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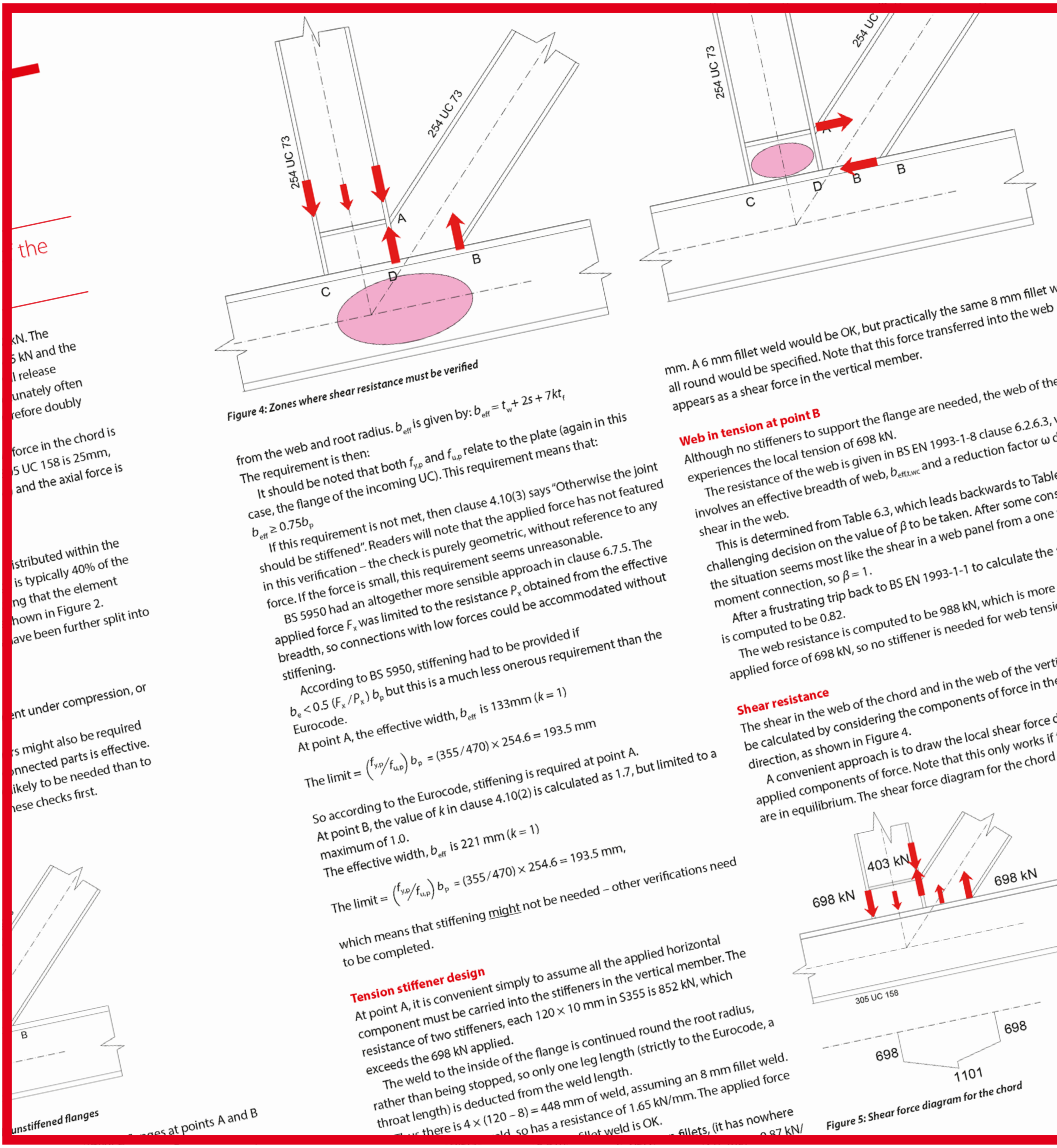

NSC
Technical Digest
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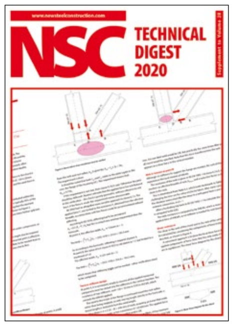

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Essential reading for designers

Nick Barrett - Editor

This is the fifth in the steel construction sector's annual series of Technical Digests of essential information culled from articles written by the sector's own technical experts and first published in the BCSA's monthly magazine New Steel Construction (NSC).

Launched after requests from readers that the technical content of NSC be brought together in an easily accessible format, the Digest has claimed a place on the essential reading section of the digital 'bookshelves' of architects and engineers. The Digests are always available for free download at the steelconstruction.info website.

The Digest is part of the steel construction sector's long-established commitment to providing everything needed to keep designers in steel up-to-date with the latest technical guidance to ensure that they can take advantage of the numerous benefits of steel as a sustainable construction material.

Design guidance and other key steel construction information is always easily accessible, either in print through NSC and technical supplements distributed through other specialist construction publications, or at steelconstruction.info, where everything relevant to steel construction, including cost as well as design guidance, is available on a free to use website, the first port of call for technical support.

NSC is a popular source of advice and news, and is where the highly regarded Advisory Desk Notes and longer Technical Articles are first published, and immediately made available on *newsteelconstruction.com*. The Digest brings together all the AD Notes and Technical Articles published in NSC in the previous year in a format that is available as a downloadable pdf or for online viewing.

AD Notes keep designers abreast of developments in technical standards. Some of them are provided following questions being asked of the sector's technical advisers. They are acknowledged as essential reading for all involved in the design of constructional steelwork.

The more detailed Technical Articles offer deeper insights into what designers need to know to produce the best steel construction projects. These articles can be in response to legislative changes or changes to codes and standards.

A technical update will occasionally be provided following a number of relatively minor changes that it is felt could usefully be brought together in one place.

Both AD Notes and Technical Articles provide early warnings to designers of changes that they need to know about and point towards sources of further detailed information available via the steel sector's other advisory routes. We hope you will continue to find the Technical Digests of value.

Joint stiffness calculation

The UK National Annex to BS EN 1993-1-8 discourages the use of numerical methods to calculate joint stiffness, relying on previous satisfactory practice. Despite this, interest in joint stiffness is increasing. Richard Henderson of the SCI illustrates the joint stiffness calculation process set out in the standard and discusses some of the issues.

Introduction

Traditionally, the UK has relied on successful past practice to classify orthodox connections – usually either nominally pinned or nominally rigid. The UK National Annex to BS EN 1993-1-8 endorses that approach and discourages the use of the numerical methods in the standard. The NA also indicates that frame design methods which utilise semi-continuous connection behaviour (the "wind-moment" method, for example) should not use a numerically calculated value, but the connection behaviour should be supported by test evidence or previous satisfactory performance.

Designers are paying increasing attention to connection stiffness, possibly because software is readily available which makes the calculation possible even for unorthodox arrangements. For a limited range of connections, BS EN 1993-1-8 presents a process to calculate the connection stiffness, utilising the same basic connection components which are used to calculate the moment resistance of the joint.

For designers not using software, this article demonstrates the numerical approach given in the standard. The example uses an existing connection design from P398[1], where the basic connection components are already established, shortening the process.

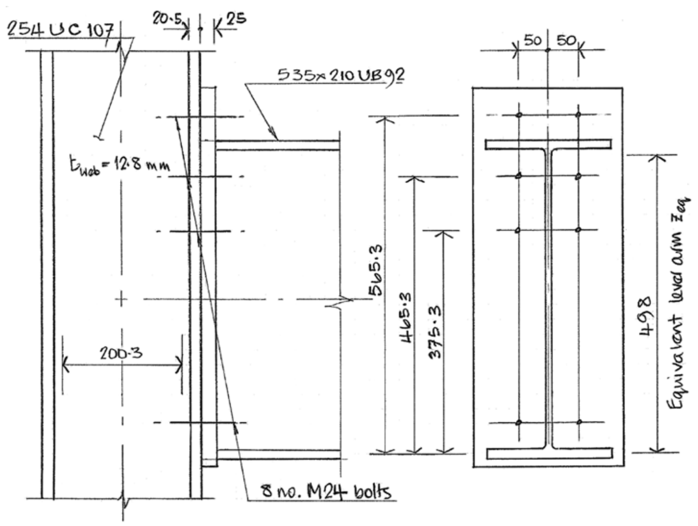

Figure 1: Joint arrangement

Numerical example

Example C2 from the Green Book for moment connections, SCI publication P398, has been used as a convenient bolted beam to column connection to illustrate the method of calculating joint stiffness. According to the UK National Annex to BS EN 1993-1-8, this joint is nominally rigid, simply because it has been designed in accordance with the Green Book.

The expression for the joint stiffness S_j is given in clause 6.3.1(4) as:

$$S_j = \frac{Ez^2}{\mu \Sigma_i \frac{1}{k_i}}$$

where: z is the lever arm defined in para 6.2.7 which depends on the type of joint and the arrangement of the bolts;
μ is the stiffness ratio defined in para 6.3.1(6);
k_i is the stiffness coefficient for basic joint component i.

The stiffness ratio, the ratio of the initial joint stiffness to the stiffness under load, is unity if the applied joint moment $M_{j,Ed}$ is less than 2/3 of the joint resistance $M_{j,Rd}$. For higher moments, the value of μ is given by:

$\mu = (1.5\ M_{j,Ed} / M_{j,Rd})^{\psi}$

The exponent ψ depends on the type of connection and is given in Table 6.8.

Example C2 in P398 is a bolted beam to column joint. The arrangement and member sizes are shown in Figure 1. The moment resistance of the joint is given as 416 kNm.

The relevant stiffness coefficients are identified in Table 6.10 of BS EN 1993-1-8 and for a single sided connection with two or more bolt rows in tension are listed as k_1, k_2 and k_{eq}. Para 6.3.3.1(4) indicates that the equivalent stiffness k_{eq} is based on k_3, k_4, k_5 and k_{10}. The joint components these stiffnesses refer to are given in Table 6.11 in the code and are listed in Table 1.

Stiffness coefficient	Component	Expression
k_1	Column web panel in shear	$0.38A_{VC}/\beta z$; (z_{eq} gives a more accurate value, see Fig 6.15)
k_2	Column web panel in compression	$0.7b_{eff,c,wc}t_{wc}/d_c$; ∞ if stiffened
k_3	Column web panel in tension	$0.7b_{eff,t,wc}t_{wc}/d_c$; ∞ if stiffened
k_4	Column flange bending	$0.9l_{eff}t_{fc}^3/m^3$
k_5	End-plate in bending	$0.9l_{eff}t_p^3/m^3$
k_{10}	Bolts in tension	$1.6A_s/L_b$

Table 1: Relevant stiffness coefficients

The quantities are defined in Table 2, taken from the example in P398.

Item	Description	Value
A_{VC}	shear area of column	3810 mm²
β	transformation parameter (Table 5.4)	1.0
z_{eq}	lever arm	498 mm
b_{eff}, l_{eff}	effective width or length	various
t	component thickness	12.8, 20.5, 25 mm
d_c	clear depth of web	200.3 mm
m	distance of bolt centre to root radius or weld toe	various
A_s	Tensile area of bolt	353 mm²
L_b	Bolt length	70.5 mm

Table 2: Values of parameters

The first challenge in calculating the stiffness components appears to be the determination of the equivalent lever arm for the column web stiffness coefficient k_1. However, the parameter depends on the effective stiffness for each bolt row r, and the height of the bolt row relative to the centre of compression of the beam flange so the calculation of the effective stiffnesses is in fact the real task. The effective stiffness for each bolt row must be calculated from the stiffness components k_i for that bolt row, given by:

$$k_{eff,j} = \frac{1}{\Sigma_i \frac{1}{k_{i,r}}}$$

The equivalent lever arm is given by:

$$z_{eq} = \frac{\Sigma_r k_{eff,r} h_r^2}{\Sigma_r k_{eff,r} h_r}$$

To complete the list of expressions for stiffness, the equivalent stiffness is given by:

$$k_{eq} = \frac{\Sigma_r k_{eff,r} h_r}{z_{eq}}$$

Using the data from Examples C1 and C2 in P398, the relevant effective widths of plate or lengths of T-stub can be determined. The value corresponds to the effective width or length which gives the lowest resistance for that component in the determination of the resistance of the joint. Where the lowest resistance is for several bolt rows acting as a group, the value for each bolt row is the total length divided by the number of bolt rows in the group, leading to the stiffnesses corresponding to each bolt row. The values are given in Table 3

Stiffness	minimum b_{eff}, l_{eff} (mm)	b_{eff}, l_{eff} (mm)	$k_{i,1}$	$k_{i,2}$	$k_{i,3}$
$k_{3,r}$	$r_1+r_2+r_3$	422/3	6.3	6.3	6.3
$k_{4,r}$	$r_1+r_2+r_3$	422/3	6.3	6.3	6.3
$k_{5,1}$	r_1	125	30.6	-	-
$k_{5,r}$	r_2+r_3	379/2	-	46.5	46.5
k_{10}	-	-	8.01	8.01	8.01
$k_{eff,r}$	-	-	2.10	2.15	2.15

Table 3: Stiffness values

As an example calculation for the first bolt row,

$$k_{eff,1} = \frac{1}{\frac{1}{6.3}+\frac{1}{6.3}+\frac{1}{30.6}+\frac{1}{8.01}} = 2.10$$

The heights of the bolt rows above the centre of compression are shown in Figure 1 and finally the value of z_{eq} can be determined. The value is:

$$z_{eq} = \frac{1.439 \times 10^6}{2994} = 498$$

The value for the equivalent stiffness is then:

$$k_{eq} = \frac{2994}{498} = 6.01$$

The remaining stiffnesses can also be calculated and the values are $k_1 = 2.91$ and $k_2 = \infty$ because of the presence of the compression stiffener.

The joint stiffness can now be calculated as follows:

$$S_j = \frac{210 \times (533.1 - 15.6)^2}{\mu\left(\frac{1}{2.91} + 0 + \frac{1}{6.01}\right)} = \frac{102}{\mu} \text{ MNm/radian}$$

The effect of the stiffness ratio μ is shown in Figure 2. For the bolted joint being considered, the value of ψ from Table 6.8 is 2.7. If the design bending moment is greater than two thirds of the bending resistance of the joint, the stiffness is reduced as indicated, to a value of about one third of the maximum stiffness when the applied moment approaches the joint resistance. It should be noted that UK practice is often to optimise the design, so a high utilisation might be expected.

Joint stiffness

Joint classification boundaries on the basis of stiffness are given in clause 5.2.2.5 and Figure 5.4 of BS EN 1993-1-8. The length of the beam and some understanding of the overall frame stiffness is needed, so some assumptions must be made. With reference to Figure 5.4, assuming a 9 m long beam and $k_b = 8$ (for frames with bracing), the requirement for the rigid classification is then $S_{j,ini} \geq k_b \, EI_b / L_b$.

Substituting values, $k_b \, EI_b / L_b = 8 \times 210000 \times 55200 \times 10^4 / 9000 = 1.03 \times 10^{11}$ or 103 MNm/radian which is greater than the stiffness calculated in section 2.0, unless $\mu = 1.0$. This assessment would therefore conclude that the joint can only be assumed to be rigid if the design moment is 2/3 of the bending resistance of the joint, or smaller. For unbraced "other frames" where the beams are at least 10 times as stiff as the columns, $k_b = 25$.

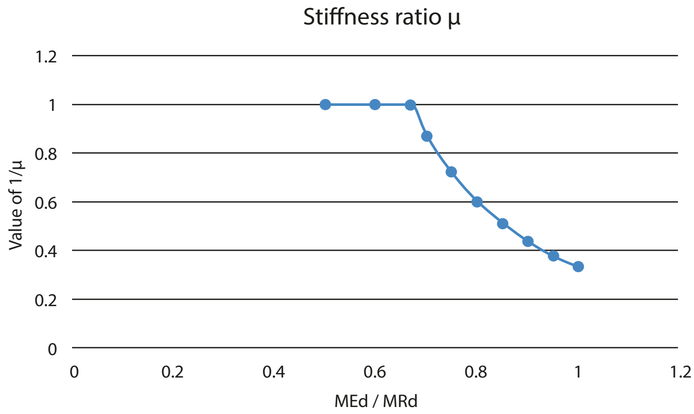

Figure 2: Stiffness ratio, μ

So for the rigid classification, the initial stiffness must be at least 322 MNm /radian so the joint would be classified as semi-rigid.

Effects of joint flexibility

BS EN 1993-1-1 clause 5.1.2(1) allows the analysis assumption of perfectly pinned or perfectly rigid, as long as the real joint behaviour does not have a 'significant' effect. As an illustration of the effects of the joint stiffness, the same beam was modelled using finite elements with rotational springs at the supports with stiffness equal to the maximum value calculated. The model is unrepresentative because no columns are included in the model. A 9 m span beam is assumed with a uniform load of 41.1 kN/m, giving a free bending moment of 416 kNm. The choice of load is arbitrary. From classical beam theory, a beam with encastré ends will have a support moment of 2/3 of the free bending moment ie 277 kNm and a mid-span moment of 139 kNm. The mid-span deflection will be 1/5 of the simply supported deflection, calculated to be 30.5 mm due to bending alone (no shear deflection). In a braced frame the joint detailed above can be classified as rigid when carrying a design bending moment of 277 kNm or less.

The FE analysis results give a support moment of 130 kNm and a mid-span moment of 286 kNm, with a maximum deflection (including shear deflection) of 20.9 mm. The support moment is about 47% of the encastré value and the deflection 3.4 times the encastré value. The introduction into an analysis model of joint stiffnesses calculated using BS EN 1993-1-8, although classified as "rigid" clearly has a profound effect on the behaviour of the structure and a decision to adopt a structural scheme that relied on frame stiffness and bolted beam to column joints would need to be considered carefully. The "wind-moment" method was shown to be adequate by frame analysis incorporating connection stiffness demonstrated by test, thus meeting the requirements of the UK National Annex.

Traditional approaches to unbraced frame deflection calculations have assumed that joints are rigid and deformation of the members is the source of overall building deflections, unless joints between members are of significant size relative to the member lengths. Such assumptions may need to be reconsidered for certain structures.

Conclusions

If joint stiffness is to be considered at all:

1) The manual calculation of stiffness is very laborious and it would be unrealistic to try to design a real structure in this way. Design software to calculate the joint stiffness is essential for projects of any significant size.
2) The sequence of design and sizing is likely to be iterative because the joint arrangements could affect both the serviceability and strength limit states.
3) Flexibility of bolted end-plate joints in beam to column connections in unbraced frame structures could have significant effects on the stability of the structure.

