Bearing splice in a column

The design of column splices is covered in BS EN 1993-1-8 where it is lumped together with the moment resistance of beam-to-column joints. Richard Henderson of the SCI illustrates the design of a column bearing splice considering the strut moment with a numerical example.

Introduction

The design of column splices is a subject that the SCI is asked about from time to time, including whether a design example is available. The Green Book[1], *Simple joints to Eurocode 3*, P358 deals with column splices in Chapter 6. The detailing rules set out in the Green Book do not mention the source of the design moments in the column which are used to check if the column is not in bearing anywhere over the cross section. Traditionally, column splices were introduced close to floor slab level so although the moments due to nominal eccentricity of the floor beams (if unbalanced) were near their maximum, the internal moments in the column were assumed to be small enough to ignore. Requirements to provide fall protection has led to the position of column splices being extended upwards to a height of 1.2 m above floor steelwork level to allow the fixing of temporary handrails. This was discussed in Advisory Desk note AD 314[2]. The internal moments are larger than for a lower splice and should be considered in the splice design.

Column Design – internal bending moment

The design of a column according to BS EN 1993-1-1 essentially follows the Perry-Robertson approach where at failure, the combined axial and bending stress in the extreme fibre is equal to the yield strength of the material. The bending moment (strut moment), is due to the assumed bow imperfection, which is amplified by the axial load. According to the UK National Annex to BS EN 1993-1-1 the bow imperfection must be back-calculated from the design resistance of the column.

The theoretical treatment of elastic buckling of a strut which leads to the elastic critical (Euler) buckling load assumes a deflected shape of a half-sine wave. This can be used to determine the deflection and therefore the bending moment at any position up the column, between points of restraint. Designers who remember the treatment of strut action in BS 5950:2000 Annex C will recognise this as the approach adopted there. A parabolic shape for the curvature could be assumed but this results in larger intermediate displacements and would therefore be on the safe side.

Other design requirements

BS EN 1993-1-8 para. 6.2.7.1(14) states that "Where members are prepared for full contact in bearing, splice material should be provided to transmit at least 25% of the maximum compressive force in the column".

Robustness requirements in Class 2B buildings demand that vertical ties are provided over the height of the building. According to BS EN 1991-1-7 para A.6(2) the column should be capable of resisting an accidental tie force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey. Column splices must therefore carry the vertical tie force which is an accidental load and reduced partial factors apply as a result. Advisory Desk note AD415[3] confirms this and provides additional information.

The stiffness of the column at the splice position must also be such that the column behaves as a continuous element.

Tolerances at the splice position

The National Structural Steelwork Specification (NSSS)[4] includes several clauses relating to permitted deviations at column splices as indicated in Table 1, which may also be found in BS EN 1090-2[5]. The design of the splice must be sufficient to accommodate the maximum deviations allowed in the specification.

Clause	Parameter	Requirement
7.2.3	Squareness of ends prepared for bearing	Ends prepared with respect to longitudinal axis of member. Plan or elevation of end $\Delta = D/1000$
9.6.10	Column splice alignment and gap between bearing surfaces	Local angular misalignment ($\Delta\theta$) occurring at same time as gap (Δ). $\Delta\theta = 1/500$. $\Delta = 0.5$ mm over at least ⅔ of the area with a maximum of 1.0 mm locally.
9.6.11	Eccentricity at column splice	Non-intended eccentricity ($e = e_x$ or e_y) about either axis. $e = 5$ mm
9.6.12	Straightness of a spliced column between adjacent storey levels.	Location (Δ) of the column in plan relative to a straight line between position points at adjacent storey levels. $\Delta = s/750$* with $s \leq h/2$ *This value is $s/1000$ in BS EN 1090-2

D = width or depth of member;
s = height of splice above lower storey; h = storey height

Table 1: Manufacturing and installation tolerances

Design Example

The following example illustrates the design method. Consider a column splice supporting five floors above. The column length below the splice extends over three storeys. Storey heights are 4.0 m. Each floor applies a load of 2800 kN. A permanent action of 3.6 kPa and a variable action of 5 kPa are assumed.

