey in height, where strength rather than stiffness govern the design. Some typical details adopted for simple connections are shown in Fig. 1.3.

The clip and seating angle connection [Fig.1.3 (a)] is economical when automatic saw and drill lines are available. An important point in design is to check end bearing for possible adverse combination of tolerances. In the case of unstiffened seating angles, the bolts connecting it to the column may be designed for shear only assuming the seating angle to be relatively flexible. If the angle is stiff or if it is stiffened in some way then the bolted connection should be designed for the moment arising due to the eccentricity between the centre of the bearing length and the column face in addition to shear. The clip angle does not contribute to the shear resistance because it is flexible and opens out but it is required to stabilise the beam against torsional instability by providing lateral support to compression flange.

The connection using a pair of web cleats, referred to as framing angles, [Fig.1.3 (b)] is also commonly employed to transfer shear from the beam to the column. Here again, if the depth of the web cleat is less than about 0.6 times that of the beam web, then the bolts need to be designed only for the shear force. Otherwise by assuming pure shear transfer at the column face, the bolts connecting the cleats to the beam web should be designed for the moment due to eccentricity.

The end plate connection [Fig. 1.3(c)] eliminates the need to drill holes in the beam. A deep end plate would prevent beam end rotation and thereby end up transferring significant moment to the column. Therefore the depth of the end plate should be limited to that required for shear transfer. However adequate welding should be provided between end plate and beam web. To ensure significant deformation of the end plate before bolt fracture, the thickness of the end plate should be less than one-half of the bolt diameters for Grade 8.8 bolts and one-third of the bolt diameter for Grade 4.6 bolts.

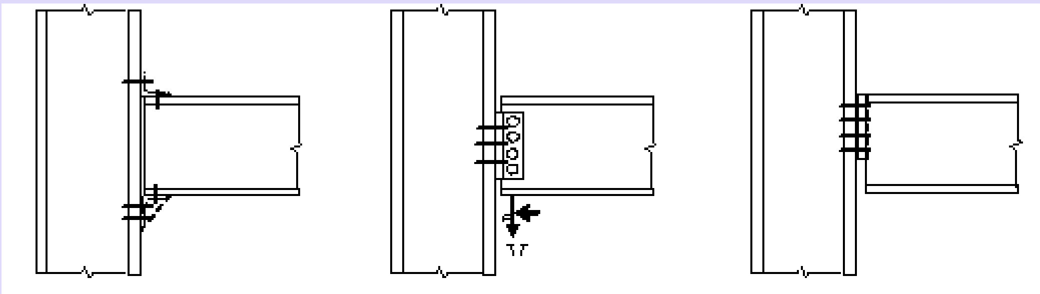

Fig. 1.3 Simple beam-to-column connections (a) Clip and seating angle (b) Web cleats (c) Curtailed end plate

1.2.2 Rigid connections:

Rigid connections transfer significant moments to the columns and are assumed to undergo negligible deformations. Rigid connections are necessary in sway frames for stability and also contribute in resisting lateral loads. In high-rise and slender structures, stiffness requirements may warrant the use of rigid connections. Examples of rigid connections are shown in Fig. 1.4.

Using angles or T-sections to connect beam flanges to the column is not economical due to the large number of bolts required. Further, these connections require HSFG bolts for rigidity. Therefore extended end-plate connections have become the popular method for rigid connections. It is fairly easy to transfer about 0.7 to 0.8 times the yield moment capacity of the beam using these connections. Column web stiffening will normally be required and the bolts at the bottom are for preventing the springing action. These bolts can however be used for shear transfer. In the case of deep beams connected to relatively slender columns a haunched connection as shown in Fig. 1.4c may be adopted. Additional column web stiffeners may also be required in the form of diagonal stiffeners [Fig. 1.4(b)] or web plates [Fig. 1.4(c)].