[1] Joints in steel construction: Moment-resisting joints to Eurocode 3

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 $s \le h/2$ *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.

$$\text{For 5 storeys,}\ \alpha_n = 1.1 - \frac{n}{10} = 1.1 - \frac{5}{10} = 0.6$$

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.

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 (over page).

Effect of bending moment

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

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

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:

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

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 ($16 \le t_f \le 40$)
Imperfection factor, α for rolled section with $h/b \le 1.2$, $t_f \le 100$ mm	0.49 (minor axis) 0.34 (major axis)

Table 2: Design parameters

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

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

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

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 $\sin(\pi \times (1.2/4.0)) = 0.81$. The design minor axis bending moment at the splice due to strut action is therefore:

$M_{z,Ed} = 0.81 \times 0.01 \times 10{,}500 = 85.1$ 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.

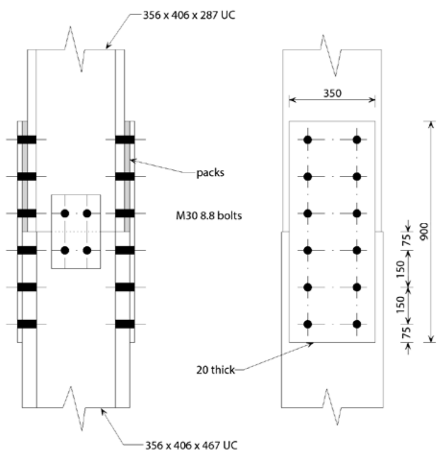

Figure 1: Splice detail

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

$215 \times 0.89 = 191$ kN

With the particular geometry of the bolt groups shown in 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 m² 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:

$$N_{u,Rd} = \frac{0.9 \times (14000 - 4{\times}33{\times}20) \times 470 \times 10^{-3}}{1.1} = 4369 > 1421 \text{ kN}$$

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.

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Offsite solutions

David Brown of the SCI reports on a recently completed research project, with some suggestions to increase still further the offsite content of steel-framed multi-storey buildings.

Offsite modular steelwork

Many designers would immediately comment that fabricated steelwork is already an offsite solution, produced in factory conditions – so what further is needed? This was the question for a research project funded by Innovate UK, led by BCSA, involving SCI, WSP, Severfield and Trimble, started in 2019. Before BREXIT and COVID-19, the UK Government had identified increased construction efficiency as a priority. Many will have heard of the so-called 'platform' approach to design for manufacture and assembly (P-DfMA) and possibly seen early examples of 'kit of parts' solutions intended to be used across a wide range of structures. The UK treasury are on record as suggesting that this approach can boost productivity whist reducing waste by up to 90%. The time was right to consider solutions that might meet this ambition.

The project was short – and was completed in February 2020. BCSA and SCI members will have received the two project deliverables – a short guide for building clients and a longer guide aimed at building designers. Both may be freely downloaded from *steelbiz* or *steelconstruction.info*

This article presents some of the project outcomes, hopefully as ideas to consider and develop in detail as required.

Project objectives

One of the initial objectives was to investigate the opportunities to integrate services into the steel frame, taking lessons from the light gauge modular industry, where this is normal practice. Repeatable units such as student accommodation or hospital wards (Figure 1) may be prepared as 'plug and play' units with most services pre-installed – so what can be done with multi-storey buildings? Opportunities exist, as the Latham report of 1994 identified: *The contributions of … M&E contractors and consultant to the construction industry is immense. The more complex the building, the higher is the likely value of the M&E input…*

Figure 1 - Typical modular hospital ward (from mtxcontracts.co.uk)

Initial enthusiasm for increased integration was dampened by a series of (current) militating factors:

- Often, the M&E design is executed by the contractor and therefore commences relatively late in the programme. By this stage the structural design is mature and opportunities for integration are limited;
- Detailed M&E design is undertaken by the contractor, so the scheme design must accommodate alternative solutions;
- Currently, M&E contractors may offer a lower price for a solution that does not require an offsite assembly facility.

If the benefits of prefabrication and preassembly of services are to be realised, the key principles are:

- An early decision that the services will be prefabricated;
- A design which is specific to offsite manufacture;
- An overall programme which delivers timely information.

Structural solutions

The project also considered structural solutions involving increased offsite fabrication and assembly, which offer benefits to the end client. It should be recognised immediately that the 'benefit' may not be in reduced initial cost – in a competitive environment, one would image that initial cost has been driven down already. Instead, the benefits arise from:

- reduced site construction periods;
- reduced waste;
- fewer site deliveries and less disruption;
- increased precision;
- earlier access for following trades;
- in some cases, more lettable floor area;
- a more readily demountable structure;
- in some cases, reduced foundations, saving cost and time.

The project team recognise that the solutions described below (and in more detail in the guides) have the status of 'proof of concept' rather than a ready-made solution. It is anticipated that solutions need finessing and modification to suit individual requirements and company manufacturing processes. Similarly, some of the solutions demand changes in responsibility compared to today's construction processes, and probably revised commercial arrangements. As an example, casting large composite floor panels offsite would demand a change in responsibility, as a minimum.

Steel composite cores

Many multi-storey buildings are stabilised by a concrete core, slip formed or jump formed. Conventional wisdom is that this is the most cost-effective and appropriate solution. A steel composite core is certainly seen as more expensive in terms of initial cost. However, a steel composite core has thinner walls (so more lettable floor area), is lighter, so has the advantage of reduced foundations, and overcomes the common tolerance and connection issues at the interface between the concrete core and the surrounding steelwork. Before dismissing the solution, designers may like to review the news stories, videos and other resources relating to the Rainier Square building, Seattle, shown in Figure 2 (over page).

The 58-storey Rainier Square building is in the news because it is stabilised by a steel composite core, consisting of panels fabricated from two steel plates held apart by bars and subsequently filled with concrete. The headlines describe this as 'radical', a 'game-changer' and having a 'revolutionary core'. The building was topped out after only 10 months, 8 months faster than the program for a conventional core, and reportedly with a 2% cost reduction (although detail on precisely which cost, or what has been valued, is not clear). What is clear is that this solution has generated some interest, with AISC (the American version of SCI) promoting the concept as 'SpeedCore', the name emphasising a key benefit of the solution.

UK designers may have a sense of deja vu, since an identical concept was being used in 2005, when it was known as 'Corefast'. Back then, the system

Figure 2 – Rainier Square (Photo: NBB/Sean Airhart)

used 'Bi-steel' panels, which separated the steel plates with a bar friction welded to both plates simultaneously. Only one manufacturer produced these panels. Now, panels may be produced by bolting between plates, or by welding. In both cases, the connecting bar protrudes through the plate. From 2005 to 2009 there was limited use of the 'Corefast' system, when there were challenges with a conventional core, or (for example) when it was advantageous to erect a tower crane on the steel core. Case studies from the time indicate that the construction period was significantly shorter than a conventional concrete core.

Figure 3 – Bi-steel core with connections for the surrounding steelwork

In addition to the claimed time saving, additional benefits arise at the interfaces with surrounding floor steelwork. Core units and steelwork is erected by the same organisation, to the same tolerance – and brackets, plates and supports may be attached to the core steelwork to facilitate on-site connections as shown in Figure 3.

Steel composite cores may not be a panacea for every structure – but the concept deserves consideration.

Dry floor plates

The project considered the possibilities of completing floor panels offsite and erecting completed floors. This approach is common in light gauge construction – so could a similar concept be used in orthodox multi-storey buildings? The concept investigated comprised panels up to 12 m long and 2.4 m wide – three panels would therefore result is a column grid of 12 m × 7.2 m.

The floorplate itself could be orthodox composite construction, or cross laminated timber (CLT). The latter is immediately seen as more expensive, but it has obvious 'green' credentials, can be manufactured to precise tolerances and is readily demountable. A CLT panel 135 mm deep could span a panel width of 2.7 m, under a variable action of 3.5 kN/m², so the solution is relatively shallow.

Whether the floorplate is CLT or conventional deck and concrete, the longitudinal and transverse junctions between panels demand special attention. Adjacent panels would need to be inter-connected, so that the floorplate forms a diaphragm, meaning that edge members such as channels which can be bolted back-to-back are a possible solution. The fire stopping and acoustic barriers at joints between panels would also need careful consideration – which are all possible if carefully engineered, in addition to the normal considerations of deflection and dynamic performance.

Conventional composite construction is hard to beat for a shallow, economic solution, so the benefits of dry floor panels, such as speed and early access for following trades would need to be valued to make such a solution worthy of further development.

Single storey columns

A P-DfMA approach favours simple components suited for several applications – so single storey columns are worthy of consideration, especially if associated with the prefabricated dry floor panels described previously. Single storey columns are easier to handle robotically, so may be an attractive solution for fabrication. Composite columns (with concrete surrounding open sections, or concrete within hollow sections) are currently not common in the UK, but have clear advantages at both ambient temperature and in fire. With some changes in the supply chain, manufacturing single storey composite columns appears entirely feasible.

Where next?

More details of the solutions outlined above are available in the guides freely available from BSCA and SCI. The project team never imagined that solutions would be adopted immediately without further development. The objective was that solutions which deliver benefits – which would have to be valued against conventional construction – should be considered in the mix. It is said that the Rainier Square building would not have progressed without the shortening of the construction program resulting from the use of a steel composite core. Those responsible intend to use the system for several other structures. Encouragement, perhaps, for the designers in the UK to consider the possibilities for their own projects.

Joint stiffness and the elastic critical load factor

The susceptibility of moment-resisting frames to global buckling is profoundly influenced by the stiffness of joints as calculated by the proposed method in BS EN 1993-1-8. Richard Henderson of the SCI illustrates the potential effects.

1 Introduction

It is unfortunate that BS EN 1993-1-1 does not adopt a succinct label for such an important parameter as α_{cr}, the authors instead choosing the descriptor "factor by which the design loads would have to be increased to cause elastic instability in a global mode". The BS 5950 label in the title above has much greater utility. The article on the calculation of joint stiffness in the February edition of *New Steel Construction* hinted at the effect on stability of the stiffness of bolted joints and the present article provides an illustrative example.

2 Example portal frame

2.1 Rigid joints

The structure used in this simple example is a pinned-foot portal frame with a horizontal rafter. Sufficient restraints are assumed to be provided to prevent out of plane and lateral torsional buckling. The frame has a span L of 30 m and a height h to the centre-line of the rafter of 15 m. The rafter is subject to a uniform load of 10 kN/m. In order to achieve a high elastic critical load factor, stiff UB columns have been adopted, consisting of 914 × 305 UB 224 rolled sections. The rafter is a 533 × 210 UB 101. Hand analysis has been carried out for amusement and checked by stick FE analysis, first assuming the joints are infinitely stiff.

2.2 Frame deflections

For the vertical load case, determining the bending moments by moment distribution requires the stiffness coefficients for the members at the joint. Assuming symmetry, these are $k_c = 3EI_c/h$ for the column and $k_b = 2EI_b/L$ for the rafter. The distribution coefficient for the column is given by $k_c/(k_b + k_c)$. No redistribution is required and the results are obtained directly as shown in Table 2.1.

Element	I value (m⁴)	Stiffness (kNm/rad)	Distribution coefficient	FEM (kNm)	Bending moment (kNm)
Column	3.76e-3	157920	0.9483	-	+711.2
Beam	6.15e-4	8610	0.0517	-750	-711.2

Table 2.1 Vertical load case: bending moments

The free bending moment in the rafter is 1125 kNm giving a mid-span moment of 413.8 kNm. The bending resistance of the rafter cross section given in the Blue Book is 901 kNm. The mid-span deflection of the rafter is given by the difference between the simply supported deflection and the upward deflection due to the end moments:

$$\delta = \left(\frac{5wL^4}{384EI_b} - \frac{M_0L^2}{8EI_b}\right) = 0.197\text{m}$$

The lateral deflection of the frame from a horizontal load at rafter level can be found using the slope-deflection equations and is given by:

$$\delta = \frac{Hh^2}{EI_bI_c}(LI_c + 2hI_b)$$

Assuming a unit load of H = 100 kN, substituting values gives a horizontal deflection at rafter level of 0.507 m.

2.3 Elastic critical load factor α_{cr}

Using the formula in para. 5.2.1(4) of BS EN 1993-1-1,

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right)$$

the α_{cr} value for the frame can be calculated. The global stiffness of the frame (H/δ) is 100/0.507 = 197 kN/m. Substituting the remaining values gives α_{cr} = 9.9. According to para. 5.2.1(3) of EC3, the frame is therefore almost stiff enough for second order effects to be ignored. Increasing the rafter by one serial size would achieve this, with α_{cr} = 10.6

2.4 Introducing joint flexibility

According to Para 5.1.2 of BS EN 1993-1-8, for elastic global analysis, joints should be classified according to their rotational stiffness. If the joint is semi-rigid, the rotational stiffness S_j corresponding to the design bending moment should be used in the analysis. A reasonable idea of the joint stiffness is therefore required to model the structure. The joint must be classified according to BS EN 1993-1-8 para. 5.2.2 and the initial rotational stiffness is denoted $S_{j,ini}$. The joint is deemed semi-rigid if:
$0.5EI_b / L \le S_{j,ini} \le 25EI_b / L$ or if $K_b / K_c < 0.1$

For the purpose of this example, the rotational stiffness of the beam to column joint has been assumed to have the same value as that calculated in February's technical article on the calculation of joint stiffness. The beam in that case was also a 533 deep UB and the joint stiffness calculated: $S_{j,ini} = k_{\theta B}$ = 100 MNm/radian. Using Table 2.1, K_b / K_c = 0.082 but $25EI_b / L$ = 107 MNm/radian so the joint is semi-rigid.

Joint flexibility increases the lateral deflection of the frame because, in addition to the rotation of the intersection of the members due to their curvature, the joints themselves rotate. The effect of this joint flexibility on the lateral deflection can be determined by assuming the joints behave as rotational springs and the members are rigid.

A similar approach to the slope-deflection equations results in the following formula for lateral deflection due to flexible joints:

$$\delta = \frac{-Hh^2}{(k_{\theta A} + k_{\theta B})}$$

Here H is the shear force in the element in kN and the k_θ parameters are the rotational stiffnesses in kNm/radian at ends A and B of the column of length h m and the deflection is in metres. This deflection is added to the deflection due to element flexure already calculated. The foot of the column is pinned so the rotational stiffness at this end is zero. Substituting the values H = 100 kN and h = 15 m gives δ = 0.113 m and a total lateral deflection of 0.620 m. The revised global stiffness of the frame is 161 kN/m and the elastic critical load factor reduces to 8.07 - second-order effects must therefore be considered.

The effect of the joint stiffness on the moments and deflections due to vertical loads can be calculated by considering rotations at the joint. The slope in the rafter is equal to the simply-supported value reduced by the slope due to the end moment. This rotation is equal to the slope in the column plus the rotation due to the flexibility of the joint.

$$\frac{wL^2}{24EI_b} - \frac{ML}{2EI_b} = \frac{Mh}{3EI_c} - \frac{M}{k_\theta}$$

The value of M is 657.4 kNm, a reduction of 53.7 kNm. The mid-span moment increases to 467.5 kNm and the mid-span deflection to 0.244 m.

The results can be confirmed by FE analysis, assuming elements have infinite shear stiffness.

2.5 Effect of connection design resistance

According to BS EN 1993-1-8 para. 6.3.1, the rotational stiffness of a beam-to-column joint $S_{j,ini}$ is reduced by a factor μ that depends on the joint utilization. If the design resistance is at least 1.5 times the design bending moment, the initial stiffness of the joint can be used in the analysis and $\mu = 1.0$. If the resistance ratio is less than 1.5 times, plastic deformation is assumed and a reduced stiffness must be used.

$\mu = (1.5M_{j,Ed} / M_{j,Rd})^{\psi}$ where $\psi = 2.7$ for a bolted end plate.

The effect of the resistance ratio on the bending moments is shown in Figure 2.1

The support (hogging) moment reduces as the margin of resistance of the

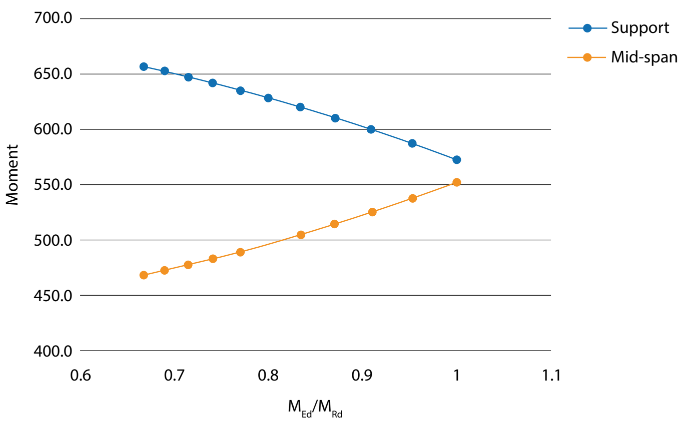

Figure 2.1 Beam Moments

joint reduces. When the joint resistance equals the design moment, the support moment has reduced from 711 kNm, the value found from classical analysis, to 572 kNm. The mid-span (sagging) moment increases correspondingly.

The effect on the elastic critical load factor is shown in Figure 2.2. The reduction in α_{cr} with reducing joint over-design is almost linear, from 8.07 to 5.93. This reduced value would require the design lateral loads to be increased by 20% to allow for second-order effects. Increasing the rafter by one serial size gives $\alpha_{cr} = 6.2$ and a lateral load increase of 19%.

An unverified estimate of initial stiffness for the specific elements in the example found a value of about 60 MNm/radian for a joint with a moment resistance of 622 kNm. This gives $\alpha_{cr} = 6.6$ and a support moment of 603 kNm. The value of M_{Ed}/M_{Rd} is therefore 0.97 and $\mu = 2.6$ approximately.

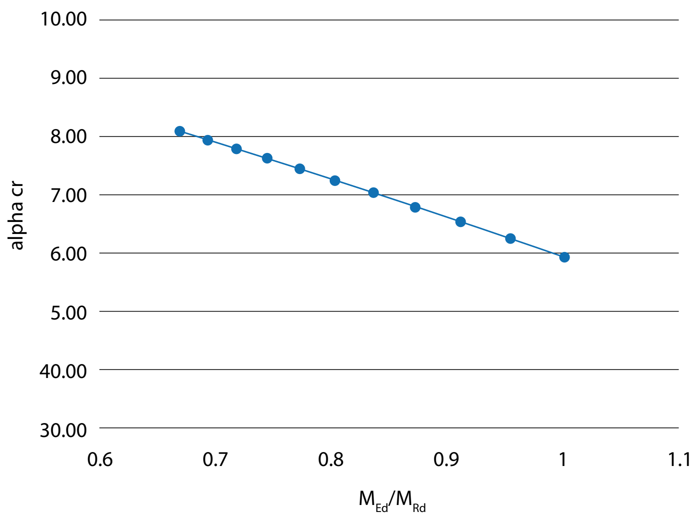

Figure 2.2 Elastic critical load factor

This value of μ corresponds to a lower joint stiffness which reduces the support moment to about 508 kNm. Iteration indicates a joint stiffness of about 29 MNm/radian giving a support moment of 538 kNm and $\mu \approx 2.05$. The corresponding value of α_{cr} is about 5.3.