To calculate the design axial compression at the splice level, the variable action reduction factor α_n given in NA.2.6 of the UK NA to BS EN 1991-1-1[6] has been calculated.

For 5 storeys, $\alpha_n = 1.1 - n/10 = 1.1 - 5/10 = 0.6$

According to NA.2.6 the same reduction factor is used to calculate the design axial compression at the base of the lower column, which supports eight storeys.

The design compression at the splice is therefore $5 \times 2100 \times 10^{-3} = 10.5$ MN. The maximum design compression in the lower column section is 16.8 MN.

▶26

▶24 Assuming S355 steel, from the Blue Book[7], a 356 × 406 UC 467 has a resistance $N_{b,z,Rd}$ of 17.1 MN with a buckling length of 4 m. A 356 x 406 UC 287 has a resistance $N_{b,z,Rd}$ of 10.6 MN for the same buckling length. These section sizes will be adopted for the lower and upper lengths of column respectively. Relevant properties for the upper column length are given in Table 2.

Property	Value
Major axis second moment of area I_y (cm⁴)	999000
Minor axis second moment of area I_z (cm⁴)	38700
Major axis elastic modulus $W_{el,y}$ (cm³)	5070
Minor axis elastic modulus $W_{el,z}$ (cm³)	1940
Area A (cm²)	366
Flange thickness t_f (mm)	36.5
Web thickness t_w (mm)	22.6
Yield strength f_y (MPa)	345 ($16 \leq t_f \leq 40$)
Imperfection factor, α for rolled section with $h/b \leq 1.2$, $t_f \leq 100$ mm	0.49 (minor axis) 0.34 (major axis)

Table 2: Design parameters

Effect of bending moment

Based on a 4 m storey height, for the minor axis, the elastic critical load is

$$N_{cr} = \frac{\pi^2 E I_z}{L^2} = \frac{\pi^2 \times 210 \times 10^6 \times 3.87 \times 10^{-4}}{16} = 50{,}131 \text{ kN (50.13 MN)}$$

The amplifier due to axial loads is:

$$\frac{N_{cr}}{N_{cr} - N_{Ed}} = \frac{50.13}{50.13 - 10.5} = 1.27$$

The initial bow imperfection is given by:

$$e_o = \frac{W_e}{A} \alpha (\bar{\lambda} - 0.2)$$

The non-dimensional slenderness and initial bow imperfection are then:

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} = \sqrt{\frac{3.66 \times 10^{-2} \times 345}{50.13}} = 0.502$$

$$e_o = \frac{1940}{366} \times 0.49 \times (0.502 - 0.2) = 0.784 \text{ cm} = 7.8 \text{ mm}$$

The amplified bow is 9.9 mm or about 10 mm. At the splice position, say 1.2 m up the column, the proportion of the maximum bow is given by sin(π × (1.2/4.0)) = 0.81. The design minor axis bending moment at the splice due to strut action is therefore:

$M_{z,Ed}$ = 0.81 × 0.01 × 10,500 = 85.1 kNm.

A similar calculation for the major axis strut moment gives $M_{y,Ed}$ = 49.3 kNm. If the reactions from the floor beams at the relevant floor levels are equal on opposite sides of the column, the strut moment is the only bending moment on the splice.

The axial and minor axis bending stresses are given by:

$$f_{tot} = f_c \mp f_b = \frac{10.5}{3.66 \times 10^{-2}} \mp \frac{85.1 \times 10^{-3}}{1940 \times 10^{-6}} = 287 \mp 43.9 \text{ MPa}$$

The cross section is always in compression at the splice.

Material for 25% of compressive force

Typical details of splices are given in The Green Book, which have been modified slightly for this example. The proposed arrangement is shown in Figure 1.

According to BS EN 1993-1-8 para 6.2.7.1(14), splice materials should be provided to transmit at least 25% of the maximum compressive force in the column (10.5 MN), which is 2625 kN, The bolts and splice plates will be verified against this design force. Assuming f_y = 345 MPa for the splice material (over 16 mm thick), the area required is 7610 mm². With two flange cover plates, the area provided is 14000 mm² (fastener holes can be ignored according to BS EN 1993-1-1 para 6.2.4(3)).

