EXTENDED END PLATE CONNECTIONS IN SPECIAL MOMENT FRAMES

Jayprakash P. Shaha¹*, Dr. M. R. Shiyekar²

¹PG Student, Department of Civil Engineering, Rajarambapu Institute of Technology, Rajaramnagar, Islampur, Dist. Sangli, Maharashtra, India - 415414.

²Research Professor, Department of Civil Engineering, Rajarambapu Institute of Technology, Rajaramnagar, Islampur, Dist. Sangli, Maharashtra, India - 415414.

ABSTRACT:
Moment connections are designed to transfer bending moments, shear forces and sometimes axial forces. Many times in the beam, presence of axial force is insignificant. Hence beam to column connection primarily designed for bending moment and shear. The design strength and stiffness of a moment connection are defined in relation to the strength and stiffness of the connected members. The design strength of a moment connection may be full-strength i.e. the moment capacity of the connection is equal to or larger than the capacity of the connected member or partial-strength i.e. the moment capacity of the connection is less than that of the connected member.

In case of special moment frames, IS:800-2007 has laid down few recommendations regarding design of beam-to-column connections on the basis of limit state design. The various types of connections are extended end plate, flush end plate and T-Stub connection. Thickness of end plate governs the failure of connections. AISC has recommended provision to find the thickness of end plate on the basis of Yield Line theory. No such provision exist in revised IS:800-2007 specification, Connection design procedure for extended end plate is illustrated using existing in IS:800-2007 and checked as per AISC-360-2010.

Keywords: Moment Resisting Connections, Steel Structure, Steel Connection, Steel Beam-Column Joints.

1. INTRODUCTION:
Section deals with the design and detailing requirements for joints between members. Connection elements consist of components such as cleats, gusset plates, brackets, connecting plates and connectors such as rivets, bolts, pins, and welds. The connections in a structure shall be designed so as to be consistent with the assumptions made in the analysis of the structure. Connections shall be capable of transmitting the calculated design actions. Where members are connected to the surface of a web or the flange of a section, the ability of the web or the flange to transfer the applied forces locally should be checked whenever necessary.
2. CLASSIFICATION OF CONNECTIONS:

2.1. Simple Shear Connections: Simple connections are assumed to transfer only shear at some nominal eccentricity and typically used in frames up to about five stories in height, where strength rather than stiffness governs the design. In such frames separate lateral load resisting system is to be provided in the form of bracings or shear walls. The connections shown in following Figure 2.1.1 (a), (b) can be assumed as simple connections in framed analysis and need to be checked only for the transfer of shear from beam to column. Examples of simple connections include single and double web angle connections.

![Figure 2.1.1 Simple Shear Connection](image1)

(a) Single Web Angle  (b) Double Web Angle

2.2. Rigid Connections: Connections with sufficient rotational stiffness may be considered as rigid. Examples of rigid connections include flush end-plate connection and extended end-plate connections. Rigid connections transfer significant moments to the columns and are assumed to undergo negligible deformations at the joint. These are necessary in sway frames for stability and also contribute in resisting lateral loads. The connections shown in following Figure 2.2.1 (a), (b), (c) can be assumed as rigid connection in frame analysis and need to be checked for both shear and moment transfer from beam to the column. Fully welded connections can also be considered as rigid beam to column connections.

![Figure 2.2.1 Rigid Connections](image2)

(a) End Plate without column stiffeners  (b) End Plate with column stiffener  (c) T-Stub

2.3. Semi-rigid Connections: Semi-rigid connections fall between the two types i.e. simple shear connection and rigid connection. The fact is that simple connections do...
have some degree of rotational rigidity as in 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. Examples of semi-rigid connections include top and seat angle connection and top and seat angle with single/double web angles as shown in figure 2.3.1

![Figure 2.3.1 Semi-rigid Connections](image)

(a) Top and Seat Angle without double web angle  
(b) Top and Seat Angle with double web angle

3. **IS 800:2007 Codal Provisions:**

Section 12 of IS 800:2007 consists design and detailing for earthquake loads on steel frames. The section deals with ordinary concentrically braced frames (OCBF), Eccentrically braced frames (EBF), Ordinary moment frames (OMF) and Special moment frames (SMF).