3 Conclusions

The above example illustrates the effect of joint stiffness on frame behaviour, in terms of the design bending moments, the deflections and the global stability and second-order effects. The sequencing of analysis and design steps is also affected as the designer must either have a preliminary idea of joint details when setting up the analysis model or iteration will be necessary.

The presence of the resistance ratio μ in the stiffness calculation potentially introduces difficulties where the frame and joints are designed by different parties. The designer could specify design moments 50% larger than those determined in the analysis, in the hope of the joint remaining elastic. The steelwork contractor could well find it challenging and expensive to satisfy such a requirement.

The UK National Annex to BS EN 1993-1-8:2005 states in clause NA.2.6 that connections designed in accordance with the principles given in the SCI publication P207[1] may be classified on the basis of the guidance given in section 2.5 of the same publication. SCI publication P398[2], the successor to P207 contains the advice that well-proportioned connections that follow the recommendations for standardisation given in P398 and designed for strength alone can generally be assumed to be rigid for joints in braced frames and single-storey portal frames.

1 SCI P207, Joints in Steel Construction – Moment Connections

2 SCI P398 Joints in Steel Construction – Moment Resisting Joints to Eurocode 3

In-plane stability of portal frames

David Brown of the Steel Construction Institute offers a reminder of the guidance covering in-plane stability of this very common form of construction. Judging by recent questions received by the SCI, the topic is not as clearly understood as it should be!

The problem(s) identified

In March and April of 2020, to offer some light relief during COVID-19 lockdown, the SCI ran a number of free webinars covering the analysis, member verification and detailing of portal frames. The webinars were significantly over-subscribed, and were repeated four times. Two topics gave rise to the most questions – in-plane stability and restraints to the inside flange. Many delegates wanted to know what in-plane effective length should be used when verifying members – particularly the column. Others wanted to apply Annex E of BS 5950 to determine an in-plane effective length. There was no problem with out-of-plane lengths – no-one questioned that out-of-plane, members should be verified between restrained positions.

Perhaps the problem is highlighted if designers are using general elastic analysis software to determine the design forces and moments around the frame and then to verify the members within it. Such software expects to complete both in-plane checks and out-of-plane checks, which naturally demands an in-plane buckling length. Portal frames are a special case, with particular rules discussed in this article.

What does BS 5950 say?

Before opening the Eurocode, it is valuable to look at the particular rules for portal frames given in BS 5950. The UK would claim to have developed most of the rules for portal frame design, backed up by many decades of successful application, so one might expect definitive guidance in our previous standard.

Portal frames are one example of a continuous frame, and may be designed elastically or plastically, so we need to look carefully at the relevant clauses.

Within the "Continuous structures" section, clause 5.2.3.1 discusses plastic analysis. The second paragraph should be sufficient to clarify the in-plane verifications needed:

> *The in-plane stability of the members in a continuous frame designed using plastic analysis should be established by checking the in-plane stability of the frame itself, see 5.5.4.*

Designers should note that according to this clause in-plane checks of individual members are not required.

Portal frames are addressed in section 5.5. Clause 5.5.2 covers elastic design:

> *If elastic global analysis is used for a portal frame, the cross-section capacity should be checked… and the out-of-plane buckling resistance should be checked….*
>
> For portal frames with no in-plane bracing…*the in-plane stability of the frame should be verified by checking the cross-section capacity and the out-of-plane buckling resistance of the members* (amplified if necessary)

Plastic design is covered in clause 5.5.3:

> *Plastic global analysis may be used for a portal frame provided that the conditions in 5.2.3 are satisfied* (which is a reference back to the clause previously quoted).

Checking the in-plane buckling of individual members in a portal frame is inappropriate – the frame buckles as a single entity, and therefore the standard demands that stability is verified by checking "the in-plane stability of the frame itself".

Multi-span frames

One potential exception to the preceding general rule is an internal column in a multi-span frame (Figure 1). In the so-called gravity combination, the bending moment in the internal column may be very small. The in-plane buckling of this member should be checked. P292 recommends an effective length factor of 1.0 for truly pinned bases, 0.85 for nominally pinned bases and 0.7 for nominally fixed bases.

Internal columns probably have no restraint at any level below the haunch. If the internal column is orientated in the orthodox direction (major axis in the plane of the frame) then the minor axis resistance will of course be critical, not the in-plane buckling. If the internal column was turned 90°, such that its weak axis was in the plane of the frame (Figure 2), or if the internal column was a fabricated section with a larger inertia out of the plane of the frame, then in-plane buckling could be critical, but it seems most unlikely.

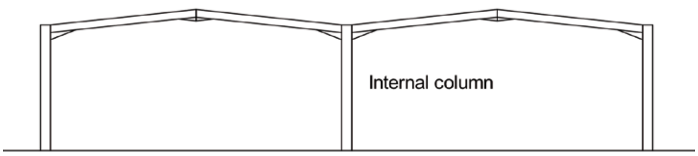

Figure 1; Multi-span portal with internal column

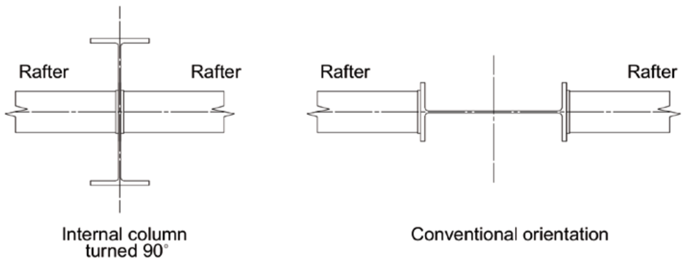

Figure 2; Orientation of internal column

In-plane buckling of the frame

According to BS 5950, in-plane stability of portal frames can be verified by three methods:

1. The sway-check method – commonly known as the *h*/1000 check, with a limited scope (and a snap-through check for multi-span frames);
2. The amplified moments method, requiring the determination of λ_{cr} and an amplifier if necessary. No amplifier is required if $\lambda_{cr} > 10$;
3. Second-order analysis.

In each method, the impact of second-order effects is considered. Satisfying the sway-check method means that second-order effects are small enough to be ignored. The amplified moments method allows for second-order effects with an amplifier unless the effects are small enough to be ignored. Second-order analysis will always allow for those effects.

Member checks in BS 5950

Having completed the in-plane buckling checks of the frame in its entirety and allowing for second-order effects if necessary, the cross section has to be checked and then out-of-plane checks completed. BS 5950 has a range of clauses covering different conditions – next to plastic hinges, with intermediate restraints to the tension flange only, tapered sections etc.

Why not Annex E?

The introduction to the Annex seems to offer opportunities for use, describing "the effective length L_E for in-plane buckling of a column or other compression member in a continuous structure with moment resisting joints should be determined using the methods given in this annex." That sounds appropriate for portals, but as one reads further, it becomes abundantly clear that this annex is limited to columns in rectilinear multi-storey frames. The annex describes columns in multi-storey beam-and-column framed buildings with …. concrete or composite floor and roof slabs. Hardly the description of a portal frame!

Eurocode rules

One would not expect the fundamental physics to change simply because the Eurocode was introduced. On that basis alone, one should be confident that the same rules apply to orthodox portal frames – that in-plane, the stability of the entire frame as one unit is critical, followed by checks of the cross section and only out-of-plane buckling checks.

The key clause is 5.2.2(7)a in BS EN 1993-1-1:

> *If second order effects in individual members and relevant member imperfections are totally accounted for in the global analysis of the structure, no individual stability check for the members according to 6.3 is necessary.*

In-plane second order effects are allowed for by determining α_{cr} (directly equivalent to λ_{cr} in BS 5950), and using an amplifier in the global analysis if necessary. Frame imperfections are allowed for by always including the equivalent horizontal forces (EHF) in every combination. The only in-plane effects that are not included in the global analysis are the individual member imperfections, such as an initial lack of straightness. To consider the impact of in-plane member imperfections, colleagues at the SCI spent (very) many hours analysing a wide range of frames with and without in-plane member imperfections. Imperfections were modelled in both directions, in each member, to produce the most onerous effect. The study concluded that the value of α_{cr} changed less than 0.3%. Two conclusions can be made. Firstly that the effect of in-plane member imperfections on the stability of the frame is small enough to be ignored – or presented another way, we can say that all relevant in-plane effects have been allowed for in the global analysis. We therefore do not need an in-plane stability check of individual members. The second conclusion is that as expected, BS 5950 was correct – *"The in-plane stability of the members in a continuous frame …. should be established by checking the in-plane stability of the frame itself"*

The global analysis has not verified the out-of-plane resistance – members still must be verified between restraints, using section 6.3 of the Eurocode, aided perhaps by the guidance in Annex BB, which is simply the guidance from BS 5950 'translated' into Eurocode nomenclature.

Member verification in section 6.3 of BS EN 1993-1-1

If (and only if) the interaction factors in expressions 6.61 and 6.62 are taken from Annex B of the Eurocode (very strongly recommended by SCI), it can be concluded that expression 6.61 deals with in-plane effects and expression 6.62 deals with out-of-plane effects. Since we have concluded that no in-plane member checks are needed (other than the possible internal columns mentioned earlier), we can dispense with expression 6.61 altogether.

As there is no minor axis moment in a portal frame, expression 6.62 reduces to a rather simpler form:

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy}\frac{M_{y,Ed}}{M_{b,Rd}}$$

The numerators are the design force and major axis moment. The denominators are the minor axis flexural resistance and the lateral torsional buckling resistance, which with some judicious interpolation can generally be obtained from look-up tables if required. In all cases, the lateral torsional buckling resistance depends on the shape of the bending moment diagram over the length being considered, reflected in the value of the factor C_1. Resources are readily available to determine the C_1 factor for different shapes of bending moment diagram. The interaction factor k_{zy} is painful to compute, but in portal frames is generally around 0.97 – there is not much loss in manual calculations if k_{zy} is assumed to be 1.0.

Conclusions

Portal frames are special in many ways, despite their frequent use in the UK. They are slender, have significant axial forces in the members, generally are sensitive to second-order effects, experience reversing bending moments and demand very careful restraints to otherwise unrestrained flanges. The objective of this article was to confirm one special design feature – that in-plane buckling is an concern for the frame as a whole, not for individual members.

1 King, C, M.
In-plane stability of portal frames to BS 5950-1-2000 (P292)SCI, 2001

Restraints around portal frames

In this second technical article on portal frames, David Brown of the Steel Construction Institute reviews the all-important correct positioning and arrangement of restraints to the inside flanges of columns and rafters. Having considered in-plane buckling in the previous article, the focus is now on controlling out-of-plane buckling.

The problem(s) identified

Charles King, well-known to many in the portal frame world and responsible for much of the guidance on this popular form of construction, used to comment when leading SCI courses that some errors in the analysis and design of a portal frame may not lead to collapse, but incorrect detailing almost certainly would. It is clear from inspecting some bare frames during erection and from questions received at the SCI that some designers remain uncertain about where restraints should be located, and what form an effective restraint might take.

Fundamental Physics

The bending moment diagram around a portal frame due to primarily "gravity loads" is well known, shown in Figure 1. At various locations, notably the column and around the haunch in the rafter, the inside flange is in compression under this combination of actions. Elements in compression wish to buckle, and eventually, if unrestrained, will buckle in the out-of-plane direction. The moment is greatest at the eaves – consequently the compression in the inner flange is at a maximum, resulting in great enthusiasm to buckle out-of-plane – which must be restrained if the frame is to remain stable.

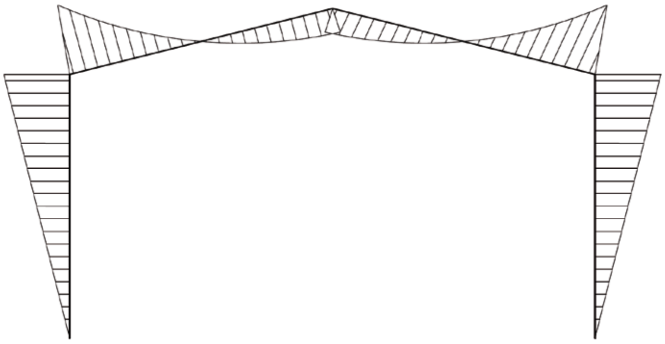

Figure 1: Being moment diagram – "gravity" combination of actions

The classic assumption about members is that they have "fork" supports, as shown in Figure 2.

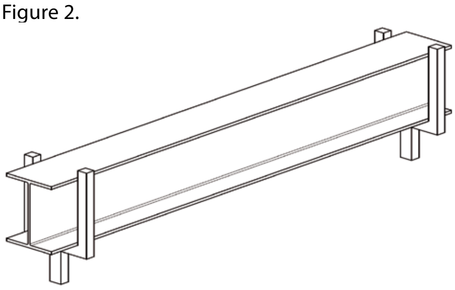

Figure 2: End fork supports – a torsional restraint

It should be noted that a "fork" support provides lateral positional restraint to each flange, thus forming a torsional restraint. It should be equally obvious that a restraint to one flange only, as shown in Figure 3a, is not providing a torsional restraint at that location.

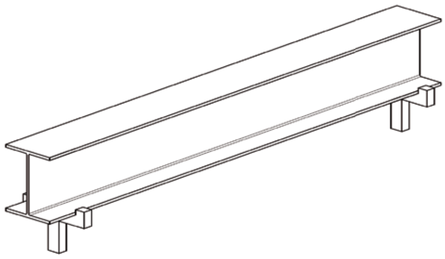

Figure 3a: Lateral restraint to one flange only

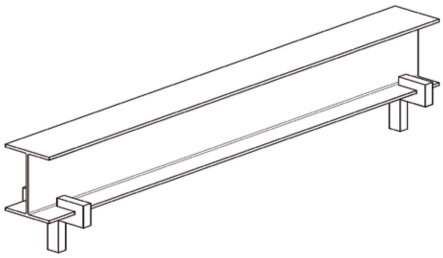

Figure 3b: Lateral and torsional restraint to one flange only

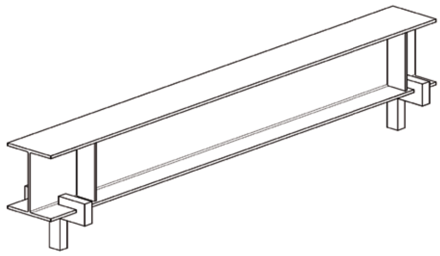

Figure 3c: Lateral and torsional restraint to one flange with web stiffeners

Some arrangement to "clamp" the one flange, as shown schematically in Figure 3b, is still not a torsional restraint, as the unattached flange is free to buckle. An arrangement with stiffeners to connect the flanges together, <u>and</u> a "clamped" flange, as shown schematically in Figure 3c is the only way to provide restraint to the "other" flange, but note the requirement for both stiffeners and a "clamped" flange.

These schematic diagrams illustrate the sorts of questions – and answers - which arise concerning restraints around portal frames. In summary:

1. A side rail or purlin connected to one flange only provides lateral restraint to that flange only, but does nothing of value for the other flange.
2. Introducing full depth stiffeners in isolation does nothing to prevent lateral-torsional buckling – the whole cross section is still able to move laterally and twist. In this situation the AISC (American equivalent of SCI) note that "transverse stiffeners are simply along for the ride"
3. Introducing stiffeners on their own, even when aligned with a side rail, does not constitute a torsional restraint, as the connection to the side rail or purlin is in no way equivalent to the "clamp" shown in Figure 3c. Bolts in clearance holes in very thin material cannot be considered to provide a rigid joint.

In the UK, the common way to restrain the inside flange is to provide small diagonal links from the inside flange to the side rail or purlin, as shown in Figure 4.

Conceptually, this triangulated system is equivalent to the web stiffeners

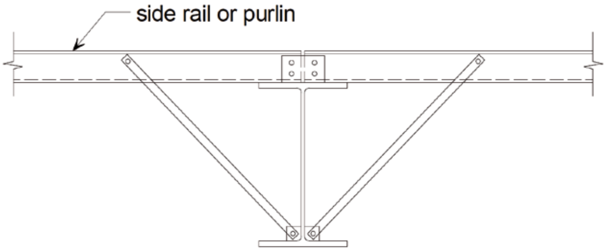

Figure 4: Stays from secondary steelwork to inner flange

shown in Figure 3c and the secondary steelwork provides lateral restraint. The necessary torsional restraint, equivalent to the "clamping" described above, is delivered by the stiffness of the secondary members acting as "U-frames" as shown in Figure 5.

U-frame action and its application to portal frames was discussed at length

Figure 5: U-frame behaviour with secondary steelwork

in *New Steel Construction* in June 2018.[1] This article included advice on when and how the stiffness of the secondary steelwork forming the U-frames should be assessed.

The small diagonal ties shown in Figure 4 are normally designed for a lateral force equal to 2.5% of the compression force in the flange, but their stiffness is equally important. If out-of-plane buckling is prevented in the first place, there is no lateral force. Since in the UK we believe that U-frame action is the underlying structural mechanics, we do not believe that the restraint forces translate into tension and compression in the side rails or purlins, nor do we insist that to be effective as part of a restraint system, the side rails and purlins must intersect with the nodes of bracing. Some other European countries make this a requirement. The secondary members must be continuous, otherwise there is no U-frame. Side rails interrupted by roller shutter doors, for example, are clearly not forming a "U" with the adjacent frame.

It is self-evident that a purlin or side rail must be located at the position where a restraint is needed, which means that judicious positioning of secondary steelwork is required, to suit both the cladding and the out-of-plane restraint to the members.

Figure 6 shows a frame during construction. The judicious spacing of purlins is evident – closer spacing in zones of high bending moment and more widely spaced purlins elsewhere.

Figure 6: Portal frame with thoughtfully spaced purlins

An alternative approach often used at the most heavily loaded location – where the underside of the haunch meets the column flange – is to position a member at this level, immediately adjacent to the inner flange. It is not adequate to simply tie all frames together at this point, as all the frames could buckle in the same out-of-plane direction. The members at this point must be triangulated back to the outside flange at some point, or connected to the foundation.