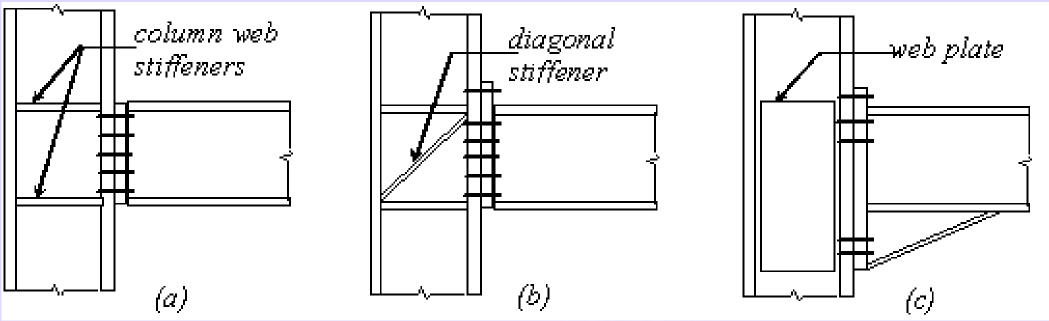

Fig. 1.4 Rigid beam-to-column connections (a) Short end plate (b) Extended end plate (c) Haunched

1.2.3 Semi rigid connections:

Semi-rigid connections are those fall between simple and rigid connections. The fact that most simple connections do have some degree of rotational rigidity was recognised and efforts to utilise it led to the development of the semi-rigid connections. Similarly rigid connections do experience some degree of joint deformation and this can be utilised to reduce the joint design moments. They are used in conjunction with other lateral load resisting systems for increased safety and performance. Use of semi-rigid connections makes the analysis somewhat difficult but leads to economy in member designs. The analysis of semi-rigid connections is usually done by assuming linear rotational springs at the supports or by advanced analysis methods, which account for non-linear moment-rotation characteristics. Examples of semi-rigid connections are shown in Fig. 1.5.

The moment-rotation characteristics will have to be determined based on experiments conducted for the specific design. These test results are then made available as data bases. Simple models are proposed in the form of equations with empirical constants derived based on test results. Depending on the degree of accuracy required, the moment-rotation characteristics may be idealized as linear, bilinear or non-linear curves.

For obtaining the moment rotation relationship the Frye-Morris polynomial model is recommended by IS 800. The model has the form shown in the following equation

$$\theta_r = C_1 (KM)^1 + C_2 (KM)^3 + C_3 (KM)^5$$

Where, K = a standardization parameter dependent upon the connection type and geometry and C_1, C_2, C_3 = curve fitting constants.