In the present study, specifications related to moment frames such as (OMF) and (SMF) are considered in which bracing members are absent and all lateral resistance is offered by moment resisting connection only.

Following are salient provisions:

### 3.1. Response Reduction Factor (R) for building system

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Lateral Load Resisting System</th>
<th>(R)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Ordinary Moment Frame (OMF)</td>
<td>4</td>
</tr>
<tr>
<td>2.</td>
<td>Special Moment Frame (SMF)</td>
<td>5</td>
</tr>
</tbody>
</table>

### 3.2. Connections, Joints and Fasteners:

3.2.1. All bolts used in frames designed to resist earthquake loads shall be fully tensioned high strength friction grip (HSFG) bolts or turned and fitted bolts.

3.2.2. All welds used in frames designed to resist earthquake loads shall be complete penetration butt welds, except in column splices.

3.2.3. Bolted joints shall be designed not to share load in combination with welds on the same faying surface.
3.3. Special Moment Frames (SMF):

3.3.1 Special moment frames (SMF) shall be made of E250B steel of IS 2062 and should be shown to withstand inelastic deformation corresponding to a joint rotation of 0.04 radians without degradation in strength and stiffness below the full yield value ($M_p$). Special moment frames meeting the requirements of this section shall be deemed to satisfy the required inelastic deformation.

3.3.1.1 Special moment frames (SMF) may be used in any seismic zone [see IS 1893 (Part 1)] and for any buildings (importance-factor values).

3.3.2 Beam-to-Column Joints and Connections

3.3.2.1 All beam-to-column connections shall be rigid and designed to withstand a moment of at least 1.2 times the full plastic moment of the connected beam. When a reduced beam section is used, its minimum flexural strength shall be at least equal to 0.8 times the full plastic moment of the unreduced section.

3.3.2.2 The connection shall be designed to withstand a shear resulting from the load combination $1.2DL + 0.5LL$ plus the shear resulting from the application of $1.2M_p$ in the same direction, at each end of the beam (causing double curvature bending). The shear strength need not exceed the required value corresponding to the load combination

(a) $1.2$ Dead Load (DL) + $0.5$ Live Load (LL) + $2.5$ Earthquake Load (EL)
(b) $0.9$ Dead Load (DL) + $2.5$ Earthquake Load (EL).

3.3.2.3 In column strong axis connections (beam and column web in the same plane), the panel zone shall be checked for shear buckling in accordance with 8.4.2 at the design shear defined in 3.2.2. Column web doubler plates or diagonal stiffeners may be used to strengthen the web against shear buckling.

3.3.2.4 The individual thickness of the column webs and doubler plates, shall satisfy the following:

$$t > \frac{(d_p + b_p)}{90}$$

where $t =$ thickness of column web or doubler plate

$d_p =$ panel-zone depth between continuity plate

$b_p =$ panel-zone width between column flanges

![Figure 3.1 Continuity plates](image)

3.3.2.5 Continuity plates (tension stiffener) (see Figure 3.1) shall be provided in all strong axis welded connections except in end plate connection.

3.3.3 Beam and Column Limitation
3.3.3.1 Beam and column sections shall be either plastic or compact. At potential plastic hinge locations, they shall necessarily be plastic.

3.3.3.2 The section selected for beams and columns shall satisfy the following relation:

\[
\sum M_{pc} \geq 1.2 \sum M_{pb}
\]

where

\( \Sigma M_{pc} = \) sum of the moment capacity in the column above and below the beam centreline; and

\( \Sigma M_{pb} = \) sum of the moment capacity in the beams at the intersection of the beam and column centrelines.

3.3.3.3 Lateral support to the column at both top and bottom beam flange levels shall be provided so as to resist at least 2 percent of the beam flange strength, except for the case described in 3.3.4.