Where are restraints needed?

The short answer is wherever the member verification demands. Member verification demands a buckling length, which in the out-of-plane direction depends on the position of the restraints. It is surprisingly difficult to find this fundamental requirement in the Eurocode. Clause 6.3.3 which covers combined bending and axial compression and is therefore applicable to members in portal frames points out in Note 1 that "the interaction formulae are based on the modelling of simply supported single span members with end fork conditions….". As shown in Figure 2, end forks provide a torsional restraint.

During the recent SCI webinars on the design of portal frames, most discussion centred on the restraint where the underside of the haunch meets the column flange, generally referred to as "Point A". Horne and Ajmani, who were responsible in the 1970s for much of the research relating to portal frames which we see in BS 5950 and now repeated in the Eurocodes, described this important location as "Point A" and the description has remained ever since.

A number of designers were not convinced that a restraint was essential at "Point A". It can be inconvenient, because if the cladding is supported at the top of the column, the next side rail down could usually be far below "Point A" if support to the cladding was the only requirement. However, there is nowhere around the frame where the compressive force in the flange is higher, so nowhere more deserving of an effective restraint. A side rail positioned for that purpose (if that system is adopted) is not an expensive and unnecessary addition, but an essential contribution to prevent collapse.

Some designers suggested that with a restraint to the inner flange some way down the column, and another restraint some way along the haunch, the situation would be adequate. The SCI response is to ask which clause is being used to verify the member – which is partly tapered and includes a change of direction of usually 84°. There are no clauses that cover a member with a nearly right-angle kink within the length.

The second common question recognised that there is very often a compression stiffener in the column at "Point A", and suggested that this combined with a side rail would restrain the inner flange. However, as explained above, a connection in the very thin material of the side rail with ordinary bolts in clearance holes is hardly the "clamp" necessary for this system to be effective.

Designers using bespoke software for portal frame design should make sure they are entirely clear what type of restraint (one flange only, or torsional, demanding restraint to both flanges) they have modelled. "Point A" will invariably be modelled in software with a torsional restraint, which must be provided in the physical structure.

Figure 7 should serve as a dramatic warning. No restraint at "Point A" has simply allowed the point to buckle laterally. This should not be allowed to happen – and yet – it is sometimes possible to see buildings under construction without this point restrained. It is also possible to see structures where the restraints have been detailed and provided to the bottom flange of the rafter, rather than the bottom flange of the haunch. At the deep end of the haunch, we would expect the compression to be in the bottom flange of the haunch and this location should be restrained. The bottom flange of the rafter, being approximately on the neutral axis of the compound section, should have hardly any force at all.

Figure 7: The result when Point A is not restrained

Conclusions

The importance of restraints to the compression flange (the location of which will vary in different combinations of actions) cannot be over-emphasised. Such restraints are fundamental to the structural stability of the frame, and omission could lead to collapse. Restraints to the inner flange must be identified, specified and provided in the actual structure.

1 U-frames in bridges, *New Steel Construction*, June 2018

Shelf angle floor beams in fire

Mark Lawson, Consultant to The Steel Construction Institute, discusses the resistance of unprotected shelf angle floor beams at elevated temperatures.

Composite floor slabs used in light steel construction are often supported by steel beams that are partially encased in the concrete slab. These beams may be required for longer spans or where walls do not align at different levels. They usually have side angles to support the slab and are known as 'shelf angle floor beams' and provide at least 30 minutes inherent fire resistance. These beams may also be fire protected by a plasterboard ceiling, and by intumescent coating or box protection for longer periods of fire resistance.

Shallow shelf angle floor beams are often designed for serviceability criteria, which means the design moment at the fire limit state is a relatively low proportion of the bending resistance of the beam. In these situations, it may be possible to verify an unprotected solution for 60 minutes fire resistance by calculating the reduced resistance based on a temperature profile through the cross section. Tabulated temperatures from the standard or temperatures determined from a software analysis may be used to determine the temperature profile through the section.

A typical case of a shelf angle floor beam is shown in Figure 1, in which a 170 mm deep composite slab is supported on 150 × 90 × 10 mm thick angles welded or bolted to the sides of a 254 × 254 × 107 kg/m UC beam used to minimise the overall floor depth. The decking has crushed ends in this case and so provides a solid block of concrete next to the beam web.

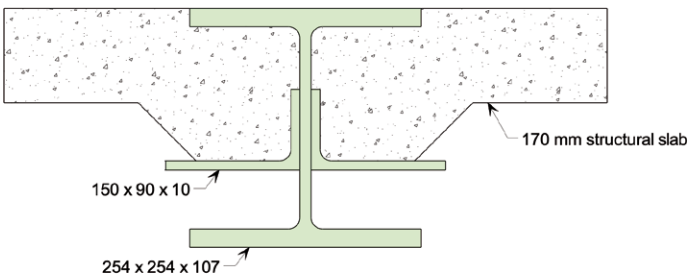

Figure 1: Cross-section through a 254 × 254 UC beam and 150 × 90 shelf angles supporting a composite slab with decking supplied with crushed ends

The fire resistance of shelf angle floor beams using UB or UC sections is given in Annex C to BS 5950-8[1]. Design guidance on the fire resistance of shelf angle floor beams in accordance with BS 5950-8 is provided in SCI publications P080[2] and P126[3]. This design process is considered to be satisfactory for Eurocode designs by taking the strength reduction factors (k_y,θ) from the Eurocode.

The verification involves calculating the reduced plastic moment resistance of the section, including the continuous shelf angles, at elevated temperatures. The plastic moment resistance uses the strength reductions of the various elements of the cross-section. The strength reduction factors in BS EN 1993-1-2[4] may be used to replace those given in BS 5950-8, (both standards are similar). The reduction factor for the design load at elevated temperatures is also given in BS EN 1993-1-2.

In light steel construction, the top of the steel section is generally cast level with the top of the slab and most of the steel section is encased by the slab and is therefore relatively cool so that its full tensile strength can be developed. The difference with respect to solid slabs is that the outer part of the angle is exposed between the ribs of the decking, which often is supplied with crushed ends for this application. Therefore, it is recommended that the contribution of the outer 50 mm of the angle should be taken as the same as for the exposed bottom flange. There is value in performing a thermal analysis by finite element modelling to be able to predict the precise temperature distribution for a particular configuration if it is to be used regularly.

Design of shelf angle floor beams in fire

The design of shelf angle floor beams in fire is presented in Annex C of BS 5950-8. Temperatures are defined for various segments of the cross-section (known as 'blocks'). These are given as:

θ_1 - Bottom flange
θ_2 - Exposed web of beam
θ_3 - Bottom leg of angle
θ_R - Angle root
θ_4 to θ_6 - Encased web of beam and vertical leg of angle
θ_7 - Top flange

These temperatures are presented in Table 1 as a function of B_e, which is the bottom flange width and D_e, which is the exposed depth of beam. The data for 30 and 60 minutes presented in Table 1 is reproduced from BS 5950-8 which also provides data for the 90 minutes case.

The temperature of the exposed bottom flange should be determined from Table 2 which is extracted from Table 10 of BS 5950-8 and is based on downstand beams supporting concrete slabs. This data is very conservative for shelf angle floor beams. Table 10 of BS 5950-8 only gives temperature data up to 60 minutes, for longer durations or for less conservative temperature data, thermal modelling may be carried out. SCI can perform this modelling.

Flange Thickness, t_f	Fire resistance 30 min	Fire resistance 60 min
10 mm	772	938
15 mm	736	933
20 mm	714	925
25mm	676	909
30 mm	638	886

Table 2: Temperature θ_1 (°C) of the exposed bottom flange for a beam supporting a concrete slab

Table 1: Block temperatures (°C) in a shelf angle floor beam as a function of bottom flange temperature at 30 and 60 minutes fire resistance

Aspect ratio of exposed depth: width of beam	Fire resistance 30 min			Fire resistance 60 min		
	θ_2	θ_3	θ_R	θ_2	θ_3	θ_R
$D_e/B_e \leq 0.6$	θ_1 - 140	475	350	θ_1 - 90	725	600
$0.6 < D_e/B_e \leq 0.8$	θ_1 - 90	510	385	θ_1 - 60	745	620
$0.8 < D_e/B_e \leq 1.1$	θ_1 - 45	550	425	θ_1 - 30	765	640
$1.1 < D_e/B_e \leq 1.5$	θ_1 - 25			θ_1		
$D_e/B_e > 1.5$	θ_1					

θ_1 = bottom flange temperature - see Table 2 and exposed depth, $D_e = h-h_c$

The temperature gradient in the web and vertical leg of the angle is given in Table C2 of BS 5950-8 as 2.3°C per mm for 30 minutes fire resistance, and 3.8°C per mm for 60 minutes fire resistance. Therefore, the depth of web with a temperature difference of 200°C over its depth is 53 mm for the 60 minute case. This may be approximated to 50 mm for analysis purposes.

The strength reduction factors for steel in Class 1 to 3 sections based on the effective yield strength at elevated temperatures are given in Table 3.1 of BS EN 1993-1-2 and are reproduced in Table 3.

Temperature (°C)	Strength reduction factor ($k_{y,\theta}$)
400	1.0
500	0.78
600	0.47
700	0.23
800	0.11
900	0.06
1000	0.04

Table 3: Strength reduction factor for steel for effective yield strength, $k_{y,\theta}$

The combined width of the two side angles should be wider than the beam in order provide a suitable bearing length of the slab. The temperature of the outer 50 mm of the exposed leg of the angle may be taken as the bottom flange temperature, θ_1 , and for analysis purposes, its average temperature may be taken as $(\theta_1 + \theta_3)/2$.

The reduced bending resistance of the embedded steel section may be determined as follows:

- The plastic neutral axis depth is determined by equating the reduced tension and compression forces based on the cross-sectional areas of these elements multiplied by their strength reduction factors - see Figure 2. It is generally found that the plastic neutral axis lies at or close to the top flange of the steel section.
- The bending resistance is determined by taking moments of the reduced resistance of each element multiplied by the distance from the neutral axis. This includes the steel section and shelf angles but not the concrete.
- The reduced bending resistance in fire is then given as a ratio of the bending resistance of the steel section in normal conditions ignoring the shelf angles.

Simplified design of shelf angle floors beams in fire

The load ratio that may be supported in fire conditions depends on the shape and depth of the steel section, the relative cross-sectional area of the shelf angles and the depth of concrete. Lighter UB sections will benefit more from the effect of the partial encasement than heavy UC sections. For an approximate design, Table 4 may be used to obtain the maximum load ratio that may be applied at the fire limit state depending on the steel section and fire resistance period. Where data is not presented in this table, such as for the 90 minute fire resistance case with low load ratios, the precise configuration may be analysed by thermal modelling. This is cost-effective if the same or similar details are used in a large project or in other projects.

	UB Sections		UC Sections
Fire resistance	$h/h_c < 1.6$	$h/h_c < 2$	$h/h_c < 1.6$
	$M_{Rd,fi}/M_{Rd}$	$M_{Rd,fi}/M_{Rd}$	$M_{Rd,fi}/M_{Rd}$
30 mins	≤ 0.65	≤ 0.50	≤ 0.60
60 mins	≤ 0.35	≤ 0.25	≤ 0.30
90 mins	≤ 0.20	Not presented	Not presented

Table 4: Maximum load ratios for unprotected partially encased UB and UC sections with side angles supporting composite slabs

In Table 4:

$M_{Rd,fi}$ is the reduced bending resistance of the partially encased section in fire conditions

M_{Rd} is the bending resistance of the UB or UC section in normal conditions

h is the beam depth and h_c is slab depth

It is also generally the case that the peak temperature at the top of the flange does not exceed the limit for insulation at 60 minutes fire exposure provided the web thickness does not exceed 18 mm or as substantiated by thermal modelling.

Where, the exposed part of the steel section is fire protected, the required fire protection of a shelf angle floor beam or a partially encased beam may be determined from the section factor of the exposed part of the section.

This method does not apply for RHS sections with side angles or a welded bottom plate (known as a slim floor beam) and in this case, the temperatures should be obtained by thermal modelling as the temperatures in the web of the RHS will be higher than for a UC section. Nevertheless, 60 minutes fire resistance can often be achieved in the case of an RHS slim floor beam and 30 minutes for an RHS section with shelf angles.

1 BS 5950-8:2003. *Structural use of steelwork in building – Part 8: Code of practice for fire resistant design*, BSI, 2003

2 *Fire resistant design of steel structures – A handbook to BS 5950: Part 8*, SCI, 1990, Lawson, R, M and Newman, G, M. (available on Steelbiz)

3 *The fire resistance of shelf angle floor beams to BS 5950: Part 8* Newman, G, M., SCI, 1993, (available on Steelbiz)

4 BS EN 1993-1-2:2005. *Eurocode 3: Design of steel structures – Part 1-2: General rules – Structural fire design*, BSI, 2006

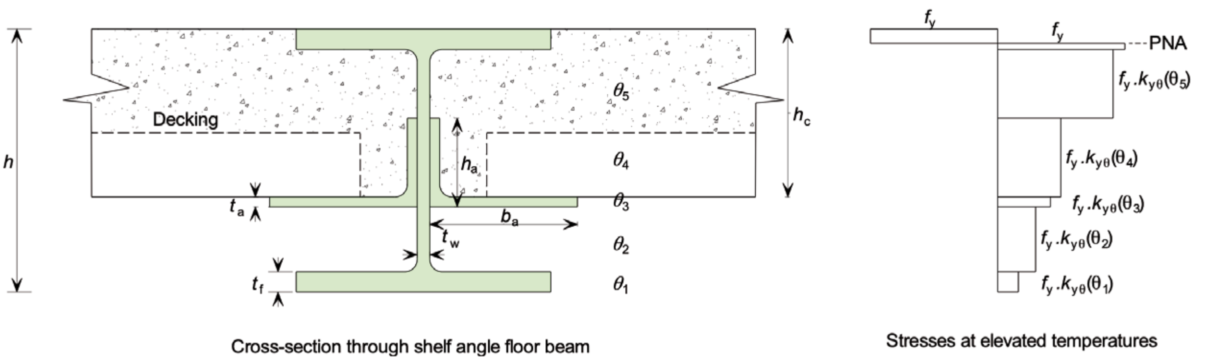

Figure 2: Plastic bending resistance of a shelf angle floor beam in fire using a UC section in this case

The development of design rules for restrained columns

Following on from the previous two articles, David Brown of the SCI looks back at the development of design rules for restrained columns. Looking at the work in the 1950s and 1970s reveals the background for many of the features found in today's design standards

The June and July/August articles on restraints around portal frames encouraged a closer look at the rules defining the resistance of restrained columns, making the link between the current rules in BS 5950 and BS EN 1993-1-1 and the salient technical papers published in the 1970s. The work on columns with restraints refers back to earlier rules covering unrestrained columns and unadopted recommendations to modify BS 449, the design standard of the time.

Early research

The most significant papers covering restrained columns are *Design of columns restrained by side-rails*[1] and *Failure of columns laterally supported on one flange*[2], both by Horne and Ajmani, published in 1971 and 1972 respectively, and the record of the associated discussion published in 1973[3]. The authors and contributors include a number of very well-known names in the steelwork world. Professor Horne OBE is a co-author of *Plastic design of Low-Rise Frames*[4] which used to be the definitive work in the UK on portal frame design and detailing.

A contributor to the discussion was Dr Morris, co-author of the aforementioned publication, and forever known by the shear stiffener which takes his name. Other contributors to the discussion include

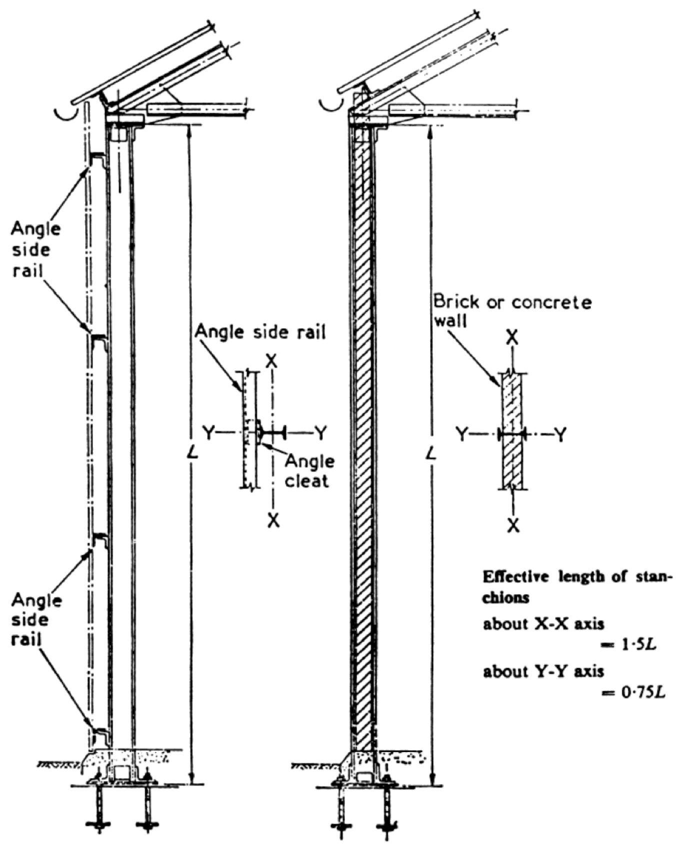

Figure 1: Effective lengths of stanchions according to BS 449 (Figure 15)

Dr Wood, known for the effective length curves found in Appendix E of BS 5950, Professor Nethercot, widely known for most things in steelwork and Mr Needham, who is known for his work with CONSTRADO, the forerunner to SCI.

Dr Ajmani was the Chief Design Engineer for the Tata Iron and Steel Company of Jamshedpur, India. Clearly Dr Ajmani would not know that decades later, his company would buy Corus, previously known as British Steel, Jaguar Land Rover and Tetley Tea - and be so significant in the UK steel industry.

The 1971 and 1972 papers present the rules for members restrained on one side only – the tension flange, as typically found with a portal frame column. The work undertaken by Horne and Ajmani leads directly to the stable length rules found in Section 5 and Annex G of BS 5950. In turn, this leads directly to the rules found in BS EN 1993-1-1 section BB.3. Some 50 years later, current design rules depend on this research from the 1970s.

The discussion of the paper is perhaps most interesting. At the time, the UK design standard was BS 449. This standard offered guidance on the effective length of "stanchions" in Appendix D, and proposed that if a stanchion was restrained by side rails, the effective length factor in the minor axis was 0.75*L*. No limitation was placed on the maximum spacing of side rails – the effective length was always 0.75*L*. Figure 1 (Figure 15 from BS 449) is interesting in that the side rails are angles, and are drawn as a considerable proportion of the stanchion depth – around 50%. This is quite different to details found today.