M30 property class 8.8 bolts have been chosen: three pairs in each flange in single shear and one pair in the web in double shear, on each side of the joint. Choosing property class 10.9 bolts does not reduce the number of bolts required. The resistances of a bolt are given in the Blue Book as 215 kN in single shear and 431 kN in double shear.

The shear resistance of the bolts in the flanges of the upper half of the joint is reduced by the presence of packs, which are 21.5 mm thick. The reduction factor β_p is given by clause 6.6.1(12) as

$$\beta_p = \frac{9d}{8d + 3t_p} = \frac{9 \times 30}{8 \times 30 + 3 \times 21.5} = 0.89$$

The shear resistance of the bolts in the flanges of the upper half of the joint is therefore

215 × 0.89 = 191 kN

With the particular geometry of the bolt groups shown in

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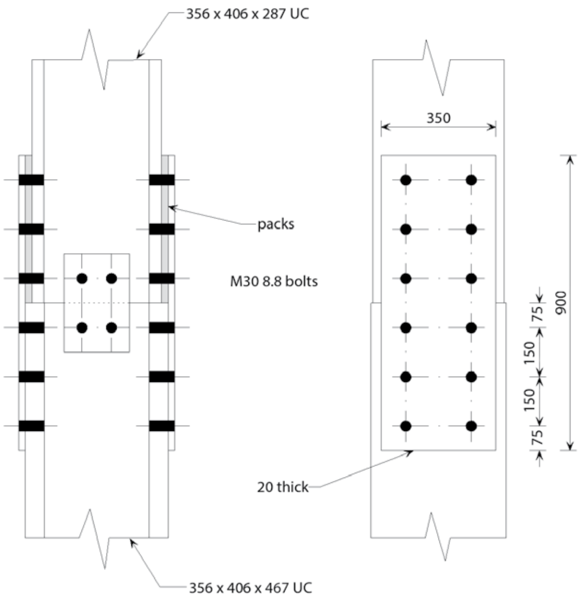

Figure 1: Splice detail

Figure 1, the bearing resistance of the bolts in the flange plates is 427 kN for the end bolts and 564 kN for the inner bolts: much higher than the shear resistance. The flange of the upper UC is 36.5 mm thick, so not critical.

In the 22.6 mm web of the UC, the bearing resistances for end and inner bolts are 483 kN and 637 kN respectively.

Noting the provisions of clause 3.7 and assuming that all the bolts behave as part of the same group, the resistance of the entire group is controlled by the lowest resistance – the shear resistance of the bolts in the flange of the upper section.

The resistance of the bolt group is therefore:

$F_{Rd} = 14 \times 191 = 2674 > 2625$ kN

Tying

The design tie force is the reaction from the largest loaded floor supported by the column. For this example, the area supported is 233 m^2 and the accidental tie force is given by:

$N_{Ed} = A(G+\psi Q)$

where Q is the characteristic variable action. The value of ψ is given in the UK National Annex to BS EN 1990:2002[8] as 0.5 for office areas. The value of N_{Ed} is therefore 1421 kN. This is less than the resistance of the bolt group. The tension resistance of the net area of the flange plates is:

$$N_{u,Rd} = \frac{0.9 \times (14000 - 4\times33\times20) \times 470 \times 10^{-3}}{1.1} = 4369 > 1421 \text{ kN}$$

Permitted deviations

Permitted deviations are not explicitly considered in design but their effect can be compared with the moment due to the amplified bow. At the splice, an angular misalignment of 1 in 500 results in a lateral displacement of 2.4 mm. The deviation in straightness between storeys results in a displacement of 1.6 mm. The maximum non-intended eccentricity is 5 mm. The amplified bow at the splice position is about 10 mm in the minor axis direction and about 4.7 mm in the major axis direction so the effects of the permitted deviations (apart from the non-intended eccentricity) is less than the amplified bow assumed in the column design.

Conclusions

The strut moment can be determined using the approach by which the column bow imperfection is back-calculated. The requirement to provide material to resist 25% of the compressive force at the splice will be enough in many cases to carry the vertical tie force in a Class 2B building. The permitted deviations are less than the implied imperfection for the critical buckling mode.

References

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