3.3.3.4 A plane frame designed as non-sway in the direction perpendicular to its plane, shall be checked for buckling.

For illustration, a case of extended end plate connection using four bolts is illustrated in detail. Various checks are required to be applied are not detailed out in IS:800-2007, However using design guide of AISC-360-2010 procedure is presented in the context of Indian conditions.

In special moment frames sections are used for beam is ISWB 450 @ 79.4 kg/m and that of column is ISWB 500 @ 95.2 kg/m.

**Following is step by step procedure:**

**Beam Side Design:**

4.1 Connection Design Moment

\[
M_{pe} = \beta_b * Z_p * (f_y / \gamma_m 0) \quad (Cl. 8.2.1.2 p.53 of IS 800 : 2007)
\]

\( = 1 * 1760590 * (250/1.1) \)

\( = 400134090.9 \text{ N.mm} \)

\[
M_{uc} = 1.2 * M_{pe} \quad (Cl.12.10.2.1 p.89 of IS 800 : 2007)
\]

\( = 1.2 * 400134090.9 \)

\( = 480160909.1 \text{ N.mm} \)

4.2 Connection Configuration: Four-Bolt Extended Unstiffened connection:

*Assumed Geometric Design Data*

\( b_p \approx b_r = 200 \text{ mm.} \Rightarrow \text{Use } b_o = 200 \text{ mm.} \)

\( g = 100 \text{ mm.} \)

\( p_f = 50 \text{ mm.} \quad p_{fo} = 50 \text{ mm.} \)
Fig 4.1 Four bolt extended unstiffened end plate connection

\[ d_e = 40 \text{ mm} \]
\[ F_{yp} = 250 \text{ N/mm}^2 \]
\[ F_{up} = 410 \text{ N/mm}^2 \]
\[ F_t = 800/1.25 = 640 \text{ N/mm}^2 \]

Bolts: High Strength Friction Bolt

Using assumed dimensions,
\[ h_0 = 450 + 50 - (15.4/2) = 492.3 \text{ mm} \]
\[ h_1 = 450 - 15.4 - 50 - (15.4/2) = 376.9 \text{ mm} \]

4.3 Determine the required bolt diameter

\[ d_{b req'd} = \sqrt{2 \times Muc/(\Phi \times F_t \times (h_0 + h_1))} \]
\[ = \sqrt{2 \times 480160909.1/(0.75 \times 640 \times (492.3 + 376.9))} \]
\[ = 27.06 \text{ mm} \]

4.4 Select trial bolt diameter and calculate the No Prying bolt moment

Use \( d_b = 28 \text{ mm} \)

Bolt tensile strength

\[ P_t = F_t \times A_b \]
\[ = 640 \times (\Pi \times 28^2/4) \]
\[ = 394081.38 \text{ N} \]

\[ M_{np} = 2 \times P_t (h_0 + h_1) \]
\[ = 2 \times 394081.38 \times (492.3 + 376.9) \]
\[ = 685071075.3 \text{ N.mm} \]

\[ \Phi M_{np} = 0.75 \times 685071075.3 \]
\[ = 513803306.5 \text{ N.mm} > M_{uc} (480160909.1 \text{ N.mm}) \]

“OK”
4.5 Determine the Required End Plate Thickness

End Plate Yield Line Mechanism Parameter

\[ s = \left( \frac{1}{2} \right) \cdot \sqrt{b_p \cdot g} \]

\[ = \left( \frac{1}{2} \right) \cdot \sqrt{200 \cdot 100} = 70.71 \text{ mm} > P_{fi} 50 \text{ mm} \]

\[ Y_p = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left( \frac{1}{p_{fo}} \right) - \frac{1}{2} \right] + \frac{2}{g} \left[ h_1 \left( p_{fi} + s \right) \right] \]

\[ Y_p = 200/2[376.9(1/50)+(1/70.71) + 492.3(1/50) - \frac{1}{2}] + 2/100[376.9(50+70.71)] \]

\[ Y_p = 3131.91 \text{ mm} \]

Required end plate thickness

\[ t_{p, \text{Reqd}} = \sqrt{\frac{1.11 \phi M_{np}}{\phi_b F_{yp} Y_p}} \]

\[ = \sqrt{1.11 \times 0.75 \times 685071075.3 / (0.9 \times 250 \times 3131.91)} \]

\[ = 28.44 \text{ mm} \]