A Professor Bryan was moved to comment that "as Sir John Baker once said, the effective length concept is most unsatisfactory in that one takes the length of a column and then multiplies it by a factor which very much depends on what you had for breakfast". Professor Bryan was complimenting Horne and Ajmani for their contribution in advancing the guidance. The work of Baker *et al* is discussed later in this article, though Baker himself correctly credits a Mr John Mason with the original quote.

Another contributor, Mr Dwight, noted that "it will at last be possible to take account of the restraint afforded by sheeting rails connected to the tension flange". Mr Dwight also commented on "the practice sometimes adopted of bracing the sheeting rail back to the inner flange of the stanchion, thereby supposedly providing restraint to the compression flange". Mr Dwight appears to be sceptical about the effectiveness of the system commonly employed nowadays. Mr Dwight assumed that there was "negligible advantage in doing this because of the great flexibility of the sheeting rail". Professor Horne proposed verifying the relative stiffness of sheeting rail and restrained member – the checks appear in the SCI publications on portal frames with the recommendation that the verification is important when the member size starts to be disproportionate compared to the side rail.

Dr Morris recalled previous practice (he referred to the mid-1950s) and the "relatively simple calculations one used". He noted that "it would seem that as our knowledge of structural behaviour is extended, the design process is refined and becomes complex, and it may be the case in the near future of reverting back to simple elastic design". Although Dr Morris was apparently enthusiastic about elastic design, some ten years later he collaborated with Professor Horne to publish the definitive guide on plastic design of portal frames[4]. Since 1970, the design process has become ever more complex, frequently reliant on computer aided analysis and member verification by software, rather than the simplicity Dr Morris suggested.

The 1970s were clearly a significant time for the development of the design rules for portal frames. In 1979, Professor Horne collaborated on a further paper considering the stability of haunched members[5]. The design rules for haunched and tapered members in Section G of BS 5950 follow from this paper, and from then were "translated" into Eurocode nomenclature in Section BB.3.2 of BS EN 1993-1-1. Eurocode expressions such as BB.14 and BB.16 (and their equivalents in BS 5950) can be immediately recognised in the 1979 paper.

The reference to "Point A", still used today, as the all-important junction between the bottom flange of the haunch and the inside face of the column is found in this paper, as illustrated in Figure 2 (Fig 1 from Horne *et al*)

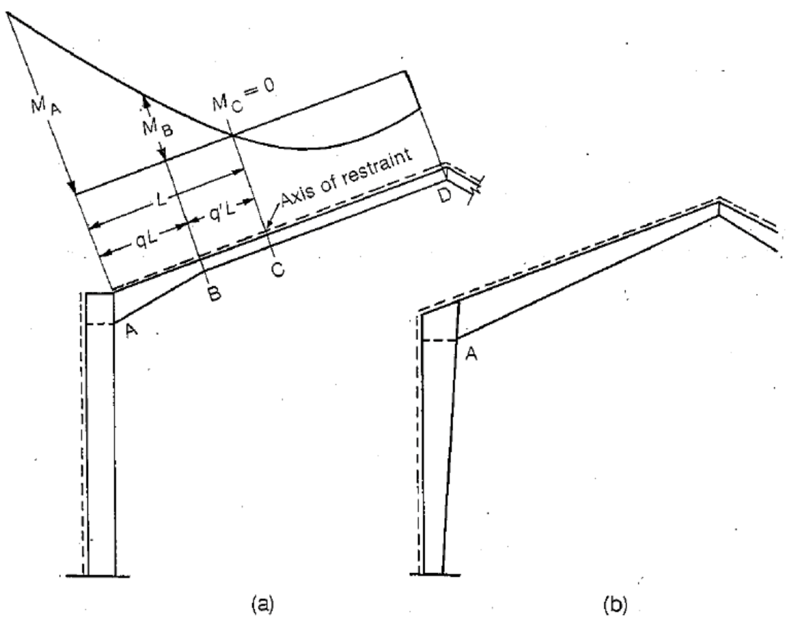

Figure 2: "Point A" – where inner flanges meet (Fig 1 from Ref 5)

The steel skeleton

The comment by Professor Bryan referring to Sir John Baker moves us back another step to the early 1950s and the two volumes of *The Steel Skeleton*[6,7]. Volume 1 covers "Elastic Behaviour and Design". Volume 2 covers "Plastic Behaviour and design" and perhaps it is no surprise that a co-author of volume 2 was Professor Horne – described as "one of the leading protagonists of plastic design".

Both volumes are worth reading, containing some really interesting history. Volume 1 looks back further to 1929 when an investigation was undertaken to investigate the application of modern theory to the design of steel structures. This review considered practice in the UK, New York, Germany, France, Spain, Prussia and Belgium. The live loads to be designed for in different countries varied, as they did in various UK cities. Edinburgh and Glasgow agreed that halls, schools and churches must be designed for 180lb per sq.ft, (8.6 kN/m^2) whereas Newcastle was content with 112 lb per sq.ft (5.4 kN/m^2). Perhaps children and worshippers were not so socially distanced north of the border.

Volume 1 also records the live load reductions that were allowed in various countries, linking to the reduction factors we find in modern codes.

The significance of the loaded area reduction is also evident in the intensities of loading surveyed. Who would want to work in a (presumably claustrophobic) small finance company, when the loading was measured at 11 kN/m^2, compared to a structural engineering company at 4.5 kN/m^2? However, when measured over a larger area, the situation is more comfortable; the loadings become 0.9 and 0.6 kN/m^2 respectively. Over a larger measured area, "consulting engineers" (as opposed to "structural engineers") fare the worst at 1.7 kN/m^2. The survey also noted that concentrated loads, such as fire cabinets and safes needed special attention – another principle found in our modern loading codes.

Of equal interest in Volume 1 are the tests undertaken on real buildings. For one building, a hotel under construction by Dorman Long, it was suggested that a few platoons of soldiers from a nearby barracks could have been used to provide a well-distributed load. In the event, point loads were suspended from the beams. Strain gauge readings were affected by the riveting operations - a hazard not experienced today. A summary of the findings is that the measured effects in beams and stanchions were not as expected. Professor Baker described the behaviour as "radically different from that assumed in the design methods in common use". One key difference was that the riveted connections were relatively stiff, making the frame behaviour more like a rigidly jointed frame than a pin-ended arrangement. It is worth remembering that designers are *modelling* loading and *modelling* the structure and its response.

The Steel Structures Research Committee produced "Recommendations for Design" based on these studies, which was essentially a semi-continuous design method, recognising the stiffness of the connection types used at the time. Unsurprisingly, the connections had to be classified (based on the detailing) – which was associated with the connection stiffness. We might reflect on the current guidance in BS 5950 that the detailing of the connections must be consistent with the assumptions made in the frame analysis, and the explicit requirement in the Eurocode to classify connections and allow for connection stiffness if the effects are significant. This is the same principle advocated in the proposed design method some 90 years ago.

The proposed design methods, published in 1936, were complicated. Baker comments that despite the constructional steel industry paying for the research for a period of 7 years "neither the industry itself nor consulting engineers generally felt any enthusiasm for the outcome of their labours". Baker noted that "whatever criticism could be levelled at the method of design which had held the field for nearly fifty years, it certainly had the merit of simplicity; in fact it would be difficult to imagine anything simpler". Baker also noted that the recommended procedures were laborious, and there was "no advantage that the average client would appreciate". The orthodox method of design was shown to be safe, if quite conservative in some cases.

Wartime regulations

Once war broke out, steel was a very important commodity. A wartime amendment was made to BS 449 which increased the permissible stresses by 25%. Baker notes that "this earned the taunt that the engineer had discovered that steel was stronger in war than in peace, whereas all he was admitting was that greater risks had to be taken in wartime".

In 1939, a recommendation was made that for government buildings, design should be based on the "more exact design methods" proposed by the Committee, but Baker notes "there is no evidence that this last wise recommendation has ever been acted upon".

Holding nothing back!

In 1943, BS 449 was revised, still not embracing the more exact methods. One senses a degree of disappointment when Baker notes "the third revision…. owes nothing to the tests of existing buildings and the other resources… except that it has achieved almost all the economy possible in beams without bothering to define the vital end-connections to be used". He suggests it perpetuates "a design method which neglects almost every effect but axial load and can only be defended on the score of expediency". He does not spare his criticism – the method which was originally adopted as "an empirical method well proved by years of use to be safe… has been changed in a haphazard way unjustified by practical experience or the results of scientific investigation". The method which so frustrates Baker is the assumption of nominal moments due to beam end reactions 100 mm from the face of a column – a method still loved in the UK found in clause 4.7.7 of BS 5950 and available to Eurocode designers via NCCI.

Baker concludes that further economy was certainly possible and that too much attention should not be paid to the complexity of the proposed design method "for it does less than justice to the abilities of steelwork designers". Some 70 years later, perhaps we are on the advent of embracing semi-continuous design, being armed with numerical methods and software that will determine the stiffness of connections and software which can include connection stiffness in the frame analysis.

Lessons from the war

Appendix B of Volume 1 reports on multi-storey steel frames subject to air attack. The appendix notes that it also shows what happens when these structures are subject to conditions of overload.

Of interest is the comment that "the floor (construction) which can best tie the members of the main frame together is to be preferred". Today, we would discuss the subject under "the avoidance of disproportionate collapse". Baker notes that "hardly ever does progressive collapse take place", unless "the explosion caused failure of certain beam-to stanchion connections" (on the façade) "and allowed the external wall framework to move outwards, when a certain amount of collapse ensued". Today, we would recognise the need for connections to not only carry the vertical shear, but also the tying forces to avoid exactly this problem. The appendix also makes recommendations about the layout of beams, which we would recognise as the arrangement of horizontal ties. Despite these clear recommendations, the disproportionate collapse at Ronan Point in 1968 is usually noted as the catalyst for the modern tying rules, perhaps because the risk of blast from high explosive ended in 1945.

Conclusions

Many of the features of our modern codes have their roots in work completed many years ago – some expressions are precisely those proposed over 50 years ago. That original work was hugely significant, influencing much of what we do today, from loading to resistance calculations. Perhaps the tools that structural engineers now have available will finally facilitate progress from the empirical methods which Professor Baker described as "almost entirely irrational and therefore incapable of refinement".

To appreciate something of the background and reasons for certain requirements must always be helpful. As Professor Baker notes: "it is important that the steelwork designer should not become a technician blindly applying irrational rules. He can only escape from this role if he has the information on which he can base better rules"

1 Horne, M. R. and Ajmani, J. L.
Design of columns restrained by side rails
The Structural Engineer, August 1971

2 Horne, M. R. and Ajmani, J. L.
Failure of columns laterally supported on one flange
The Structural Engineer, September 1972

3 *Failure of columns laterally supported on one flange; Discussion*
The Structural Engineer, July 1973

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Truss joint design – open sections

Following on from the truss joint design presented in the October 2019 issue, David Brown of the SCI reviews the design of a joint with a conventional arrangement of open sections.

Conventional – or common?

The October issue of New Steel Construction addressed a heavily loaded joint in a truss, explaining the thought process that led to the decisions firstly to orientate the UC chord members with web horizontal, secondly to use similar sized sections for the web members to facilitate the joint design and lastly to fabricate the node from plate.

In common practice, truss joints between open sections are often simply arranged with the webs vertical and with the web members as smaller sections than the chords.

That arrangement leaves the connection designer to determine how the forces in the members are to be transferred, recognising that elements in the joint are often perpendicular to each other, which is never ideal.

The particular joint considered in this article is shown in Figure 1, although the thought process and element verifications are more important than the actual detail.

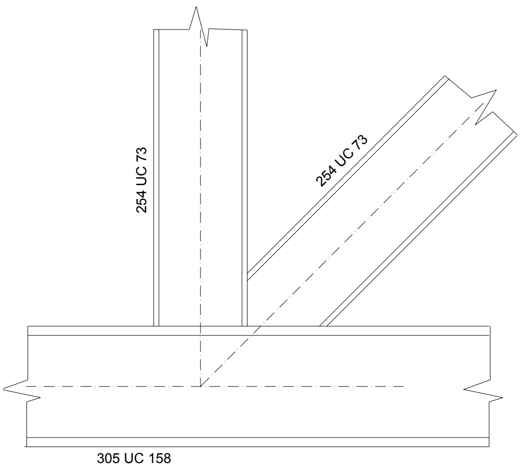

Figure 1: Open section joint

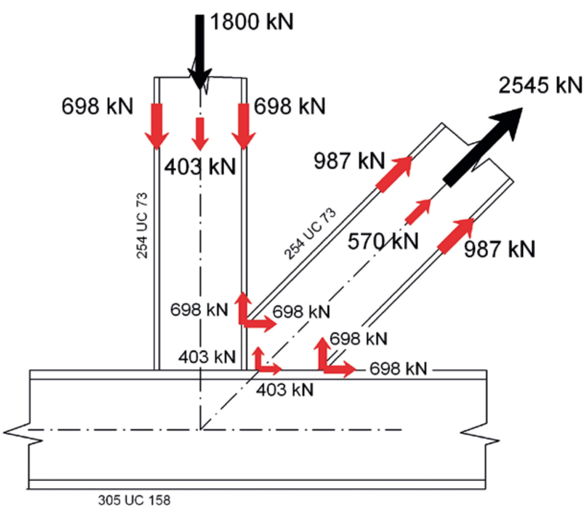

Figure 2: Forces in cross section elements

The vertical web member has an axial compression of 1800 kN. The diagonal, which is at 45°, therefore has an axial tension of 2545 kN and the joint is in vertical equilibrium. Many connection designers will release anguished howls at this point, since in reality they are unfortunately often given 'envelope' forces which are not in equilibrium and therefore doubly challenging to address.

For the purposes of this example, it is assumed that the force in the chord is 75% of its tension resistance. Because the flange of the 305 UC 158 is 25mm, the design strength is 345 N/mm^2 (all members are S355) and the axial force is therefore 5200 kN.

Distribution of forces

A helpful approach is to consider how the forces are distributed within the elements of the cross section. The area of a UC flange is typically 40% of the entire cross section (38.8% for the 254 UC 73), meaning that the element forces in the diagonal and vertical members are as shown in Figure 2.

At the connection points, these element forces have been further split into the two orthogonal components.

Connections to unstiffened flanges

Under local loads, webs might need reinforcement under compression, or under tension.

Before those checks are considered, stiffeners might also be required to stiffen the flange so that the full width of connected parts is effective. Stiffeners required for this purpose are more likely to be needed than to reinforce the web, so it is wise to complete these checks first.

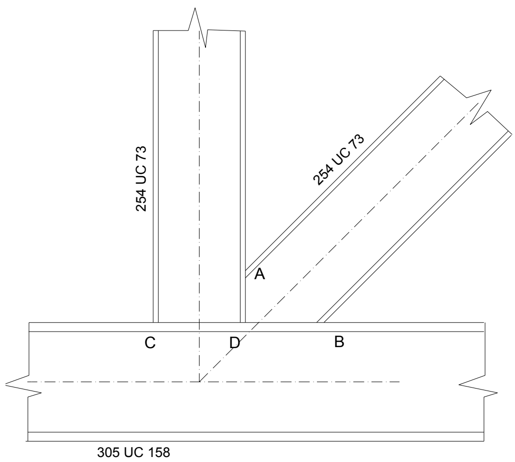

Figure 3: Connections to potentially unstiffened flanges

There are connections to (potentially) unstiffened flanges at points A and B (in tension) and C and D (in compression) as shown in Figure 3.

If the flange is unstiffened, the more flexible tips of the flange deform and the stress distribution across the connected plate (in this case the flange of the incoming UC) is non-uniform. Design codes calculate an effective breadth, over which the stress is assumed to be uniform.

The verification is covered in clause 4.10 of BS EN 1993-1-8. The effective breadth, b_{eff} must be calculated, which assumes a spread through the flange

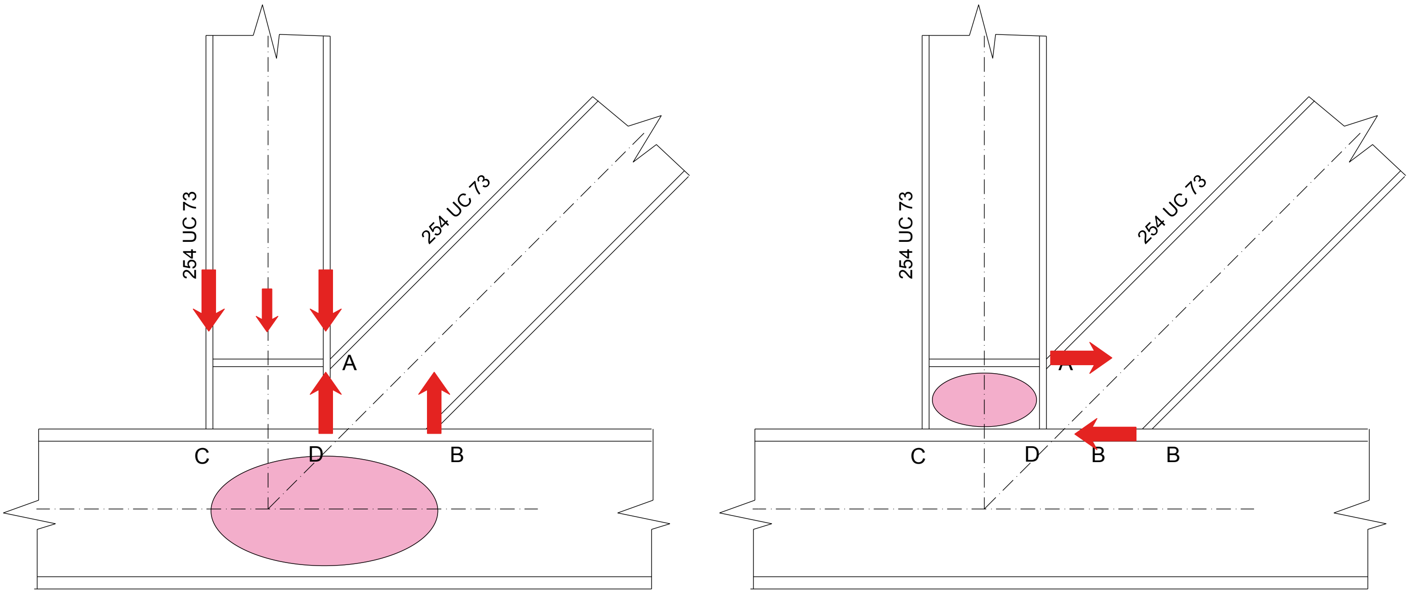

Figure 4: Zones where shear resistance must be verified

from the web and root radius. b_{eff} is given by: $b_{eff} = t_w + 2s + 7kt_f$

where $k = \left({}^{t_f}\!/_{t_p}\right)\left({}^{f_{y,f}}\!/_{f_{y,p}}\right)$ but $k \leq 1$

The requirement is then:

$$b_{eff} \geq \left({}^{f_{y,p}}\!/_{f_{u,p}}\right) b_p$$

It should be noted that both $f_{y,p}$ and $f_{u,p}$ relate to the plate (again in this case, the flange of the incoming UC). This requirement means that: $b_{eff} \geq 0.75b_p$

If this requirement is not met, then clause 4.10(3) says "Otherwise the joint should be stiffened". Readers will note that the applied force has not featured in this verification – the check is purely geometric, without reference to any force. If the force is small, this requirement seems unreasonable.