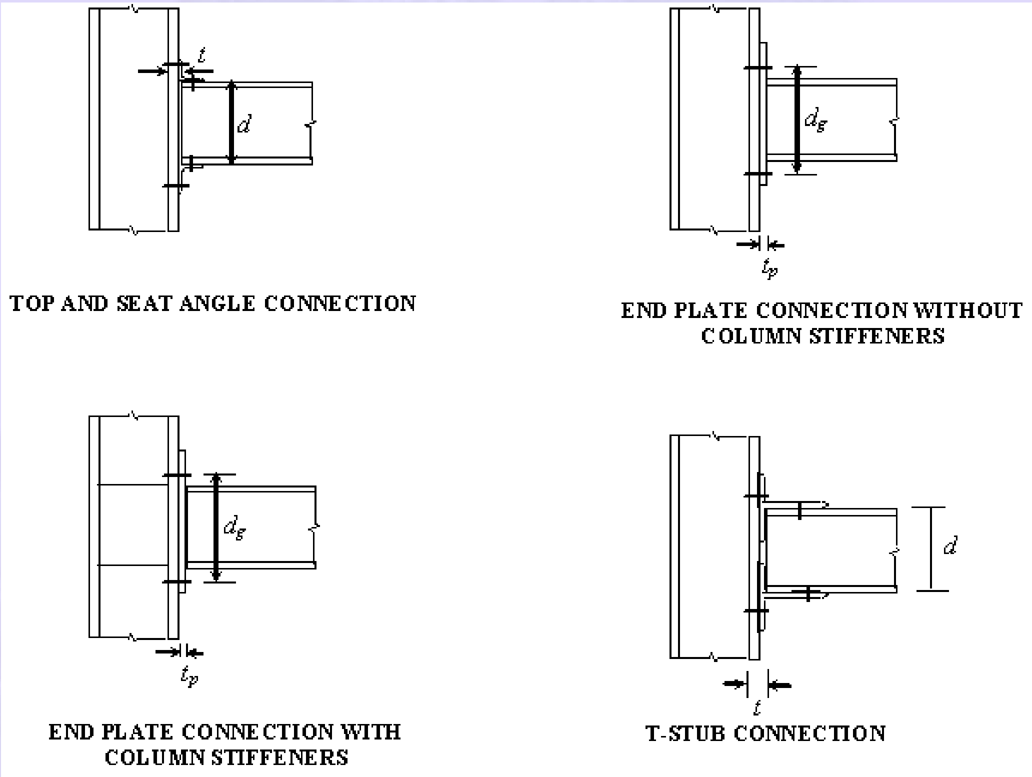

Fig. 1.5 Semi-rigid beam-to-column connections

Table.1.2. shows the curve fitting constants and standardization constants for Frye-Morris Model. (All size parameters are in mm) Depending on the type of connection, the stiffnesses given in Table.1.3 may be assumed either for preliminary analysis or when using a linear moment curvature relationship. The values are based on the secant stiffenesses at a rotation of 0.01 radian and typical dimension of connecting angle and other components as given in the Table 1.3.

The major advantage of semi-rigid connections is that they are cheaper than rigid connections and allow the optimum utilization of the beam member. To understand the second point, consider a beam with simple supports over a span L, subjected to a concentrated load W at mid-span. The mid-span bending moment will be WL/4. On the other hand, if the beam is provided with rigid supports, the maximum moment is WL/8 and occurs at the mid span as well as the support. The moment at the support gets transferred to the column and so may not be desirable. By using a semi-rigid connection we can control the mid span and support moments to the desired value.

Table 1.2. ***Connection constants in frye –morris model***

Connection type	**Curve-fitting constants**	**Standardization constants**
Top and seat angle connection	$C_1 = 8.46 \times 10^{-4}$ $C_2 = 1.01 \times 10^{-4}$ $C_3 = 1.24 \times 10^{-8}$	$K = 1.28\times10^{-6} d^{-1.5} t^{-0.5} l_a^{-0.7} d_b^{-1.5}$
End plate connection without column stiffeners	$C_1 = 1.83 \times 10^{-3}$ $C_2 = -1.04 \times 10^{-4}$ $C_3 = 6.38 \times 10^{-6}$	$K = 9.10\times10^{-7} d_g^{-2.4} t_p^{-0.4} d_b^{-1.5}$
End plate connection with column stiffeners	$C_1 = 1.79 \times 10^{-3}$ $C_2 = 1.76 \times 10^{-4}$ $C_3 = 2.04 \times 10^{-4}$	$K = 6.10\times10^{-5} d_g^{-2.4} t_p^{-0.6}$
T-stub connection	$C_1 = 2.1 \times 10^{-4}$ $C_2 = 6.2 \times 10^{-6}$ $C_3 = -7.6 \times 10^{-9}$	$K = 4.6\times10^{-6} d^{-1.5} t^{-0.5} l_t^{-0.7} d_b^{-1.1}$

Where

d_a = depth of the angle in mm t_a= thickness of the top angle in mm

l_a= length of the angle in mm d_b= diameter of the bolt in mm

d_g= center to center of the outermost bolt of the end plate connection in mm

t_p= thickness of ends- plate in mm

t= thickness of column flange and stub connector in mm

d= depth of the beam in mm l_t= length of the top angle in mm

Table 1.3 Secant stiffnesses

Sl No	Type of Connection	Dimension in mm	Secant Stiffeness kNm/radian
1.	Single Web Connection Angle	d_a=250 t_a=10 g=35	1150
2.	Double Web –Angle Connection	d_a=250 t_a=10 g=77.5	4450
3	Top and seat angle connection without double web angle connection	d_a=300 t_a=10 l_a=140 d_b=20	2730
4	Header Plate	d_p=175 t_p=10 g=75 t_w=7.5	2300

1.3 Summary

The types of connections between beam and column were described. The connection configurations were illustrated and the advantages of semi-rigid connections were outlined. The method of modeling the non linear moment rotation relationships was illustrated.