4.6 Select end plate thickness

Use \( t_p = 30 \text{ mm} \)

4.7 Calculate factored beam flange force

\[ F_{fu} = M_{uc} / (d_b - t_{fb}) \]

\[ = 480160909.1 / (450 - 15.4) \]

\[ = 1104834.121 \text{ N} \]

4.8 Check shear yielding of extended portion of end plate

\[ \Phi R_n = \Phi (0.6 F_{yp}) b_p t_p \]

\[ = 0.9 \times 0.6 \times 250 \times 30 \times 30 \]

\[ = 810000 \text{ N} \]

Check Inequality of above equation.

\[ F_{fu} / 2 = 552417.06 \text{ N} < \Phi R_n 810000 \text{ N} \]

“OK”

4.9 Check shear rupture of extended portion of end plate

\[ A_n = [b_p - 2(d_b + 2)]t_p \]

\[ = [200 - 2(28 + 2)]30 \]

\[ = 4200 \text{ mm}^2 \]

\[ \Phi R_n = \Phi (0.6 F_{wp}) A_n \]

\[ = 0.75 \times 0.6 \times 410 \times 4200 \]

\[ = 774900 \text{ N} \]
Check Inequality of above equation.
\[ \frac{F_{u}}{2} = 552417.06 \text{ N} < \phi R_n \ 774900 \text{ N} \]

"OK"

### 4.10 Check Compression Bolts Shear Rupture Strength

Nominal shear capacity of bolt \( V_{dsb} = V_{n}/ \gamma_{mb} \) (Cl. 10.3.3 p.75 of IS 800 : 2007)

\[ V_{n} = (f_{u}/\sqrt{3}) \ast n \ast c/s \text{ area of bolt} \]
\[ = (800/\sqrt{3}) \ast 1 \ast (\Pi/4) \ast 28^2 \]
\[ = 284403.74 \text{ N} \]

\[ V_{dsb} = \frac{V_{n}}{\gamma_{mb}} \]
\[ = \frac{284403.74}{1.25} = 227522.99 \text{ N} \]

Bearing capacity of bolt \( V_{dpb} = V_{npd}/ \gamma_{mb} \) (Cl. 10.3.4 p.75 of IS 800 : 2007)

\[ V_{npd} = 2.5 \ast K_{b} \ast d \ast t \ast f_{u} \]
\[ K_{b} = e/3 \ast d_{o}, P/3 \ast d_{o}, f_{ub}/f_{u}, 1 \text{ (whichever less)} \]
\[ 40/(3 \ast 28) = 0.47, \ 115.4/(3 \ast 28) - 0.25 = 0.8, 1 \]
\[ K_{b} = 0.47 \]
\[ V_{npd} = 2.5 \ast 0.47 \ast 28 \ast 30 \ast 410 \]
\[ = 404670 \text{ N} \]

\[ V_{dpb} = \frac{404670}{1.25} = 323736 \text{ N} \]

Shear Capacity of bolt at least between \( V_{dsb} \) and \( V_{dpb} \)

\[ V_{u} = 227522.99 \text{ N} \]
\[ V_{u} \leq \phi R_n = \phi n_{b} F_{v} A_{b} \]
\[ = 0.75 \ast 4 \ast (800/\sqrt{3}) \ast (\Pi/4) \ast 28^2 \]
\[ = 852778.67 \text{ N} \]

\[ V_{u} = 227522.99 \text{ N} \leq (\phi R_n) 852778.67 \text{ N} \]

"OK"

Bolt Subjected to Combined Shear and Tension:

A bolt required to resist both design shear force \( V_{sd} \) and design tensile force \( T_{b} \) at the same time shall satisfy:

\[ \left( \frac{V_{sd}}{V_{ab}} \right)^2 + \left( \frac{T_{b}}{T_{ab}} \right)^2 \leq 1.0 \]

where

\( V_{sd} = \) factored shear force acting on the bolt = 218.5/8 = 27.31kN

\( V_{ab} = \) design shear capacity = 227522.99 N

\( T_{b} = \) factored tensile force acting on the bolt = 81.937/4 = 20.48 kN.