BS 5950 had an altogether more sensible approach in clause 6.7.5. The applied force F_x was limited to the resistance P_x obtained from the effective breadth, so connections with low forces could be accommodated without stiffening.

According to BS 5950, stiffening had to be provided if $b_e < 0.5\ (F_x / P_x)\ b_p$ but this is a much less onerous requirement than the Eurocode.

At point A, the effective width, b_{eff} is 133mm ($k = 1$)

The limit = $\left({}^{f_{y,p}}\!/_{f_{u,p}}\right) b_p$ = (355/470) × 254.6 = 193.5 mm

So according to the Eurocode, stiffening is required at point A.

At point B, the value of k in clause 4.10(2) is calculated as 1.7, but limited to a maximum of 1.0.

The effective width, b_{eff} is 221 mm ($k = 1$)

The limit = $\left({}^{f_{y,p}}\!/_{f_{u,p}}\right) b_p$ = (355/470) × 254.6 = 193.5 mm,

which means that stiffening might not be needed – other verifications need to be completed.

Tension stiffener design

At point A, it is convenient simply to assume all the applied horizontal component must be carried into the stiffeners in the vertical member. The resistance of two stiffeners, each 120 × 10 mm in S355 is 852 kN, which exceeds the 698 kN applied.

The weld to the inside of the flange is continued round the root radius, rather than being stopped, so only one leg length (strictly to the Eurocode, a throat length) is deducted from the weld length.

Thus there is 4 × (120 – 8) = 448 mm of weld, assuming an 8 mm fillet weld. This is a transverse weld, so has a resistance of 1.65 kN/mm. The applied force is 698 / 448 = 1.56 kN/mm, so 8 mm fillet weld is OK.

That force must be transferred to the web, between fillets, (it has nowhere to go at the other flange!), so the force in the weld is 698 / (4 × 200) = 0.87 kN/mm. A 6 mm fillet weld would be OK, but practically the same 8 mm fillet weld all round would be specified. Note that this force transferred into the web appears as a shear force in the vertical member.

Web in tension at point B

Although no stiffeners to support the flange are needed, the web of the chord experiences the local tension of 698 kN.

The resistance of the web is given in BS EN 1993-1-8 clause 6.2.6.3, which involves an effective breadth of web, $b_{eff,t,wc}$ and a reduction factor ω due to shear in the web.

This is determined from Table 6.3, which leads backwards to Table 5.4 and a challenging decision on the value of β to be taken. After some consideration, the situation seems most like the shear in a web panel from a one sided moment connection, so $\beta = 1$.

After a frustrating trip back to BS EN 1993-1-1 to calculate the shear area, ω is computed to be 0.82.

The web resistance is computed to be 988 kN, which is more than the applied force of 698 kN, so no stiffener is needed for web tension.

Shear resistance

The shear in the web of the chord and in the web of the vertical member can be calculated by considering the components of force in the appropriate direction, as shown in Figure 4.

A convenient approach is to draw the local shear force diagram due to the applied components of force. Note that this only works if the applied forces are in equilibrium. The shear force diagram for the chord is shown in Figure 5.

Looking in the Blue Book, the shear resistance of the 305 UC 158 is 1130 kN, so it seems highly unlikely that the chord web will be satisfactory when the shear stress is considered in combination with the axial stress.

Shear and axial stress combined

The combination of stresses can be considered using the Von Mises criterion, found in clause 6.2.1 of BS EN 1993-1-1. Designers may not often use this clause, as normal cases have their own specific verifications later in

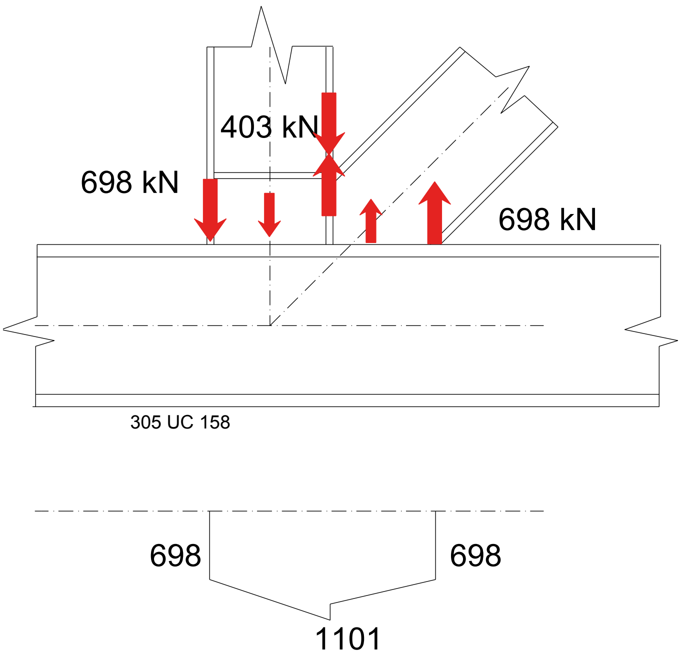

Figure 5: Shear force diagram for the chord

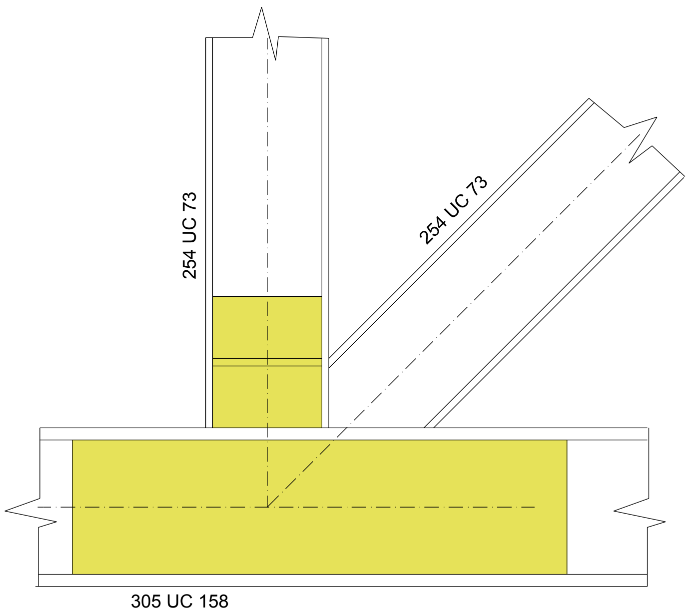

Figure 6: Final joint detail

section 6, but this elastic check is useful in unorthodox situations.

Considering just longitudinal and shear stresses, the criterion becomes:

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2 + 3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right)^2 \leq 1$$

The ratio $\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2 = 0.75$

The value of τ_{Ed} must be calculated as it is the elastic shear stress at the neutral axis. Normally, designers calculate a plastic resistance, so do not know the value of τ_{Ed}.

Designers will use an expression such as $\tau = \frac{SA\bar{y}}{It}$, depending on the form of the mnemonic they use!

The values of $\bar{y}$ and A can be taken directly from the section properties for tees cut from UC sections.

$$\tau_{Ed} = \frac{1101 \times 10^3 \times 10100 \times 13^3}{38700 \times 10^4 \times 15.8} = 242 \text{ N/mm}^2$$

Substituting into the Von Mises criterion:

$(0.75)^2 + 3\left(\frac{242}{345/1}\right)^2 = 2.03$, which is unsatisfactory, as expected.

Supplementary web plate

A supplementary web plate is one option, with the design rules given in clause 6.2.6.1. Because of limited research, the contribution of a supplementary web plate is limited to a maximum thickness equal to the web it reinforces, even if the additional plate is thicker than the web. Adding a plate to the other side of the web makes no further increase in the shear resistance, which seems implausible.

With the objective of using the Von Mises criterion a second time, the shear stress in the compound section must be calculated. Although a thicker plate was selected, the calculated inertia, area and distance to the centre of gravity used only the additional 15.8 mm permitted by the Standard. The longitudinal stress was also reduced by considering the additional area, once again limiting the credited addition to the 15.8 mm, despite specifying a 20 mm plate.

The calculates stresses were
$\tau_{Ed} = 125$ N/mm^2 and $\sigma_x = 218$ N/mm^2
Substituting the Von Mises criterion:

$$\left(\frac{218}{345/1}\right)^2 + 3\left(\frac{125}{345/1}\right)^2 = 0.79$$

The length of plate past the critical area needs to be sufficiently long so that the welds can transfer the axial forces assumed in the supplementary web plate.

Other checks

The same process is needed for shear in the vertical member (see Figure 4), where it will be found that reinforcement is also required. Welds between the web and chord members must also be designed.

The compression resistance of the web at point C (Figure 3) requires verification – but is not a problem with the supplementary web plate provided.

The final joint is indicated in Figure 6. Instead of supplementary web plates, a detail using diagonal shear stiffeners could be developed, although the room for diagonal members is rather limited.

Conclusions

As always, a thoughtful consideration of the member selection and member orientation might have avoided some of the more expensive reinforcement required for this particular detail. A second observation is that the necessity to stiffen without any reference to the applied force seems very onerous – it is hoped that some work can be done to modify this requirement.

The good news is that the proposed revisions to BS EN 1993-1-8 do allow more benefit to be taken from supplementary web plates. Finally, the example serves as a reminder that the Von Mises criterion, presented in clause 6.2.1, can be useful when no other option exists.

Design of beam-column splice connections according to Eurocode 3

Ricardo Pimentel of the SCI discusses the design of beam-column splice connections considering second-order effects due to combined flexural and lateral torsional buckling according to Eurocode 3.

Introduction

Buckling phenomena cause additional internal forces within members due to local second order effects (P-δ). Recent NSC articles [1], [2], [3] introduced these effects, giving theoretical background and practical applications. Reference [3] provides a detailed worked example of the assessment of the second order bending moment on columns due to strut action for column splices designed under pure compression. Members subjected to major axis bending that are susceptible to lateral torsional buckling are also subjected to second order effects, because the major axis bending induces a horizontal deflection (minor axis - δ_h), vertical deflection (major axis - δ_v) and a cross-sectional rotation (θ) as illustrated in Figure 1. Such deformations will increase as the applied bending moment increases. When the bending moment is close to the so-called elastic critical moment, the deformation increases rapidly and failure occurs.

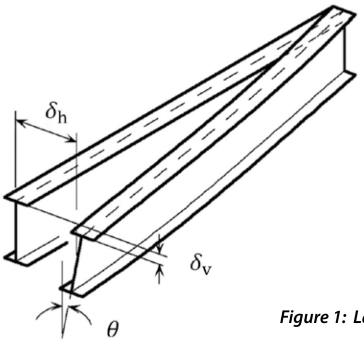

Figure 1: Lateral torsional buckling mode shape

Addressing second order effects

Whilst for a strut an equivalent initial bow imperfection can be back-calculated relatively easily and amplified to account for the second order effect, the problem for lateral torsional buckling phenomena offers a much more complex challenge. Although the effects of the vertical displacement and rotation have an impact on the lateral torsional buckling resistance of the member, the consideration of an equivalent horizontal out of plane bow imperfection offers a good approximation to establish the initial member imperfection. EN 1993-1-1 clause 5.3.4 (3) supports this approach. A precise analysis including the amplification of the initial member imperfection is complex and usually undertaken by numerical analysis with advanced finite element model tools. A reasonable approximation can be achieved by manual methods, as demonstrated in this article. The process described is useful when designing splice connections in unrestrained beams.

Lateral torsional buckling failure criteria

The design buckling resistances for buckling phenomena according to Eurocode philosophy are calibrated based on an elastic cross section failure, where all imperfections (such as residual stresses, lack of straightness, etc.) are accounted for by an equivalent imperfection factor α. Second order local effects are implicitly considered by the Eurocode design method (section 6.3). Reference [1] explains this concept for a strut. Using the same principles for an element subjected to lateral torsional buckling, the buckling failure can be understood as a critical stress, for which two components can be identified: (i) component due to major axis bending (σ_{M_y}); (ii) component due to the second order bending moment under minor axis bending ($\sigma_{M_{z,P\delta,LTB}}$).

Out of plane bending moment due to lateral torsional buckling

If the buckling failure is considered as an elastic cross section failure (with a material yield strength of f_y), the following condition can be established:

$$f_y = \sigma_{M_y} + \sigma_{M_{z,P\delta,LTB}}$$

According to Eurocode nomenclature, the buckling resistance can be established as the product of the reduction factor for buckling phenomenon χ multiplied by the design characteristic resistance. As the characteristic resistance is directly proportional to the material resistance, the stress at lateral torsional buckling failure can be established as $\chi_{LT} \cdot f_y$ (described as the critical buckling stress). The stress $\sigma_{M_{z,P\delta,LTB}}$ can be defined based on cross section properties and the second order bending moment $M_{z,P\delta,LTB}$, which leads to:

$$f_y = \chi_{LT} \cdot f_y + \frac{M_{z,P\delta,LTB}}{W_{el,z}}$$

Dividing the previous equation by the critical buckling stress, it can be demonstrated that:

$$\frac{f_y}{\chi_{LT} \cdot f_y} = \frac{\chi_{LT}}{\chi_{LT} \cdot f_y} \cdot f_y + \frac{M_{z,P\delta,LTB}}{\chi_{LT} \cdot f_y \cdot W_{el,z}} \leftrightarrow M_{z,P\delta,LTB} = \left(\frac{1}{\chi_{LT}} - 1\right) \cdot \chi_{LT} \cdot M_{z,el,Rk}$$

Where $M_{z,el,Rk}$ is the out of plane elastic bending resistance of the cross section.

According to the Eurocode definition, χ_{LT} is the ratio between the buckling bending resistance and the characteristic bending resistance of the cross section. As the buckling bending resistance ($M_{b,Rd}$) should be always less than the applied bending moment ($M_{y,Ed}$), it can be approximately (and conservatively) assumed that:

$$\chi_{LT} = \frac{M_{b,Rd} \cdot \gamma_{M1}}{M_{y,el,Rk}} \approx \chi_{LT} = \frac{M_{y,Ed} \cdot \gamma_{M1}}{M_{y,el,Rk}}$$

Where γ_{M1} is that partial factor for buckling phenomenon according to the UK NA to BS EN 1993-1-1 [4].

This leads to:

$$M_{z,P\delta,LTB} = \left(\frac{1}{\chi_{LT}} - 1\right) \cdot \frac{M_{z,el,Rk}}{M_{y,el,Rk}} \cdot M_{y,Ed} \cdot \gamma_{M1} \qquad \text{Eq (1)}$$

The complexity of the procedure is related to the calculation of χ_{LT}. For cases where section 6.3.2.3 (2) of EN 1993-1-1 is applied, $M_{z,P\delta,LTB}$ should be multiplied by "f".

Splices of elements under compression

Splices subjected to axial compression should be designed for the following forces:

1. N_{Ed} – Applied axial force;
2. $M_{i,P\delta,FB}$ – Second order bending moment due to strut action (flexural buckling) about the axis "i".

It should be clear that a member only experiences flexural buckling under one of its axes. The design bending moments $M_{i,P\delta,FB}$ should be only considered about the weak axis for flexural buckling (i.e. the axis which shows the higher slenderness – reflected in a higher value of $\overline{\lambda}$ - according to EN 1993-1-1 section 6.3.1.2).

The second order bending moment due to strut action can be calculated as follows:

$$M_{i,P\delta,FB} = N_{Ed} \cdot e_{P\delta,i} = N_{Ed} \cdot e_{0,i} \cdot k_{amp,i} \cdot \gamma_{M1} \qquad \text{Eq. (2)}$$

Where:

N_{Ed} is the applied axial load;

$e_{0,i}$ is the initial bow imperfection about axis "i" equal to $\frac{W_{el,i}}{A}\,\alpha\,(\bar{\lambda}_i - 0.20)$;

$e_{P\delta,i}$ is the bow imperfection accounting for the second order effects;

$k_{amp,i}$ is the amplification factor equal to $\frac{N_{cr,i}}{N_{cr,i} - N_{Ed}}$;

$W_{el,i}$ is the elastic modulus of the cross section about axis "i";

A is the cross-section area;

α is the equivalent imperfection factor according to EN 1993-1-1 section 6.3.1.2;

$\bar{\lambda}_i$ is the non-dimensional slenderness according to EN 1993-1-1 section 6.3.1.2 about axis "i";

$N_{cr,i}$ is the elastic critical buckling load for flexural buckling under the axis "i": $N_{cr,1} = \frac{\pi^2 EI_i}{L_{crit,i}}$, where I_i is the second moment of area about axis "i" and $L_{crit,i}$ is the buckling length about axis "i";

N_{Ed} is the applied axial load on the column.

Splices of elements under bending

Splices within unrestrained segments subjected to major axis bending should be designed for the following forces:

1. $M_{Ed,y}$ – Applied bending moment under the major axis;
2. $M_{Ed,z}$ – Applied bending moment under the minor axis;
3. $M_{z,P\delta,LTB}$ – Second order bending moment due to lateral torsional buckling.

Beam-column splices

Beam-column splices can be exposed to the following design forces:

1. N_{Ed} – Applied axial force;
2. $M_{Ed,y}$ – Applied bending moment about the major axis;
3. $M_{Ed,z}$ – Applied bending moment about the minor axis;
4. $M_{i,P\delta,FB}$ – Second order bending moment due to strut action (flexural buckling) about the axis "i";
5. $M_{z,P\delta,LTB}$ – Second order bending moment due to lateral torsional buckling;
6. $M_{i,P\delta,Amp}$ – Moments due to the amplification of the applied bending moments due to the strut action about the axis "i".

As for elements under compression, the design bending moments $M_{i,P\delta,FB}$ should be only considered about one of the cross-sectional axes for flexural buckling. Beam-columns experience an additional bending moment $M_{i,P\delta,Amp}$ which is related to the amplification of the applied bending moments due to the presence of axial load. The second order bending moments due to the presence of axial force can be calculated considering the amplification factor about the axis "i" as follows:

$$M_{i,P\delta,Amp} = M_{Ed,i} \cdot \left[\frac{N_{cr,i}}{N_{cr,i} - N_{Ed}} - 1\right] \qquad \text{Eq. (3)}$$

The minor axis bending moment $M_{z,P\delta,Amp}$ should be always considered. The effects of $M_{y,P\delta,Amp}$ and $M_{z,P\delta,Amp}$ should not be considered together: designers should consider two independent combination of action where $M_{y,P\delta,Amp}$ or $M_{z,P\delta,Amp}$ are considered. This is because the second order effects will only develop about one of the member axes, i.e. either LTB will govern and the beam will deform sideways, or a major axis second order bending moment will be generated.

The procedure described above comprises segments under a uniform bending moment profile along the segment. To assess other bending moment profiles, designers may consider the value of $C_{m,i}$ from EN 1993-1-1 Table B.3. For such cases, the values of $M_{i,P\delta,Amp}$ obtained from equation 3 may be multiplied by the values of $C_{m,i}$.

As a summary, the design forces for a beam-column splice can be established by the following equations:

$$N_{Ed,splice} = N_{Ed} \qquad \text{Eq. (4)}$$

$$M_{Ed,y,splice} = M_{Ed,y} + [M_{y,P\delta,FB}] + \{M_{y,P\delta,Amp}\} \qquad \text{Eq. (5)}$$

$$M_{Ed,z,splice} = M_{Ed,z} + [M_{z,P\delta,FB}] + \{M_{z,P\delta,LTB}\} + M_{z,P\delta,Amp} \qquad \text{Eq. (6)}$$

Pairs of effects within the square and round brackets should not be considered simultaneously. Designers should consider them individually and assess which combination of forces gives the most onerous design condition.

Second order bending moment distribution along an unrestrained segment

The bending moment diagrams calculated according to equations 1, 2 and 3 represent a maximum value at mid span of an unrestrained segment. The second order bending moments follow a sinusoidal shape between points of inflexion (points between which the effective length is measured) of: $M_{i,P\delta}(x) = M_{i,P\delta,max} \cdot \sin(\pi \cdot x / \ell)$, where "$x$" is the position from a point of inflexion and "ℓ" is the length between points of inflexion (for a pinned column, this is the column length).

Comparison with BS 5950 approach

Previous UK practice design addressed second order effects for columns, beams and beam-column splices according to BS 5950 [6]. Further guidance was given by SCI AD notes 243 [7] and AD 244 [8].

The second order out of plane bending moment is addressed by BS 5950 Annex B.3. While BS 5950 established the second order bending moment based on a relationship between yield strength and bending strength for lateral torsional buckling, the Eurocode nomenclature establishes it based on the parameter χ_{LT}. The parameter χ_{LT} can also be understood as a relationship between the allowable buckling stress and the yield strength. Therefore, $1 / \chi_{LT}$ represents the same relationship as proposed by BS 5950. The factor m_{LT}, which considers the bending moment diagram shape along the segment, is accounted for while calculating χ_{LT} according to EN 1993-1-1 6.3.2 (within the elastic critical bending moment - M_{cr}). Both BS 5950 and Eurocode 3 approach have the same background.

Strut action is defined by Annex C.3 of BS 5950. Both BS 5950 and EN 1993-1-1 approaches to address flexural buckling are based on an elastic cross section failure due to the combined stresses of axial load and second order bending moments due to the strut action. If the same buckling resistances are assumed, and considering the elastic section modulus, the simplified method from Annex C.3 of BS 5950 tends to give conservative values in comparison with equation 2. A similar answer for the strut moment is obtained if the applied load is close to the buckling resistance.

Second order effects for members subjected to combined axial load and bending are defined by Annex I.5 of BS 5950. The expression $1/(p_{Ei}/f_c - 1)$ gives the same answer as $[(N_{cr,i}/(N_{cr,i} - N_{Ed}) - 1]$ if the same buckling resistances are assumed. The values of m_y and m_x according to BS 5950-1 Annex I.5 (which should be defined according to BS 5950-1 4.8.3.3.4) are similar to the values defined by EN 1993-1-1 Table B.3.

Calculation example

Consider a UB 533 × 165 × 66 beam-column element with an unrestrained segment of 5 m length subjected to an axial load of 150 kN and a linear bending moment diagram between 165 kNm and 82.5 kNm. A splice connection is located at 1/3 (1.67m) of the unrestrained segment length, closer to the point of maximum bending moment. The bending moment at the splice location is therefore 137.5 kNm. The calculation of the second order design forces to design the splice connection is summarized in the table on the next page. Member resistances are taken from the Blue Book.

Conclusions

1. Lateral torsional buckling failure can be considered by means of an equivalent initial horizontal bow imperfection under the minor axis of the profile, which must then be amplified;
2. Considering the member lateral torsional buckling capacity, it is possible to estimate the cross-section forces at failure;
3. The failure criteria for lateral torsional buckling is assumed to be elastic failure of the cross section considering major axis bending and the second order bending moment due to lateral torsional buckling; strut

Section properties and resistances, critical loads (S355); UB 533 × 165 × 66	Eurocode buckling Resistances	EN 1993-1-1 P-δ effects	Critical design effects for splice design
$A = 83.7\ \text{cm}^2$	$N_{b,rd,y} = 2890\ \text{kN}$	$k_{amp,y} = 1.005$	$N_{Ed,splice} = N_{Ed} = 150\ \text{kN}$
$W_{el,y} = 1340\ \text{cm}^3$	$N_{b,rd,z} = 598\ \text{kN}$	$k_{amp,z} = 1.267$	$M_{Ed,y,splice} = M_{Ed,y} = 137.5\ \text{kNm}$
$W_{el,z} = 104\ \text{cm}^3$	$M_{b,rd} = 225\ \text{kNm}$	$\bar{\lambda}_z = 2.04$	$M_{Ed,z,splice} = M_{z,P\delta,FB} + M_{z,P\delta,LTB}$ $M_{Ed,z,splice} = 1.3 + 16.2 = \pm 17.5\ \text{kNm}$
$W_{pl,y} = 1560\ \text{cm}^3$	Note: $C_1 \approx 1.35$	$e_{0,z} = 7.8\ \text{mm}\ (\alpha = 0.34)$	The set of design actions presented above give the most onerous design scenario according to equations 5 and 6.
$W_{pl,z} = 166\ \text{cm}^3$		$e_{P\delta,z} = 9.9\ \text{mm}$	
$I_y = 35000\ \text{cm}^4$		$M_{z,P\delta,FB,max} = 1.5\ \text{kNm}$	
$I_z = 859\ \text{cm}^4$		$M_{z,P\delta,FB}$ (@ 1.67 m) = 1.3 kNm	
$M_{y,pl,Rd} = 554\ \text{kNm}$		$M_{z,P\delta,LTB,max} = 18.7\ \text{kNm}$	Note: the value of χ_{LT} was calculated as $M_{b,rd} / M_{y,rd} = 225/554 = 0.41$
$M_{z,pl,Rd} = 59\ \text{kNm}$		$M_{z,P\delta,LTB}$ (@ 1.67 m) = 16.2 kNm	
$M_{y,el,Rd} = 474\ \text{kNm}$		$M_{y,P\delta,Amp,max} = 0.86\ \text{kNm}$	
$M_{z,el,Rd} = 36.9\ \text{kNm}$		$M_{y,P\delta,Amp}$ (@ 1.67 m) = 0.74 kNm	
$N_{cr,y} = 29017\ \text{kN}$		($C_{m,y}$ is assumed as 1 considering the low value of $M_{y,P\delta,Amp,max}$)	
$N_{cr,z} = 712\ \text{kN}$			

Design forces and bending moments for splice design

action effects also need to be accounted for in beam-columns;

4. EN 1993-1-1 approaches for beam and beam-column splices follow the same principles as BS 5950.

References

1 Pimentel, R., Stability and second order of steel structures: Part 1: fundamental behaviour; New Steel Construction; vol 27 No 3 March 2019;

2 Pimentel, R., Stability and second order of steel structures: Part 2: design according to Eurocode 3; New Steel Construction; vol 27 No 4 April 2019;

3 Eurocode 3 - Design of steel structures - Part 1-1: General rules and rules for buildings; BSI, 2014;

4 NA BS EN 1993-1-1+A1 UK National Annex to Eurocode 3 - Eurocode 3 - Design of steel structures - Part 1-1: General rules and rules for buildings; BSI, 2014;

5 Henderson, R., Bearing splice in a column; New Steel Construction; vol 28 No 3 March 2020;

6 BS 5950, Structural use of steelwork in building: Part 1: Code of practice for design - Rolled and welded sections, BSI, 2000;

7 SCI Advisory Desk Notes: AD 243: Splices within unrestrained lengths;

8 SCI Advisory Desk Notes: AD 244: Second order moments

Advisory Desk 2020

AD 436: Section classification of a flat plate

SCI is sometimes asked how to determine the section class of a flat plate as BS EN 1993-1-1 does not include this section in table 5.2. The purpose of this note is to provide guidance.

A flat plate of width *b* and thickness *t* loaded in axial compression is not susceptible to local buckling because there is no intersection of plates to provide a stiff axis. Classification for axial compression is therefore irrelevant.

If the plate is acting as a beam with the minor axis vertical, lateral torsional buckling about the minor axis does not occur.

Lateral torsional buckling can occur due to bending about the major axis. It is assumed that the member is not likely to be designed plastically so the relevant limit is that for Class 3. SCI recommends a value of $b/t \leq 19\varepsilon$ to provide a conservative limit for the Class 3 - Class 4 boundary.

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AD 437: Curtailment of transverse bar reinforcement in composite beams with steel decking designed using Eurocodes

The purpose of this Advisory Desk Note is to provide guidance on the curtailment of transverse bar reinforcement in slabs on composite beams with steel decking, designed to EN 1994-1-1. Such information was previously presented in AD 325, for design to BS 5950-3.1, but the provisions in EN 1994-1-1, and the clauses in EN 1992-1-1 to which it refers, give more explicit coverage of this topic than the BS rules. The approach to transverse bar curtailment is therefore different.

The transverse reinforcement is provided to transfer longitudinal shear force from the steel beam, via the shear connectors, out into the effective breadth of the slab. Transverse bar reinforcement may be needed to supplement the resistance of the mesh in the slab, and these bars must extend a sufficient distance from the beam centreline.

Internal beams

A fundamental difference between EN 1994 and BS 5950 is that the former adopts a so-called 'strut and tie' model, through which shear resistance is determined from consideration of concrete struts in compression and reinforcement ties in tension. A component of the force in the struts resists longitudinal force in the slab, and the component transverse to the beam axis is resisted by the reinforcement.

For composite beams with decking spanning perpendicular to the longitudinal axis of the beam, the critical transverse shear plane is adjacent to the steel flange. However, for decking running parallel to the beam the critical plane is normally in the nearest crest in the decking to the shear connectors (see Figures 1 and 2).

When considering the need for bar anchorage beyond these critical planes, for design to BS 5950, AD 325 made certain assumptions (with both a simplified and rigorous model) about how the force in the slab decreases across the effective width. The Eurocodes remove the need for such assumptions by providing explicit guidance:

1. EN 1994-1-1, 6.6.6.2 makes reference to EN 1992-1-1, 6.2.4
2. EN 1992-1-1, 6.2.4 (7) states that the reinforcement should be anchored beyond the strut requirement (see EN 1992-1-1, Figure 6.7)
3. EN 1992-1-1, 8.4.4 defines how to determine anchorage length

With reference to Point 2, determining the location that corresponds to 'beyond the strut requirement' is not obvious, particularly given that different angles can be chosen for the struts in what can be an iterative procedure. As a (slightly) conservative simplification, the point beyond which anchorage is needed may be assumed to be the critical planes, as defined above. This also results in an approach that is common to that used in design to BS 5950.

Point 3 refers to clauses that consider the tensile strength of the concrete, the strength of the reinforcing bars, and a number of other parameters. For typical bars in typical concrete the result will be a need for an anchorage length similar to the familiar value of 40 *d* (where *d* is the bar diameter). When lightweight concrete is used greater anchorage lengths are required, as a function of the concrete oven-dry density (see EN 1992-1-1, 11.3.1). Should larger bars be chosen than are necessary, such that they are stressed below yield, shorter anchorage lengths will suffice.

Although the Eurocode methodology makes no reference to the effective breadth of slab in the context of transverse shear resistance, this nevertheless remains an area of concrete subject to significant in-plane stresses. In the absence of a more rigorous analysis where a number of planes are considered rather than just the critical plane (which would most likely show that mesh alone is sufficient in the outer reaches of the effective

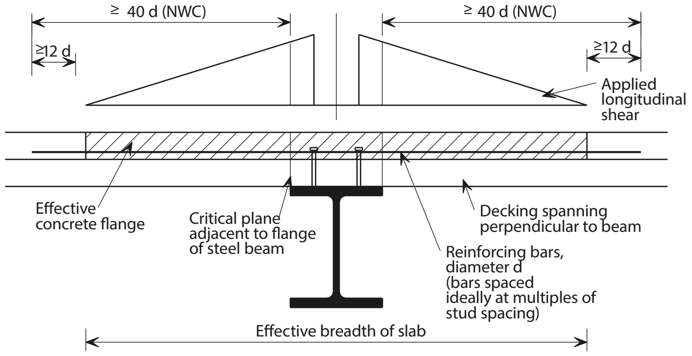

Figure1: Assumed pattern of transverse shear stresses and anchorage lengths, beam with transverse decking

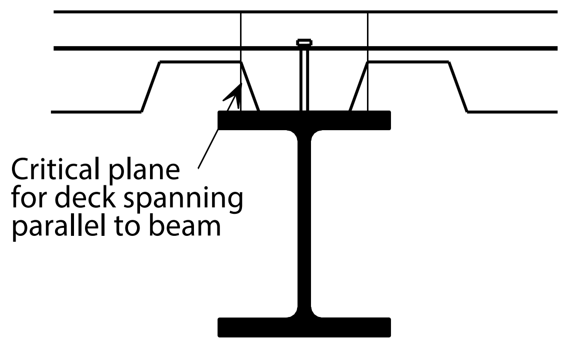

Figure2: Critical plane for deck spanning parallel to beam

breadth), we therefore recommend that the bars extend at least 12 *d* beyond the effective breadth. This is also in keeping with BS 5950 practice (AD 325 Simplified Method)

It is important to note that when the decking is perpendicular it may contribute to the transverse reinforcement needed, but when the decking is parallel it cannot be taken into account (it has no 'in-plane' tensile resistance so cannot contribute in a strut and tie model).

Edge beams

Notwithstanding differences in the definition of anchorage length, EN 1994-1-1, 6.6.5.3 contains detailing guidance for edge beams that aligns with that given in BS 5950-3.1:

- If the edge of slab from the centreline of the nearest shear connectors is less than 300 mm then place U-bars around the shear connectors
- Where headed studs are used, the U-bars must have a diameter not less than half that of the studs, and the distance from the edge of the slab to the centreline of the nearest studs should not be less than 6 d_s (where d_s is the stud diameter)

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AD 438: Non-slip connections to BS 5950

This AD deals with the BS 5950 provisions for connections designed to be non-slip in service, as described in clause 6.4.1(b).

Designers now using the Eurocodes will be familiar with Category B, slip-resistant at serviceability and Category C, slip-resistant at ultimate, as set out in Table 3.2 of BS EN 1993-1-8. In the Eurocodes, it is also clear that for Category B connections, the design slip resistance is compared to the serviceability loads.

Turning back to BS 5950 may led to some uncertainty about which loads to use when calculating slip resistance, particularly for connections designed to be non-slip in service. Clause 6.4.2 specifies the slip resistance P_{sL} as:

- For connections designed to be non-slip in service:
  $P_{sL} = 1.1K_s \mu P_0$
- For connections designed to be non-slip under factored loads:
  $P_{sL} = 0.9K_s \mu P_0$

In both cases, the resistance should be compared to the ultimate loads. This is made clear by the note at the end of clause 6.4.1: ***NOTE The resistance of a friction grip connection to slip in service is a serviceability criterion, but for ease of use is presented in a modified form, suitable for checking under factored loads.***

For connections which are designed to be non-slip in service, BS 5950 does not reduce the loads, but rather increases the calculated resistance of the bolts (compare the 1.1 factor with 0.9 in the above expressions) to give an equivalent result.

AD 274 gives advice on the capacity after slipping, covered in clause 6.4.4 of BS 5950. This is an important check for connections designed to be non-slip in service and is designed to ensure that if it slips, the connection does not fail at ultimate loads.

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AD 439: Transverse reinforcement in composite beams

This Advisory Desk note has been produced to reflect the publication in 2015 of P405 *Minimum degree of shear connection rules for UK construction to Eurocode 4*. As a result, AD 241: *Transverse reinforcement in composite beams* is redundant.

Transverse reinforcement in the form of mesh or additional loose bars is required in composite beam design to transfer the longitudinal shear force from the shear connectors (typically studs) into the effective width of the slab. Traditionally, light mesh reinforcement has been used throughout the slab, as a 'deemed to satisfy' approach, although BS 5950-3.1 and EN 1994-1-1 give explicit guidance than can result in a requirement for additional reinforcement.

This updated Advisory Desk Note focuses on Eurocode design, although the principles also apply to BS 5950 design so the guidance given may be readily adapted.

The requirements for transverse reinforcement in EN 1994-1-1 are based on the premise that the longitudinal shear resistance of the slab must be greater than the resistance of the shear connectors (i.e. the longitudinal shear force that can be transferred to the slab). Thereby the ability to achieve 6 mm slip at failure is maintained, because failure of the connectors will always be more critical. However, and this was the origin of AD 241, the number of shear connectors found in many composite beams is greater than the number needed to achieve the required beam resistance. Often, the design of composite beams is governed by serviceability limits, and they are not designed to achieve their full bending resistance. In such cases the studs provided are needed in order to satisfy the rules for minimum degree of shear connection, which are associated with limiting slip at the steel to concrete interface. So in terms of beam resistance alone, fewer studs could be used, and therefore less transverse reinforcement.

AD 241 therefore proposed applying a reduction factor to the longitudinal shear force that was a function of the applied moment divided by the moment resistance.