\( T_{ab} = \) design tension capacity
\[ T_{ab} = T_{nb} / \gamma_{mb} \]
T_{nb} = 0.9 * f_{ub} * A_n \\
= 0.9 * 800 * (\pi/4) * 28^2 \\
= 443341.55 N

\left( \frac{V_{sb}}{V_{ub}} \right)^2 + \left( \frac{T_b}{T_{ub}} \right)^2 \leq 1.0

(27.31 * 1000 / 227522.99)^2 + (20.48 * 1000 / 443341.55)^2 = 0.016 < 1

"OK"

4.11 Design Welds

4.11.1 Beam flange to end plate weld

Weld size for connection of I section to end plate = (1/4) * end plate thickness. \\
= (1/4) * 30 \\
= 7.5 mm

4.11.2 Beam web to end plate weld

Weld size for connection of flange of I section beam to end plate \\
= Thickness of flange \\
= 15.4 mm \\
Use = 10 mm

Column Side Design

4.12 Check the column flange for flexural yielding

s = \left( \frac{1}{2} \right) * \sqrt{bc * g} \\
= (1/2) * \sqrt{200 * 100} = 70.71 mm

c = P_{fo} + t_{sb} + P_{fi} \\
= 50 + 15.4 + 50 = 115.4 mm

Y_c = \frac{b_{bc}}{2} \left[ h_1 \left( \frac{1}{s} \right) + h_0 \left( \frac{1}{s} \right) \right] + \frac{2}{g} \left[ h_1 \left( s + \frac{3c}{4} \right) + h_0 \left( s + \frac{c}{4} \right) + \frac{c^2}{2} \right] + \frac{g}{2}

Y_c = \frac{200}{2} [376.9 (1/70.71) + 492.3 (1/70.71)] + 2/100 [376.9 (70.71 + (3*115.4)/4)] + 492.3 (70.71 + 115.4/4) + 115.4^2/2] + 100/2

Y_c = 3577 mm

Required Unstiffened column flange thickness

\frac{1}{f_{bc, reqd}} = \frac{\phi M_{np}}{F_{bc} Y_c} \\
= \sqrt{1.11 \times 0.75 \times 685071075.3 / (0.9 \times 250 \times 3577)} \\
= 26.84 mm > t_{bc} 15.4 mm

Therefore, add flange stiffeners

Assume, 16 mm stiffeners plate

\[ t_s = 16 \text{ mm} \]
\[ P_{so} = P_{si} = \frac{(c - t_s)}{2} \]
\[ = \frac{(115.4 - 16)}{2} \]
\[ = 49.7 \text{ mm} \]

For stiffened column flange

\[ Y_c = \frac{b_x}{2} \left[ h_1 \left( \frac{1}{s + p_{so}} \right) + h_2 \left( \frac{1}{s + p_{wa}} \right) \right] + \frac{2}{9} \left[ h_1 (s + p_{si}) + h_2 (s + p_{so}) \right] \]

\[ Yc = \frac{200}{2} \left[ 376.9 \left( \frac{(1/70.71) + (1/49.7)}{2} \right) + 492.3 \left( \frac{(1/70.71) + (1/49.7)}{2} \right) \right] + \frac{2}{100} \left[ 376.9 (70.71 + 49.7) + 492.3 (70.71 + 49.7) \right] \]

\[ Yc = 4082.81 \text{ mm} \]

Determine reduced stiffened column flange thickness

\[ t_{fc, req} = \sqrt{\frac{1.11 \phi M_{w}^o}{\phi_b F_{yw} Y_c}} \]
\[ = \sqrt{1.11 \times 0.75 \times 685071075.3 / (0.9 \times 250 \times 4082.81)} \]
\[ = 24.91 \text{ mm} < t_{fc} 15.4 \text{ mm} \]