As noted above, a big change since AD 241 was originally written has been the publication of P405. Covering composite beams with both transverse and parallel decking, and considering a wider range of variables than EN 1994-1-1, it provides new rules for minimum degree of shear connection. In many cases the number of studs needed on a beam has dramatically reduced compared to the EN 1994-1-1 provisions. It is worth noting that one of the variables considered is the beam utilisation in bending, with minimum degree of connection now varying according to:

$$\left[\frac{M_{Ed}}{M_{Rd}}\right]^2$$

Applying the original guidance given in AD 241 alongside the guidance in P405 could therefore result in a certain amount of double counting. Moreover, when beams are designed in accordance with P405 it is unlikely that the 'old problem' of being unable to accommodate sufficient transverse reinforcement to provide a resistance in excess of that of the shear connectors will remain.

By applying the rules in P405 (which appear in numerous design software packages) there is no need for AD 241.

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AD 440: Fire design of external steelwork

SCI publication *P375 Fire resistance design of steel framed buildings* gives guidance on (amongst other things) the design of external steelwork when heated by a fire in a building fire compartment. This AD note clarifies which part of the publication should be used for this purpose.

It has been brought to SCI's attention that some users of P375 are misinterpreting which section of the document is relevant to the design of external steelwork. *Section 3.4.2 Compartment fires – external members* refers to Annex B of BS EN 1991-1-2[1] for the model describing the compartment fire conditions and the flames emanating from openings. The expressions in Section 3.4.2 are used in the calculation of the relevant radiative and connective heat fluxes. Design of external steelwork using these heat fluxes should be based on Annex B of BS EN 1993-1-2[2] as described in para. 4.2.5.4. of the same standard.

Section 3.3.2 External fire curve gives the nominal temperature-time curve intended for the outside of separating external walls as defined in para. 1.5.3.5 of BS EN 1991-1-2 and presented in para. 3.2.2 of the same standard. It is not intended for use in the design of external steel members.

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1. BS EN 1991-1-2:2002 *Eurocode 1: Actions on structures Part 1-2: General actions – Actions on structures exposed to fire*
2. BS EN 1993-1-2:2005 *Eurocode 3: Design of steel structures – Part 1-2: General rules – Structural fire design*

AD 443: The use of fully threaded bolts

SCI has been surprised to hear of the use of fully threaded bolts being questioned, as these have been in common use – and have been the standard bolt used – for very many years.

The potential advantage of partially threaded bolts is that they obviously have a slightly higher shear resistance if the shear plane is in the unthreaded length. The disadvantages of calculating precise unthreaded lengths, which must be neither too long nor too short, and relating each bolt length to specific connections, far outweigh the increased resistance. On site, multitudinous bags of different bolt lengths give ample opportunity to install the wrong bolts. In contrast, a standard M20 x 60 mm fully threaded bolt may be used in the vast majority of site connections.

The use of fully threaded bolts was recommended in the first "Green Book" of 2002[1] and the Eurocode version of 2014[2].

Concerns with fully threaded bolts may relate to the supposed increased in bearing deformation, if the threads engage with the steel rather than the unthreaded shank. Investigations of the behaviour of fully threaded bolts were reported by Graham Owens in 1992[3]. Although fully threaded bolts in bearing show a lower initial stiffness, the bearing strength actually increases slightly, due to the constraint offered when the threads dig into the plate material. The deformation in bearing of a fully threaded bolt is slightly more than that of a plain shank, but the increase is not relevant when bolts are already in 2 mm oversize holes.

If designers are concerned about deformation in a joint, the issue does not concern whether fully threaded or unthreaded bolts are specified – the difference in performance is insignificant. If deformation in the joint must be avoided, preloaded assemblies must be specified.

It should be noted that shear and tension resistances quoted (in the Blue Book, for example) always use the cross section in the threaded length as the basis of the resistance calculations – and are therefore safe.

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1. *Joints in steel construction. Simple connections* (P212), SCI and BCSA, 2002
2. *Joints in steel construction. Simple joints to Eurocode 3* (P358), SCI and BCSA, 2014
3. Owens, G, W., *The use of fully threaded bolts for connections in structural steelwork for buildings.* The Structural Engineer, Volume 70, September 1992

AD 447: Openings in composite slabs

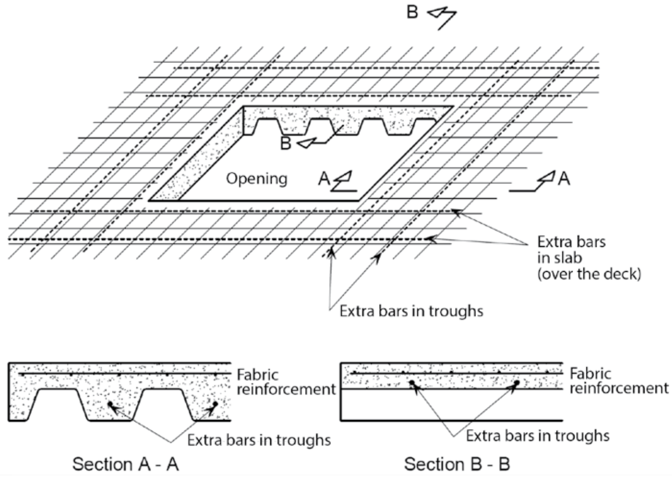

Figure 1: Beam strips around a medium sized opening

It is now over ten years since the revised edition of P300 was published by SCI. This work, in collaboration with the Metal Cladding and Roofing Manufacturers' Association (MCRMA), covered best practice for the design and construction of composite beams and slabs. It benefitted from considerable practitioner input from the members of the MCRMA's now disbanded Decking Group, remains widely referenced and is mostly still applicable.

One perennial problem with anything composite is that other aspects of a building, such as the need to accommodate services, often result in an inconvenient desire to cut holes in structural concrete (and composite) slabs. In P300 we collated what individual decking manufacturers were saying in their literature in order to provide guidance on how to deal, structurally, with small, medium and large openings:

Small - openings up to 300 mm square. Unlikely to present a problem structurally and do not normally require additional reinforcement.
Medium - openings between 300 mm and 700 mm square. Normally require additional reinforcement to be placed in the slab (see Figure 1, which is taken from P300). This is also the case if the openings are placed close together.
Large - openings greater than 700 mm square. Should be trimmed with additional permanent steelwork back to the support beams.

Two aspects of this guidance are worthy of further consideration, namely what is the critical dimension, and how to deal with openings which are placed close together.

The critical dimension of an opening

Although the guidance given in P300 refers to square openings, the dimensional limits actually need only apply to the width of the opening (perpendicular to the direction of span of the slab). This is because they are based on the ability of the slab, without additional measures for small openings and with additional measures for anything larger, to transfer self-weight and loads transversally between ribs. A small opening could be over one metre long, so long as it wasn't more than 300 mm wide.

It is also worth adding that although 300 mm is provided as general guidance, for the unusual (in the UK) case of a slab with extra bars in the troughs, their positioning relative to the opening needs to be considered. A 300 mm wide opening could very easily 'interrupt' a bar in a trough. Such interruption would need to be compensated for by placing additional longitudinal bars in the adjacent troughs using the beam-strip model adopted for medium-sized openings.

For medium-sized openings it is also worth remembering that some of the reinforcement in the beam-strips will be relatively susceptible to fire. Bars in troughs may have sufficient concrete cover to keep them cool, but bars (and fabric) in the slab between the ribs will become hot and loose considerable strength. Fire protection may be needed to ensure that the beam-strips retain their integrity in a fire.

Multiple openings

In some situations with multiple small or medium-sized openings it will not be possible to accommodate beam-strips between adjacent openings (with or without supplementary reinforcement) to carry the additional loads around the opening. They should then be treated as one (larger) effective opening. Beam-strips are designed using the same philosophy around, and potentially within (to pick up any local areas of otherwise unsupported slab), this larger area.

Health and safety and site practice

The above considerations only concern the structural ability of the slab. Of course, attention must be paid to some form of protection when there is any kind of opening, to avoid a potential hazard on site.

And finally, as noted in P300, small and medium-sized holes in the deck should not be cut until after the concrete around the opening has cured.

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AD 448: Support to profiled steel decking

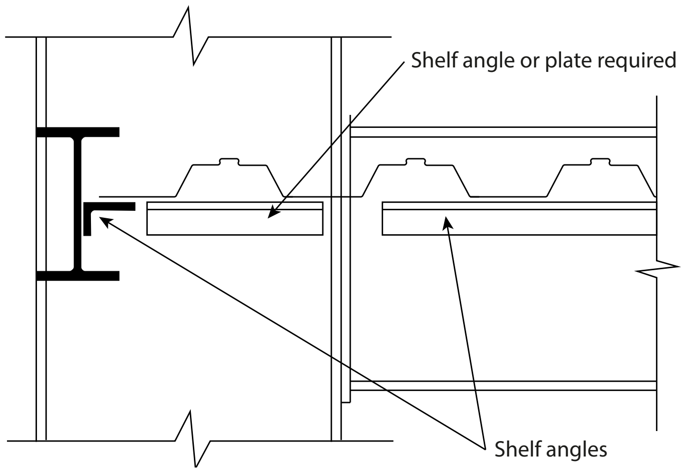

Figure 1: Decking penetrated by a 'wide' column and requiring additional end support

It is now over ten years since the revised edition of P300 was published by SCI. This work, in collaboration with the Metal Cladding and Roofing Manufacturers' Association (MCRMA), benefitted from considerable practitioner input. Since the demise of the MCRMA's decking group, more recent practitioner comment on updating the content has come from BCSA's Cold Formed and Metal Decking Group.

One area where the BCSA Group felt it was worth adding some more detail concerns the support provided to steel decking around penetrations, and at ends and edges.

Support around a penetration

The guidance in P300 says that flashing should not be used to support decking around penetrations. Such supports should be provided by shelf angles or similar. The BCSA Group confirms this approach, and notes that the need for shelf angles should be identified at the design stage. This will mean they can be included as part of the shop fabrication and thereby fixed in a controlled environment rather than on-site and potentially working at height.

P300 states that such (structural) support should be provided when the decking is penetrated by a column resulting in a deck edge dimension in excess of 250 mm with no beam underneath to provide support. Figure 1 is taken from P300, showing a column with a 'width' in excess of 250 mm and the decking around this column therefore requiring end support from an angle fixed to the column web. The figure also shows shelf angles providing end support to the decking abutting the beam framing in to the column flange, and edge support to the decking abutting the beam framing in to the column web.

The BCSA Group has added some detail to this requirement, based on the fact that decking is effectively one-way spanning and so noting that:

- Up to 250 mm is acceptable as a structurally unsupported length along the edge of decking
- At the ends of the decking this critical dimension should be reduced to 50 mm

End and edge supports

The BCSA Group confirms that the guidance given in P300, namely that shelf angles should be used, remains current. Also, as for support around penetrations, that the shelf angles are identified during design and included as part of the shop fabrication.

When a soffit is exposed, and so aesthetics are important, where practical continuous support should be provided to all ends and edges.

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AD 450: Resistance of composite slabs to concentrated loads

EN 1994-1-1[1] clause 9.4.3 is entitled *effective width of composite slabs under concentrated point and line loads*. It has been the cause of much confusion, as explained below. We are now confident about our interpretation of this clause, and in particular the limits of 7.5 kN and 5.0 kN/m^2 quoted in its part 5).

The purpose of EN 1994-1-1 9.4.3

For design purposes composite slabs are, not unreasonably, assumed to be one-way spanning. Span is in the direction of the ribs, which add significantly to the depth of the slab and make its stiffness in this direction considerably greater than its transverse stiffness. A question that then arises is what width of slab can be assumed to be active in supporting a concentrated load?

A typical composite slab might span 3.5 m, and could be anything from 6 m to 12 m or more 'wide' (i.e. transverse to the assumed spanning direction). Clause 9.4.3 tells the designer how much of this width can be assumed to carry a concentrated load, acting as a beam. Figure 1 below is taken from EN 1994-1-1:

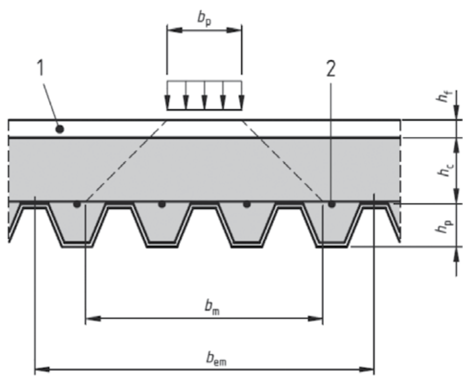

Figure 1: Widths associated with a concentrated load (1 indicates topping)

A load with a physical width b_p distributes at 45 degrees through the depth of slab (and any finishes) above the decking. It then distributes further, to a total width b_{em}, which is the width of slab assumed to carry the load (acting as a beam). The total width b_{em} is a function of the span type (internal or end), the load position within the span, and what physical behaviour is being verified (bending moment and longitudinal shear, or vertical shear resistance). Reference should be made to EN 1994-1-1 equations 9.2, 9.3 and 9.4.

The need for the limits given in 9.4.3 (5)

In 5) of this clause it is noted that 'nominal transverse reinforcement may be used without calculation' (i.e. assumed to be adequate) provided the following maxima are not exceeded for the 'characteristic imposed loads':

- Concentrated load 7.5 kN
- Distributed load 5.0 kN/m²

It is worth noting that, although EN 1994 1-1 clearly states these are limits for imposed loads, given the purpose of this clause other types of point and line loads should also be included in the verification.

It has long been assumed by many – including ourselves in P359 – that the inclusion of a 'squared' in the second of these limits was a 'typo', given that clause 9.4.3 concerns itself with point and line loads (not distributed loads). The wording in ECCS publication 087 (dated 1995) Design Manual for Composite Slabs[2] seemed to confirm this assumption. Some software has also, conservatively, misinterpreted this clause – for example using the defined contact area of a point load to determine a value per metre squared, to check against the second criterion.

The key to understanding what 5) is about is to consider the context. As noted above, it falls within a section of EN 1994-1-1 concerned with calculating the effective width of slab that may be assumed to support a concentrated load. That part of the width b_{em} that goes beyond b_m is a function of the transverse slab stiffnesses, and the definitions of b_{em} given in EN 1994-1-1 are for a typical slab. A slab that was subject to a very high concentrated load might not be typical – it could be designed to be appropriately strong and stiff in the direction of the ribs (its assumed span direction), but might then be relatively more flexible than 'typical' in the transverse direction (for which no explicit design is normally carried out). That relative flexibility would result in the concentrated load being carried over a narrower strip of slab.

So the intent of checking against the two limits defined in 5) is to ensure that the slab is not subject to excessive concentrated loads, so that it remains 'typical'. To do this the designer should consider all the loads on a given area of slab (between the supporting beams on all four sides), be they UDL, point loads or line loads, and check that:

- The heaviest concentrated load does not exceed 7.5 kN
- The sum of all the loads divided by the area of slab does not exceed 5.0 kN/m²

Unless both of these criteria are satisfied the slab should be designed considering the effects of transverse bending moments under the concentrated loads, with appropriate transverse reinforcement provided (see below). Alternatively, the effective width could be limited to b_m, so that no transverse distribution is assumed (or transverse slab stiffness needed). This option was explicitly stated in the ENV (so-called pre-standard) version of Eurocode 4[3].

It is important to recognise that these are 'rule of thumb' limits, so particularly unusual situations are worthy of more detailed analysis. For example, a combination of small UDL combined with a significant line load (the sum of which satisfied the 5.0 kN/m² limit), would result in very different behaviour from a large UDL combined with a small line load (also less than 5.0 kN/m²). The former situation would place greater demands on the ability of the slab to distribute load effects transversely. To avoid such situations a third limit that line loads should not exceed 5.0 kN/m was proposed in ECCS 087[2]. An alternative line load limit is given in Reference [5].

The fact that the UDL limit of 5.0 kN/m² does not allow significant concentrated loads to be supported in addition to the uniformly distributed loads typically present, is an indication that composite slabs are not well suited to carrying large concentrated loads.

Designing the slab for transverse bending

As noted above, if the stated load limits are exceeded then the slab must be designed explicitly for transverse bending, and appropriate transverse reinforcement provided. Whereas EN 1994-1-1 9.4.3(6) simply gives a general reference to EN 1992-1-1[4] for guidance, Reference [5] proposes a simple way of determining the transverse bending moment that can then be used in the standard design of a reinforced concrete beam strip that passes under the load.

By analogy with the load width b_m, the load length a_m is assumed to be given by:

$$a_m = a_p + 2(h_f + h_c)$$

Where h_f and h_c are the thickness of any finishes and depth of concrete above the deck, respectively, and a_p is the contact length of the load.

The transverse bending moment due to the load Q_{Ed}, per metre length (in the direction of the slab span) is then given by:

$$M_{Ed} = \frac{Q_{Ed}\,(b_{em}-b_m)}{8 \cdot a_m}$$

As a footnote it is worth remembering that software tends to consider one metre wide strips of slab – there is no facility to input the width of slab. Some post-processing of outputs in order to verify compliance with this clause may therefore be necessary.

References:

[1] BS EN 1994-1-1:2005, Eurocode 4 - Design of composite steel and concrete structures - Part 1-1: General rules and rules for buildings, BSI, 2005.

[2] ECCS 087 – Design Manual for Composite Slabs; Technical Committee 7 - Cold Formed Thin Walled Sheet Steel; Technical Working Group 7.6 - Composite Slabs, 1995.

[3] DD ENV 1994-1-1:1994, Eurocode 4. Design of composite steel and concrete structures. General rules and rules for buildings (together with United Kingdom National Application Document), BSI, 1994.

[4] BS EN 1992-1-1:2004+A1:2014, Eurocode 2: Design of concrete structures. General rules and rules for buildings, BSI 2004.

[5] Johnson, R. P, Wang, Y. C., Composite Structures of Steel and Concrete, Fourth edition, 2019; Wiley Blackwell.

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AD 453: Accumulated deviations in erected steelwork

Questions about tolerances continue to arrive at SCI's Advisory Desk – often concerning the potential to sum all the possible deviations to reach a (usually large) tolerance on the final position of a component.

The suggestion is that, (for example) the base of a column can be out of position, and the column can be out-of-plumb, and the connections for a façade beam can be out of position, and the beam itself can have a lack of straightness. Combine that situation with some fabricated bracket (with its own set of tolerances) connected to the beam and the potential for a large deviation at measured locations is obvious.

The National Structural Steelwork Specification (NSSS), which is now in its 7th Edition, deals with this by adopting a "root sum of the squares" approach. The accumulated sum of several independent sources of deviation (Δ1, Δ2, Δ3 etc) is given by:

$$\Delta_{sum} = \sqrt{\Delta_1^{\,2} + \Delta_2^{\,2} + \Delta_3^{\,2}\ \text{etc}}$$

SCI advice is that when certain locations are critical (usually at interfaces with other components), it is much better to build in provision for adjustment than to argue about tolerances later.

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