Therefore, add flange stiffeners

**4.13 Calculate Strength of Unstiffened Column Flange to Determine Stiffener Design Force**

\[ \phi M_{cf} = \phi_b F_{yw} Y_c t_{cf}^2 \]
\[ = 0.9 \times 250 \times 3577 \times 15.4^2 \]
\[ = 190872297 \text{ N.mm} \]

\[ \phi R_n = \phi M_{cf} / (d - t_{bn}) \]
\[ = 190872297 / (450 - 15.4) = 439190.74 \text{ N} < F_{fu} 1104834.12 \text{ N} \]

**4.14 Calculate Web Buckling Strength**

(Cl. 8.7.3.1 p.67 of IS 800 : 2007)

Cross section of column = (b1 + n1).t_w

Where,

b1 = stiff bearing flange for length

n1 = dispersion of load through web at 45° to the level of half the depth of section.

b1 = (half of flange thk of beam + Pfo + de) * 2
\[ = (15.4/2 + 50 + 40) = 97.7 \times 2 = 195.4 \text{ mm} \]

n1 = (225 + 225) = 450 mm

Cross section of column = (195.4 + 450) * 9.2
\[ = 5937.68 \text{ mm} \]

Slenderness ratio = 0.7 * d / (t_w/2\sqrt{3})
\[ = 0.7 \times 450 / (9.2/2\sqrt{3}) \]
Using class ‘C’ curve, \( f_{cd} \) obtained
\[
f_{cd} = 85.226 \text{ N/mm}^2
\]
Web Capacity of column = \( f_{cd} (b_1+n1) \cdot t_w \)
\[
= 85.226 \cdot (195.4+450) \cdot 9.2
= 506044.71 \text{ N} < F_{fu} = 1104834.121 \text{ N}
\]
“Not Good”
Therefore, column stiffener required

**4.15 Calculate Web Crippling Strength** (Cl. 8.7.4 p.67 of IS 800 : 2007)
\[
F_w = ((b_1+ n2) \cdot t_w \cdot f_y / (\gamma_m0))
\]
Where,
- \( b_1 \) = stiff bearing flange for length
- \( n2 \) = length due to dispersion at flange web junction at a slope of 1:25.
  \[
  = 4 \cdot h2 \cdot 2 = 4 \cdot 33.00 \cdot 2 = 264 \text{ mm}
  \]
- \( t_w \) = thickness of web
  \[
  = ((195.4 + 264) \cdot 9.2 \cdot (250/1.1))
  = 960563.63 \text{ N} < F_{fu} = 1104834.121 \text{ N}
  \]
“Not Good”
Therefore, column stiffener required

**4.16 Determine stiffener design force**
\[
F_{cu} = F_{fu} - \min \varphi R_n
\]
\[
439190.74
\]
\[
= 1104834.121 - \min 506044.71
\]
\[
= 960563.63
\]
\[
F_{cu} = 665643.38 \text{ N}
\]

**4.17 Stiffener design**
Area of c/s for stiffener on compression side = \( (b_f - t_w) \cdot t_s \)
\[
= (200 - 9.2) \cdot 16
= 3052.8 \text{ mm}^2
\]
Capacity of stiffener = \( 3052.8 \cdot (250/1.1) \)
\[
= 693818.18 \text{ N}
\]
As Capacity of stiffener 693818.18 N > \( F_{cu} = 665643.38 \text{ N} \)
“OK”

**4.18 Check for Shear Buckling** (Cl. 12.11.2.3 p.90 of IS 800 : 2007)
Using post-critical method:
The nominal shear strength is given by:
\[
V_n = V_{cr}
\]
Where

\( V_{cr} \) - shear force corresponding to web buckling

\[ V_{cr} = A v \tau_b \]

\( T_b \) = shear stress corresponding to web buckling

\( \lambda_w \) = non-dimensional web slenderness ratio for shear buckling stress

\( \tau_{cr,c} \) = the elastic critical shear stress of the web

\[ \frac{\kappa \pi^2 E}{12(1-\mu^2)(\ln \lambda)} \]

= \( 5.35 \times \Pi^2 \times 2 \times 10^5 / (12(1 - 0.3^2) (450/9.2)^2) = 404.214 \)

\[ \lambda_w = \frac{f_{yw}}{f_{yw}^{\frac{1}{3}}} \]

= \( \sqrt[3]{250/(\sqrt[3]{404.214})} = 0.59 \)

\( \lambda_w < 0.8 \) then Use

\[ \tau_{bw} = f_{yw}^{\frac{1}{3}} = 250/(\sqrt[3]{3}) = 144.337 \]

\( V_{cr} = A v \tau_b \) = (area of web) \( \times \tau_b = (500 \times 9.2) \times 144.337 = 663950 \) N = 663.950 kN.

\( V_d \) = design strength = \( V_n/ \gamma_m = 663.950 /1.1 = 603.59 \) kN.

Shear in panel = \((1.2 \times M_p)/c + V = (1.2 \times 400.13)/0.5 + 284.05 = 1244 \) kN > 603.950 kN.

As 603.950 kN is a web buckling resistances as per Clause 8.4.2.2.

Therefore, Remaining shear is to be resisted by stiffener = (1244 - 603.95) = 640.05 kN.

Capacity of stiffener in compression = cross section area of stiffener \( \times F_{cd} \)

\[ = 2 \times 120 \times (250/1.1) \]

\[ = 872727 \) N = 872.72 kN > 640.05 kN

“OK”

4.19 Check for web panel of column at connection (Cl. 12.11.2.4 p.90 of IS 800: 2007)

\[ t > (d_p+b_p)/90 \]

where \( t \) = thickness of column web or doubler plate \( t = 16 \) mm

\( d_p \) = panel-zone depth between continuity plate \( d_p = 468 \) mm

\( b_p \) = panel-zone width between column flanges. \( b_p = 470.6 \) mm

\( 16 > (468+470.6)/90 = 10.42 \) Use doubler plate of thickness 16 mm

“OK”

4.20 Check for Beam and Column Limitation

The section selected for beams and columns shall satisfy the following relation:

\[ \sum \frac{M_p}{M_{ph}} \geq 1.2 \]

where
\[ \Sigma M_{pc} = \text{sum of the moment capacity in the column above and below the beam centreline} \]
\[ \Sigma M_{pb} = \text{sum of the moment capacity in the beams at the intersection of the beam and column centrelines}. \]

\[ \Sigma M_{pc} = M_p \text{ of ISWB 450} = 400134090.9 \text{ Nmm} \]
\[ \Sigma M_{pb} = M_p \text{ of ISWB 450} = 400134090.9 \text{ Nmm} \]
\[ (400134090.9 / 400134090.9) = 1 < 1.2 \]

"Not Good"

Therefore, provide column for greater strength. Using Column ISWB 500 @ 95.2 kg/m,

For ISWB 500 \( Z_p = 2351.35 \text{ cm}^3 \), For beam ISWB 450 \( Z_p = 1760.59 \text{ cm}^3 \)

\[ (\Sigma M_{pc} / \Sigma M_{pb}) = (Z_p,\text{column} / Z_p,\text{beam}) = (2351.35/ 1760.59) = 1.33 > 1.2 \]

"OK"

4.21 Summary:

Beam: ISWB 450

Column: ISWB 500

4E End Plate: 30 mm. by 630 mm.

Bolt Diameter: 28 mm.

Bolt Grade: 8.8 High Strength Friction Grip Bolt

Column Requires Stiffeners

4.22 Final details for 4E moment end plate connection design.

5. CONCLUSIONS:

- Compared to AISC codal provision for Extended end plate connection, Indian Standard specification are too conservative as moment for design of connection is to be taken as 1.2\(M_p\) rather \(M_p\).
- This has resulted over design of connection as compared to AISC provision.
• In section 12 of IS 800:2007, provision for SMF especially for beam to column connection are not sufficient to check the connection details.
• AISC provided many checks such as thickness of end plate, column flange for flexural yielding, column stiffener design, Hence IS 800:2007 needs addition of recommendation in this regards.

REFERENCES: