

NSC

alls between the
n with increased resistance
ablished approach in BS 5950,
ude a basis for using an increased modulus
sections but this omission is now to be
B in prEN 1993-1-1 now includes the
to-plastic section modulus for semi-compact
ions and circular or elliptical hollow sections.

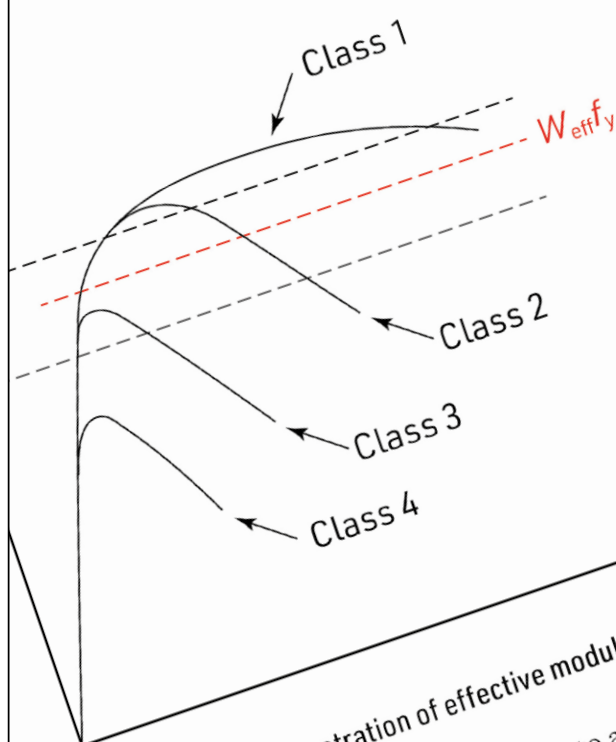


Figure 1: Illustration of effective modulus

The Annex also takes the effect of axial force into account but in this case an adjusted bending resistance is determined by applying a factor to the resistance in the absence of axial force, calculated using the elasto-plastic modulus. The factor depends on the ratio of the design axial force to the plastic axial resistance.

Detailed Proposals in Annex B

... have been examined in detail for the case of symmetrical ... formulae for the revised proposals are as indicated ... the plastic and

... compression, and the
slenderness ratios at the bound
ed from $c/t \leq 33\epsilon$ to $c/t \leq 28\epsilon$.
expressions for rolled or welded I or H sections are as follows:

$$\beta_{ep,y} = \text{Max} \left\{ \frac{c - 10\epsilon}{t_f}; \frac{c - 83\epsilon}{38\epsilon}; 0 \right\} \text{ but } \beta_{ep,y} \leq 1.0$$

$$\beta_{ep,z} = \text{Max} \left\{ \frac{c - 10\epsilon}{6\epsilon}; 0 \right\} \text{ but } \beta_{ep,z} \leq 1.0$$

In the first expression, 10ϵ is the limiting slenderness for class 2
outstand flanges subject to bending; 83ϵ is the limiting slenderness
for internal compression parts subject to bending and 38ϵ is the li
slenderness for class 3 for internal compression parts subject to

2.1 Resistance of cross-sections

The resistance of cross-sections that are required to sustain c
bending and axial compression is established by determining
elasto-plastic bending resistance reduced in the presence of
section classification is established first, followed by the e
modulus. The bending resistance is then found and reduc
ratio of the design compression force to the plastic com
the section.
The process is illustrated below in an example and
with the equivalent outcome determined using the B

3 Example

3.1 prEN 1993-1-1 procedure

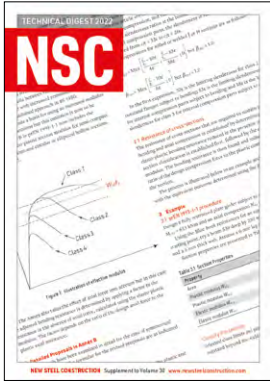
Design a fully restrained plate girder subject to
 $M_{y,Ed} = 615$ kNm and an axial compression $N_{Ed} =$
Using the Blue Book resistances for an equ
starting point, try a beam 530 deep by 230 w
and a 6 mm thick web. Assume a 6 mm leg y
Section properties are presented in Tab

Table 3.1 Section Properties

Property
Area
Plastic modulus $W_{pl,y}$
Plastic modulus $W_{pl,z}$
Elastic modulus $W_{el,y}$
Elastic modulus $W_{el,z}$

Classify the section

Selected class limits are pre
outstand beyond the weld



NSC

**TECHNICAL
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Essential reading for net-zero carbon designers



Nick Barrett - Editor

This is the seventh 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 Technical Digest has claimed a place on the essential reading section of the digital 'bookshelves' of architects and engineers. The Digest brings together all the Advisory Desk Notes and Technical Articles published in NSC in the previous year and is available as a free downloadable pdf at the steelconstruction.info website, or for online viewing.

The Digest is part of the steel construction sector's long-established commitment 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, which is more important than ever as the construction industry gets fully behind the drive to net-zero carbon.

Design guidance and other key steel construction information including details of how the steel construction sector is supporting the drive towards net-zero carbon 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 popular Advisory Desk Notes and longer Technical Articles from the steel sector's own experts – that are included in the Technical Digest – are first published, and immediately made available on newsteelconstruction.com.

Advisory Desk Notes keep designers abreast of developments in technical standards. Some of them are provided following questions being asked of the sector's technical advisers and 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 most efficient and sustainable steel construction projects. These articles can be in response to legislative changes or changes to codes and standards. Technical updates 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.



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Wind actions on single storey buildings

David Brown of the SCI comments on some of the issues frequently raised when determining the loading on this common form of construction.

Single storey buildings

Portal frame buildings and other single storey structures are said to account for around 45% of the structural steel used in the UK. Considering the steel tonnage used in a single multi-storey building, there is obviously considerable demand for industrial, retail, storage and distribution buildings. “Single storey” may be an inappropriate description as some storage and distribution structures are equivalent in height to three or four-storey offices. This article covers some of the questions on BS EN 1991-1-4 which arrive at the SCI’s Advisory Desk.

First, the pressure

Who would ever want to undertake calculating this by hand? Many companies who manufacture purlins and side rails provide software to assist in the design and selection of appropriate members. This software will always need to calculate the peak velocity pressure q_p and will usually present the information for each 30° segment around the site. At a stroke, the heartache of working through the standard and the UK National Annex has been bypassed.

Some warnings are however necessary. Several programs use “BREVe” to determine the wind pressure, a component which has been around for some time, leading to some compatibility issues with operating systems. Users will generally be presented with a table of intermediate results, inviting the user to modify the assumed values. Although it may be tempting to simply accept the table, users really should ensure they are content with the presented values. The important values to check and adjust if necessary are:

- Site altitude
- Distance from the sea (or significant inland water)
- Distance from edge of town
- If in town, the average obstruction height and spacing (h_{ave} and x respectively, from A.5 of BS EN 1991-1-4)

Since the underlying data has a certain granularity (for example, altitude might be anywhere within the surrounding 2 km) users should expect some odd values if comparing with OS maps. As the database reflects a point in time, subsequent urbanisation may have an impact on the assumed values.

The default values for h_{ave} and x may be 6 m and 20 m respectively. The value of 20 m was suggested in BRE Digest 436, Part 1, from 1999, which gave guidance on BS 6399-2. The 1999 version of the digest is not readily available, having been completely updated to reflect the Eurocode. The value of 6 m may reflect an assumed two-storey shelter height. Local knowledge is essential to determine the correct values. It is assumed that “irreversible urbanisation” will mean that shelter only increases, which seems optimistic in reality.

Peak velocity pressure without software

Manual calculation is of course possible, though for the author, not desirable. Calculations could consider the same 12 segments as software. The SCI recommendation is to consider four 90° quadrants and determine the most onerous values of the various factors in each quadrant. The peak velocity pressure would then be the most onerous of the four. This “by quadrants” method generally gives reasonable results compared with considering twelve segments, and avoids the significant conservatism of taking the most onerous value from anywhere around the site and assuming these most onerous values all apply to wind blowing from one direction.

Whichever approach is followed, the assessment to this stage has only considered the site – the orientation of the building is not yet relevant. In most cases it is not necessary to know the building orientation, unless there is particular benefit in calculating a different pressure for each face. It would be unusual to have different side rails on different faces of the structure (for example). Pressures on individual faces may be important if the building is not symmetric, there is some ground feature affecting one side only or there are openings on one face only. If pressure on an individual face is important, the pressure on a face must be determined considering a range of directions $\pm 45^\circ$ from the normal to each face as shown in Figure 1, not just the direction perpendicular to each face – the full 360° around the site must be included.

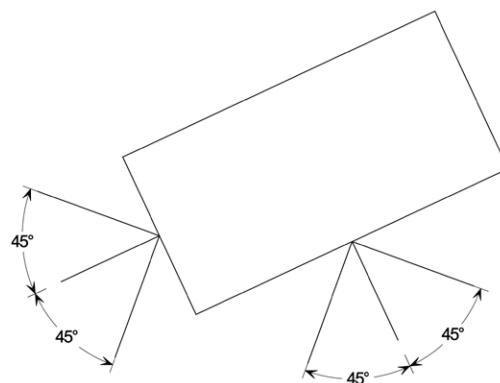


Figure 1: Wind pressure on individual faces

Designers undertaking manual calculations will need to interrogate figures NA.7 (reproduced below) and (if in Town) NA.8. SCI is not aware of any expressions which define the curves in these figures. Some time ago, csv files were available via IStructE, which could, with some thoughtful interpolation, be used to determine a value, but these are no longer available.

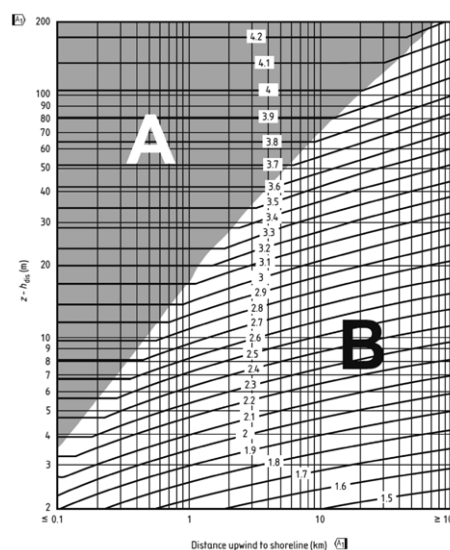


Figure 2; Figure NA.7 (from the UK NA to BS EN 1991-1-4)

A helpful free tool is available to download from www.rwdimedia.com/encalculator_program.html which will enable precise values to be determined.

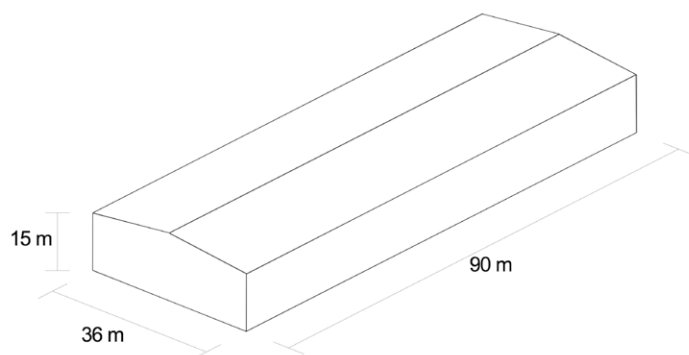


Figure 3: Example building

Internal pressure coefficients

Mature designers will immediately recognise two values, of +0.2 or -0.3. These values appeared in Appendix E of CP3:Chapter V from 1972 and Table 16 of BS 6399-2 from 1997. The same values appear in Note 2 to clause 7.2.9(6) of BS EN 1991-1-4, presented as an option if the designer is unable to, or does not wish to, calculate the precise value based on building geometry and opening ratio.

BRE Digest 436 Part 1 from 1999 offered further guidance that “the positive value $C_{pi} = +0.2$ is now less likely to be a critical design case. The positive value can only occur when the side walls are impermeable and the front face is permeable”. The Digest also advised that “The internal pressure coefficient for completely clad enclosed warehouse-type buildings without opening windows, may be taken as $C_{pi} = -0.3$.”

The reference to opening windows relates to a dominant opening, discussed later. After BRE Digest 436 was published, many designers of “warehouse-type” buildings changed their practice to only consider a coefficient of -0.3, though some considered an additional coefficient of zero as a replacement to the +0.2.

Generally, it is advantageous to calculate the actual internal pressure coefficient, using Figure 7.13 and expression 7.3 from the Eurocode. If it is reasonable to assume that the roof is impermeable and the elevations equally permeable, the calculation is simply based on area.

With a 15 m tall portal frame building, 36 m span and 90 m length (Figure 3), the wind may blow parallel or perpendicular to the ridge.

Wind parallel to the ridge

Area of elevations with suction = $2 \times (90 \times 15) + (36 \times 15) = 3240 \text{ m}^2$

Area of all elevations = $3240 + (36 \times 15) = 3780 \text{ m}^2$

$$\mu = \frac{3240}{3780} = 0.86$$

For Figure 7.13 (reproduced as Figure 4), $h/d = 15/90 = 0.17$

From Figure 7.13, $C_{pi} = -0.25$

Wind perpendicular to the ridge

Area of elevations with suction = $(90 \times 15) + 2 \times (36 \times 15) = 2430 \text{ m}^2$

Area of all elevations = 3780 m^2

$$\mu = \frac{2430}{3780} = 0.64$$

For Figure 7.13 (reproduced as Figure 4), $h/d = 15/36 = 0.42$

From Figure 7.13, with interpolation, $C_{pi} = -0.02$

Particularly for wind blowing perpendicular to the ridge, this is a considerable improvement from the use of $C_{pi} = -0.3$. Only unusual geometry will result in a positive internal pressure, supporting the advice in the BRE Digest.

Dominant openings

CP3:Chapter V has the guidance that C_{pi} should be taken as the more onerous of +0.2 and -0.3 “when there is only a negligible probability of a dominant opening occurring during a severe storm”. It is not clear how this should be

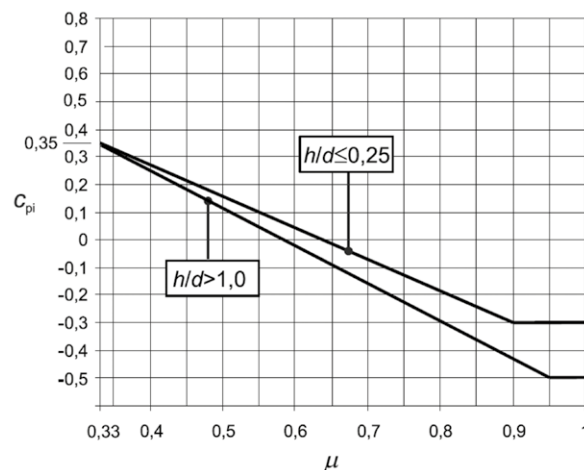


Figure 4: Figure 7.13 from BS EN 1991-1-4

assessed. It would be expected that certain structures, such as fire stations or lifeboat buildings might well have open doors in a severe storm. Perhaps a 24-hour distribution warehouse might also need to continue operations. Allowing for a dominant opening will serve to inflate or deflate the building and will probably increase the required member sizes since C_{pi} is up to 90% of C_{pe} at the location of the opening. In the author’s experience from previous decades, tenders might be qualified in small text that “it has been assumed that in the event of a severe storm, all openings will be shut”, which was code for “we have not allowed for dominant openings”.

The Eurocode does not allow this practice – clause 7.2.9(3) insists that if openings that would be dominant are assumed to be shut, the condition with the door or window open should be considered. Fortunately, the Eurocode also specifies this as an accidental combination of actions, meaning that equation 6.11b from EN 1990 should be used to verify this case. The actions are unfactored in the accidental combination, which may mean that the original member sizes remain satisfactory. An opening does not need to be large to be dominant – advice is given in clause 7.2.9(4) of BS EN 1991-1-4.

Note that zone A (just around the corner from the windward face) has a more onerous coefficient than zone E, the leeward face, as shown in Figure 5. A dominant opening in zone A could be particularly onerous.

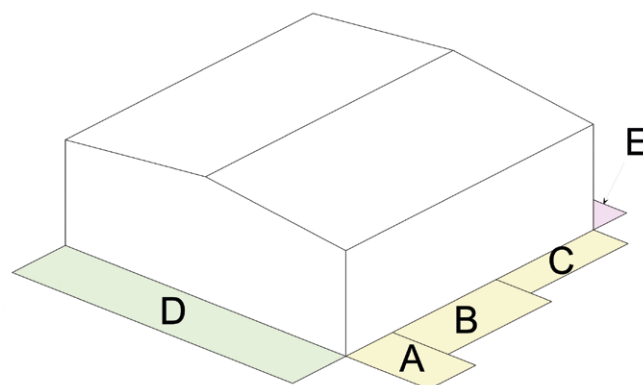


Figure 5: Wall coefficients

Local external pressure coefficients

Where wind flows around a corner, or over the eaves, or over the ridge, increased turbulence leads to higher local suctions. CP3: Chapter V presented coefficients for these zones, but noted in clause 7.2 that “they should not be used for calculating the load on entire structural elements such as roof walls or the structure as a whole”. This gave rise to the widespread practice (at that time) of neglecting the local zones for the design of the structure. There is no permission to ignore the local zones in BS 6399-2 or BS EN 1991-1-4.

Further resources

SCI publication P394 contains comprehensive advice on the application of BS EN 1991-1-4 and a worked example. The three parts of both the 1999 and 2015 versions of BRE Digest 436 contain helpful guidance and are recommended reading. ■

Portal frames with flexible joints using Kleinlogel-type formulae

Single-span portal frames with flexible joints can be analysed using formulae similar to those produced by Kleinlogel. Concept designs or "hand" checks of detailed designs can be carried out using such methods. Richard Henderson of the SCI discusses the background.

Introduction

Adolf Kleinlogel produced a book¹ containing formulae setting out the internal bending moments and direct forces in rigid jointed frames subject to different load cases. A selection of these for single storey gable and pitched portal frames have been reproduced in the Steel Designer's Manual (SDM) for several editions. Their use has probably dwindled over the years due to the promotion of additional economy using plastic design and the ease of analysis using finite element (FE) software packages. One of the limitations on the usefulness of the formulae published in the SDM is that none are included for frame deflections. The formulae were produced using elastic analysis assuming bending deformations only and can be derived using the slope-deflection equations.

In 2020, two technical articles on the calculation of joint stiffness in steel design^{2,3} were published in *New Steel Construction* magazine. In anticipation of increasing implementation of joint flexibility in frame design, Kleinlogel's formulae have been developed to include joint stiffnesses.

Structural analysis and joint stiffness

The traditional UK approach to structural analysis of frames assumes that joints between members are either perfectly pinned or fully rigid. Members are also assumed to have negligible size compared with the frame geometry. In rigid-jointed frames, the distribution of bending moments and their deflections depends on the flexural stiffness of the elements which are assumed to be axially rigid. In general, if a joint is designed to resist the calculated bending moment, the effect of joint stiffness is assumed to be negligible and this assumption produces reasonable results. A historical exception to this approach where joints designed as pinned for vertical loads were assumed to provide resistance to wind loads was developed into the wind-moment method of design⁴.

If the joints between members in a frame are not fully rigid, the bending moments and deflections are influenced by the joints' rotational and shear stiffnesses. Eurocode 3 Part 8⁵ includes sections on the calculation of the rotational stiffness of joints and their classification by stiffness for use in structural analysis. In the UK, the National Annex advises against the use of semi-continuous elastic design, except where it is supported by test evidence based on satisfactory performance in similar situations. This is because of a lack of confidence in the accuracy of the proposed method of determining joint stiffnesses and, consequently, in the validity of structural modelling when such joint stiffnesses are included. The traditional approach where joints are either assumed to be pinned or fully rigid is advocated.

In due course it is assumed that the models of joint stiffness will be validated by testing. At such a time, the inclusion of the rotational stiffness of joints in analysis models may well be required when considering certain kinds of structure. For example, portal frames built using cold formed sections often include joints that, by their nature, are of low stiffness. Allowance for joint flexibility in this form of structure is essential. Finite element software packages already allow the rotational stiffnesses to be included in an analysis model, but there is no hand method of calculation readily available for concept design of simple frames or for checking the impact of joint stiffness on the frame bending moments and sway deflections.

Slope-deflection equations

Readers of a certain vintage will remember that the slope-deflection (S-D) equations are developed for a beam element and relate the end rotations and relative displacement of the ends to the end moments; they can be derived from consideration of the curvature along the element see Figure 1.

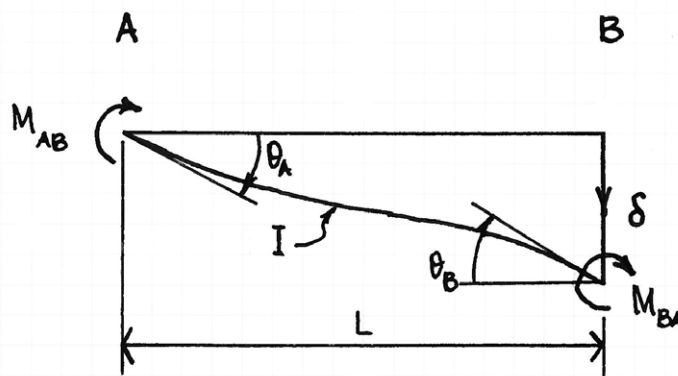


Figure 1: Deformation of beam element

For a beam element A-B, length L , second moment of area I , the equations are:

$$M_{AB} = \frac{2EI}{L} \left[2\theta_A + \theta_B - \frac{3\delta}{L} \right] - M_F$$

$$M_{BA} = \frac{2EI}{L} \left[2\theta_B + \theta_A - \frac{3\delta}{L} \right] + M_F$$

where θ_A is the rotation or slope at end A and δ is the deflection of B relative to A. M_{AB} is the bending moment at A and M_F is any fixed-end moment that may be present for a given load case⁶. Simple statically indeterminate frames can be analysed by using the slope-deflection equations to eliminate the redundancies. At a joint between two elements, the sum of the internal moments derived for each member is zero. So for elements A-B and B-C, joined at B, $M_{BA} + M_{BC} = 0$. The assumption of axially rigid members allows vertical and horizontal deflections at the ends of sloping members to be related to each other.

Modified Kleinlogel Formulae

The introduction of flexible joints means that the end rotations are increased for a given bending moment in a joint normally assumed to be rigid and reduced in a joint normally assumed to be pinned, with a corresponding redistribution of bending moments. This principle is already in use in the analysis and design of portal frames as described in SCI publication P399⁷. In the UK, portal frames are often assumed to have pinned feet in the strength analysis and this approach produces upper-bound bending moments in the columns and rafters.



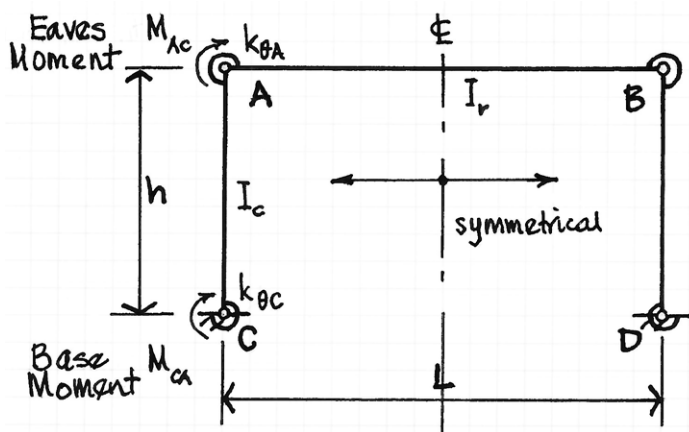


Figure 2: Goal post portal frame

However, when considering sway stability and serviceability, nominal values for the rotational stiffness of bases are usually included and this reduces eaves deflections and global second-order effects.

To avoid complicated formulae with many terms, Kleinlogel introduced simplifying notation for specific frames. For the case of a uniform load w applied to the rafter of a pinned-foot goal post frame of height h and span L (Figure 2), Kleinlogel gives the eaves (rafter to column joint) moments as:

$$M_A = M_B = -\frac{wL^2}{4N}$$

where $N = 2k + 3$ and $k = I_r/I_c \cdot h/L$. The I values are those of the rafter and column, as indicated by the suffices. The same frame with fixed feet has the following eaves moments:

$$M_A = M_B = -\frac{wL^2}{6N_1}$$

The moments at the column feet are half the eaves moment:

$$M_C = M_D = \frac{wL^2}{12N_1}$$

where $N_1 = k + 2$. These formulae provide the “end cases” where the joint stiffnesses are either zero (pinned) or infinite (fixed).

The equivalent formulae to Kleinlogel’s for the goal post portal frame (see Figure 2) with joint stiffness included, carrying a uniform load on the rafter can be determined as follows.

Introducing a rotational spring of stiffness k_θ at the foot of column C-A, with units kNm/radian, where the rotation is θ_c , the bending moment due to the spring is $k_{\theta c} \cdot \theta_c$. At C, the sum of the moment in the column and the moment in the rotational spring is zero or, using the S-D equation with h denoting the column height,

$$k_{\theta c} \theta_c + \frac{2EI_c}{h} \left[2\theta_c + \theta_A - \frac{3\delta}{h} \right] = 0$$

For a symmetrical load case, $\delta = 0$. At the eaves joint, the slope of the rafter A-B at the joint is the sum of the rotations due to the beam stiffness and the spring stiffness. There is no deflection in the rafter because the column elements are axially rigid. Using the S-D equation:

$$\frac{M_{AB}L}{2EI_r} + \frac{M_{AB}}{k_{\theta A}} = [2\theta_A + \theta_B]$$

Solving for the bending moments, at the column feet,

$$M_{CA} = M_{DB} = \frac{wL^2}{12} \cdot \frac{(1 - 2K_C)K_A}{(2 - K_C + kK_A)}$$

At the eaves,

$$M_{AC} = M_{BD} = -\frac{wL^2}{12} \cdot \frac{(2 - K_C)K_A}{(2 - K_C + kK_A)}$$

In these formulae,

$$K_C = \frac{2EI_c}{k_{\theta c}h + 4EI_c}; K_A = \frac{k_{\theta A}h}{k_{\theta A}h + 2kEI_c}$$

The joint stiffnesses $k_{\theta c}$ and $k_{\theta A}$ are those of the column feet and eaves respectively. For a pinned joint, the coefficients K_C and K_A equal 0.5 and 0 respectively; for a rigid joint the corresponding values are 0 and 1. Kleinlogel’s formulae are reproduced when these values are substituted. A formula for eaves deflection under lateral load can also be developed.

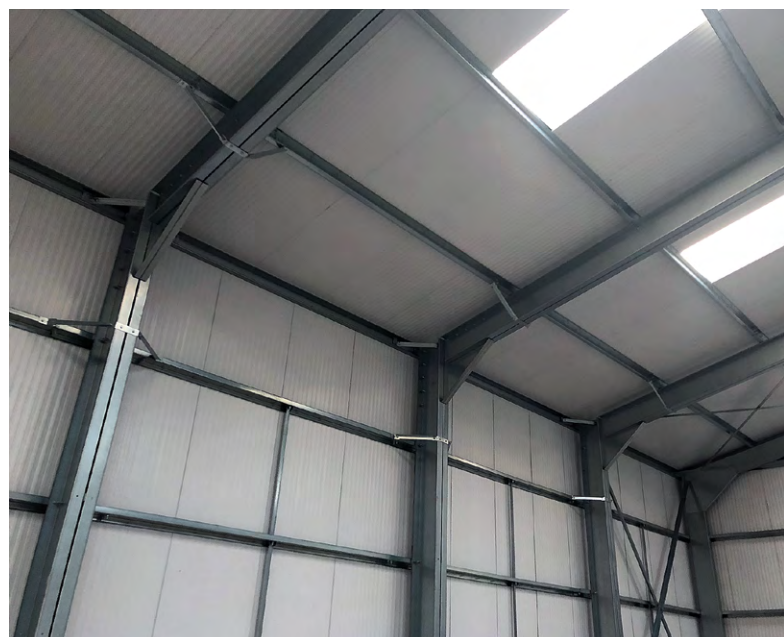
Using this approach, simple frames with members of uniform cross-section can be analysed using the S-D equations with rotational springs at the joints. The analysis gives the same results as a stick FE analysis if the elements are chosen to exclude shear stiffness. The effect of joint stiffness can be investigated and graphs drawn using spreadsheets to show the impact on bending moment distribution around the frame, on the elastic critical load factor and on such serviceability issues as eaves spread.

Conclusions

The slope-deflection equations modified to include joint stiffness can be used effectively as a checking or investigatory tool in the design of simple frames such as goal-post or pitched portals, where joint stiffness is to be included in the design.

In their development, elements are assumed to be uniform in cross section along their length and to exhibit bending deformation only, unlike element formulations in some FE software which also include shear deformation.

In a future article, the effect of varying joint stiffness on characteristics such as frame stability and eaves spread is investigated. Spreadsheet tools to allow rapid calculations are discussed. ■



1. Kleinlogel, A, *Rahmenformeln 11*, Wilhelm Ernst & Sohn
2. Henderson, R, *Joint stiffness calculation*, NSC, February 2020
3. Henderson, R, *Joint stiffness and the elastic critical load factor*, NSC, May 2020
4. Salter, P R, Couchman, G H, Anderson, D, *Wind-moment design of low rise frames (P263)*, SCI, 1999.
5. BS EN 1993-1-8:2005 *Eurocode 3: Design of steel structures - Part 1-8: Design of joints*
6. Marshall, W T, and Nelson, H M, *Structures*, Pitman, 1972
7. Henderson, J R, *Design of steel portal frame buildings to Eurocode 3 (P399)*, SCI, 2015.

Portal Frames with flexible joints

In the second of two technical articles considering the effect of allowing for joint flexibility in the analysis and design of portal frames, Richard Henderson of the SCI presents examples illustrating the effect of such joints on frame stability and structural actions.

Introduction

In the first article presented in the February edition of NSC, the theoretical development of equations similar to Kleinlogel's that allow for joint flexibility was illustrated. In the present article, example frames are considered and the variation with joint stiffness of eaves and apex bending moments, α_{cr} for the frame and eaves spread, are discussed.

Pitched portal frames

The analysis of two frames is presented to illustrate the effects of allowing for joint stiffness, one of which was taken from the example presented in SCI publication P397¹. The characteristics of the frames examined are set out in Table 1 and features of the behaviour of the frames are discussed below. Frame I has columns and rafters formed of rolled UB sections and Frame II is made from pairs of cold-formed lipped channel sections back-to-back. Frame I is not fully representative of a conventional frame because the rafters are assumed to be uniform whereas for economy, rafters are usually haunched near the column joint.

Table 1: Frame characteristics

Item	Frame I	Frame II
Span L (m)	30.0	12.0
Eaves height h (m)	15.0	3.0
Rise r (m)	3.0	1.608
Column	914 × 305 UB 224	Pairs of lipped channels
Column I_{yy} (m ⁴)	3.76×10^{-3}	2.42×10^{-5}
Rafter	533 × 210 UB 101	Pairs of lipped channels
Rafter I_{yy} (m ⁴)	6.15×10^{-4}	2.42×10^{-5}
Uniform load (kN/m)	20.0	6.33

The frames have been analysed assuming fully rigid joints at the eaves and apex and with the base stiffnesses recommended by the SCI for different load cases:

- for frame stability, 10% of the column stiffness (i.e. 10% of $4EI/h$ Nm/radian);
- for serviceability, 20% of the column stiffness;
- for design actions, with fully pinned bases.

The results of the analysis of Frame I is shown in Table 2.

Table 2 Results for rigid joints at eaves and apex

Item	Units	Frame I
α_{cr}	-	7.11
Eaves spread	mm	124
Eaves moment	kNm	1332.1
Apex moment	kNm	-651.4
Column stiffness	MNm/radian	211
20% of column stiffness	MNm/radian	42.2

The frame has been analysed to show the effect on some of the parameters of varying the spring stiffness of the eaves joint while keeping the apex joint stiffness constant. The results have been normalized by varying the rotational stiffness of the joint from a value equal to 1% of the column stiffness to 100% for each frame. The apex joint stiffness has been fixed at 100 MNm/radian. Graphs showing the eaves moment and maximum sagging moment moments are presented in Figure 1. The bases have been assumed to be pinned for this case.

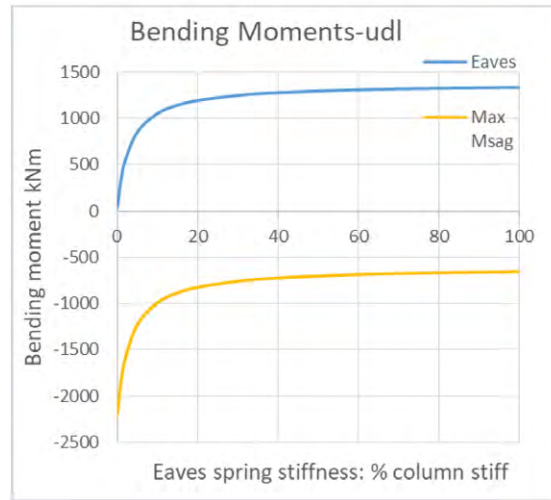


Figure 1 Eaves and maximum sagging moments: Frame I

It can be seen that the bending moments change rapidly over increases in eaves joint stiffness from 1% to 20% of the column stiffness. At 20% of the column stiffness, the eaves moment is 89% of the value at 100% and nearly 90% of the value when both joints are assumed to be rigid.

Figure 2 shows the value of α_{cr} plotted for the same range of eaves joint stiffness. A base stiffness equal to 10% of the column stiffness has been assumed in the analysis. The rafter axial force has been allowed for in calculating α_{cr} as described in Reference 1.

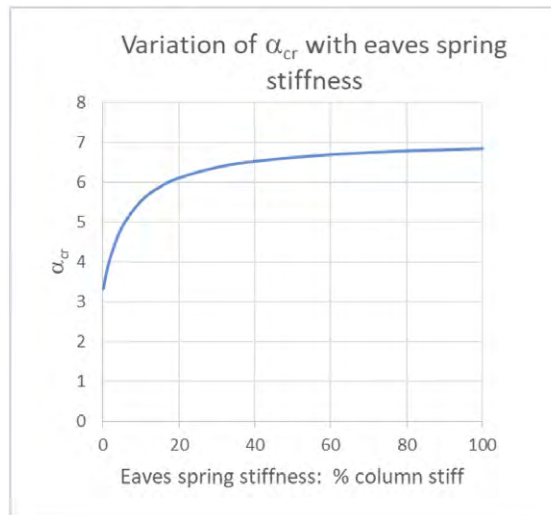


Figure 2 Elastic critical load factor: Frame I

The value of α_{cr} at 20% is 6.1 compared with values at 100% of 6.84 and for fully rigid eaves and apex joints, of 7.11. These values correspond to amplifiers of 1.2, 1.17 and 1.16 respectively.

Figure 3 shows the eaves spread for frame 1 for various eaves stiffnesses and exhibits similar characteristics to the other properties with much of the reduction in eaves spread occurring over the first 20% increase in eaves joint stiffness. The spread reduces to 146 mm for eaves joints with 100% of the column stiffness. The values presented are due to the same factored uniform load as for the other parameters, not an unfactored load corresponding to variable loads only.

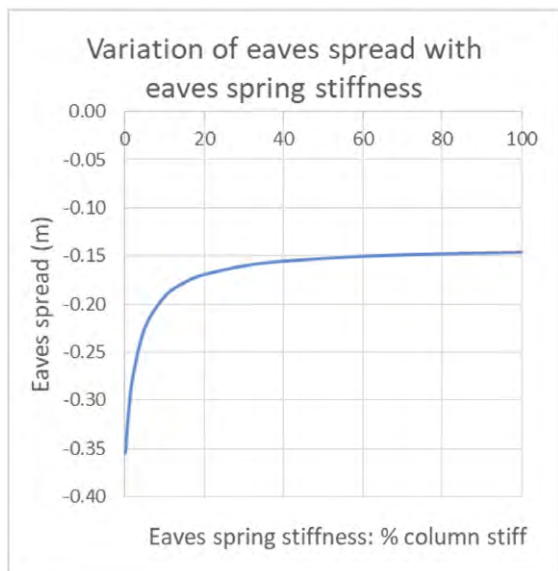


Figure 3 Eaves spread: Frame I

Constant eaves joint stiffness

A similar analysis was carried out keeping the eaves joint stiffness constant and varying the apex joint stiffness from zero to 100% of the column stiffness. This has little effect on the elastic critical load factor if determined (as here) by applying equal lateral loads to both eaves because the apex bending moment is zero. Adopting a spring stiffness at the base of 10% of the column stiffness, in accordance with SCI advice, results in α_{cr} increasing from 6.89 to 7.14. (With a pinned base to the column, the value varies from 3.54 to 3.65). The eaves and apex bending moments do vary with changes in the apex joint stiffness in the vertical load cases. The results are shown in Figure 4. The rafter moments in the vertical load case vary more slowly with changes in the apex joint stiffness than with changes in the eaves joint stiffness. The maximum sagging moment is equal to -542.6 kNm when the joint stiffness is 20% of the column stiffness. The corresponding maximum moment is -662.0 kNm for rigid joints, an increase of 22% of the smaller value.

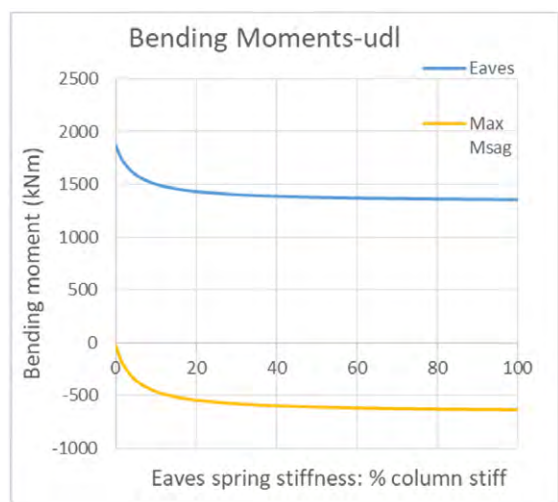


Figure 4 Eaves and maximum sagging moment

Joint Stiffness Estimates

The calculation of joint stiffness in reference 1 was made for a 533 x 210 UB 92, one serial size lighter than the rafter in Frame I. The joint was taken from example C.2 presented in the Green Book for moment connections² and the value determined was 102 MNm/radian. This value is a little under half the column stiffness for Frame I. Joints detailed “efficiently” such that the design resistance is only slightly higher than the design bending moment are subject to a reduction in stiffness to allow for plastic deformation as discussed in reference 2. If an “efficiently” detailed joint in conjunction with a relatively low joint stiffness is initially provided, iteration is likely to be necessary to achieve a suitable α_{cr} value for the frame and adequate joint resistance.

Cold Formed Portal Frames

Portal frames made from cold formed members are used for smaller buildings and in the agricultural sector. Columns and rafter elements are commonly back-to-back lipped channel sections. Rafters are generally not haunched or provided with a knee brace and joints are effected using gusset plates between the pairs of channels, bolted through the channel webs. The gusset plates can also be flanged and bolted to the channel flanges. The joints in such frames are less stiff than those used for rolled sections where end-plate connections are adopted, particularly if the gusset plates are unflanged.

In Frame II, as indicated in Table 1, the same elements have been adopted for both column and rafter and take the form of lipped, back-to-back channels 261 mm deep with 76 mm wide flanges. The material thickness is 2.86 mm (2.9 mm galvanized). A similar analysis to that on Frame I has been carried out on Frame II. The joints consist of gusset plates between the channel webs connected with through bolts, with estimated stiffnesses of 2.0 MNm/radian and 1.8 MNm/radian at eaves and apex respectively. The results are shown in Table 3.

Item	Units	Frame II	
		Rigid Joints	Flexible Joints
α_{cr}	-	16.2	11.8
Eaves spread	mm	33	61
Eaves moment	kNm	56.6	54.1
Apex moment	kNm	-27.1	-30.9
Column stiffness	MNm/radian	6.77	6.77
Rafter axial load	kN	19.5	18.7
Base shear	kN	-18.9	-18.0

Table 3 Results for rigid joints at eaves and apex

The results show that the joint stiffness can have a profound effect on the stability of the frame and on the deflections at the eaves. In the example shown, when the joint stiffness is included in the analysis, the α_{cr} value is reduced by 27% and the eaves spread is almost doubled, relative to the values for rigid joints.

Conclusions

The slope-deflection equations modified to include joint stiffness can be used effectively as a checking or investigatory tool in the design of simple frames such as goal-post or pitched portals, where joint stiffness is to be included in the design.

In the development, elements are assumed to be uniform in cross section along their length and to exhibit bending deformation only, unlike element formulations in some FE software which also include shear deformation.

In the examples considered, joint stiffnesses of at least 20% of the column stiffness produce joint moments of about 90% of the value for fully rigid joints. The α_{cr} value for the same column stiffness is also approximately 90% of the rigid joint value.

If it is possible to achieve joint stiffnesses of at least the magnitude estimated in reference 1, i.e. about 100 MNm/radian, their effect on the characteristics of conventional portal frames made from hot rolled sections should not result in significantly different element sizes or frame behaviour.

For certain types of frame and joints, such as cold formed portal frames with gusset plate joints, the inclusion of joint stiffnesses in the design calculations could have profound implications³ ■

1. Koschmidder, D M, and Brown, D G, *Elastic design of single span steel portal frame buildings to Eurocode 3 (P397)*, SCI, 2012.
2. *Joints in steel construction. Moment-resisting joints to Eurocode 3 (P398)* SCI and BCSA, 2013.
3. Brown, D G, *Design of cold-formed portal frames*, NSC, March 2015.

The use of semi-continuous joints to reduce steel weight and cost

Graham Couchman of the SCI revisits a publication from 1997 which, as we become increasingly concerned with embodied carbon, and therefore efficient use of materials, may be more relevant now than it has ever been. The design guidance offers the potential to significantly decrease beam weights, with no increases in joint complexity and cost, and only marginal increases in frame analysis and design complexity.

Introduction

Has the time finally come for semi-continuous frames? BCSA's *UK structural steelwork: 2050 decarbonisation roadmap*¹, produced with input from SCI and launched at the end of 2021, identified 'design efficiency' as one of the key ways for steel construction to achieve its targets. Design's estimated potential to contribute 18% of the 'change' needed is clearly very significant, and much of that potential is available to be exploited right now. SCI's guide to *Semi-continuous Braced Frames* (P183)² was published in 1997, and remains entirely valid. It was written at a time when SCI was focused primarily on 'structural engineering', and had only one or two experts working on 'sustainability'. Today these previously separate disciplines have effectively merged as far as much of the guidance and advice SCI offers is concerned, with a major focus of the steelwork industry in the UK now being 'sustainable structural engineering'.

Reducing the weight of steel by 18% may sound a lot, but Prof Julian Allwood and his team at the University of Cambridge have published a number of papers³, suggesting that UK steel construction designers use nearly twice as much steel as necessary in order to satisfy design codes and regulations. Clearly this number depends on the building type, and would not apply to single storey industrial buildings. Much overdesign is down to non-technical issues, including low design fees, and the order of the design and procurement process (which means some loads are unknown at the time of design so blanket values are used, and designers are tempted to include some 'bunce' to accommodate subsequent changes of specification without needing to re-design).

There are two obvious results of low design fees, which are paid without recognising the correlation between the time spent on design and the 'quality' of the outcome. These results are, designing the most heavily loaded element

of its type (e.g. a beam in a floor zone) and using the same size for other elements, and making the assumption that the joints are nominally-pinned. The latter choice makes the frame design simple in two senses of the word – easy to do (because there is no stiffness continuity between members), and 'simple' as such frames are known. Simple design is also chosen because it is recognised that much of the fabrication cost of a frame is associated with the joints, so complexities such as stiffeners and haunches are best avoided. The common split of responsibility between design of the beams and columns, and design of the joints, is also unhelpful if trying to develop a solution that optimises the member and joint 'sizes' and recognises the interaction between them. Assumed zero moments in the joints of a 'simple frame' mean that sagging moments in the simply-supported beams are maximised for a given level of loading. Rigid joints, used to produce continuous construction, are not used in braced frames unless exceptional circumstances dictate.

Semi-continuity offers many of the benefits of continuity, in terms of reducing beam sagging moments, deflections and therefore sizes, whilst retaining most/all of the benefits of simple construction in terms of fabrication cost and erection ease. This is because many so-called pinned joints, perhaps with some 'thickening' of components, do in fact offer reasonable stiffness and strength. Stiffeners and haunches are not needed to achieve very beneficial stiffness and strength – semi-rigidity and partial strength respectively, to use terminology from EN 1993-1-8. Figure 1 shows a schematic representation of the moment-rotation behaviour of a semi-continuous joint, illustrating the characteristics of stiffness, strength and (of equal importance, but often forgotten and very difficult to accurately predict) rotation capacity. The stiffness reduces from an initial elastic value, through ever decreasing secant values as the applied moment increases and the joint moves from elastic to elasto-plastic to plastic behaviour.

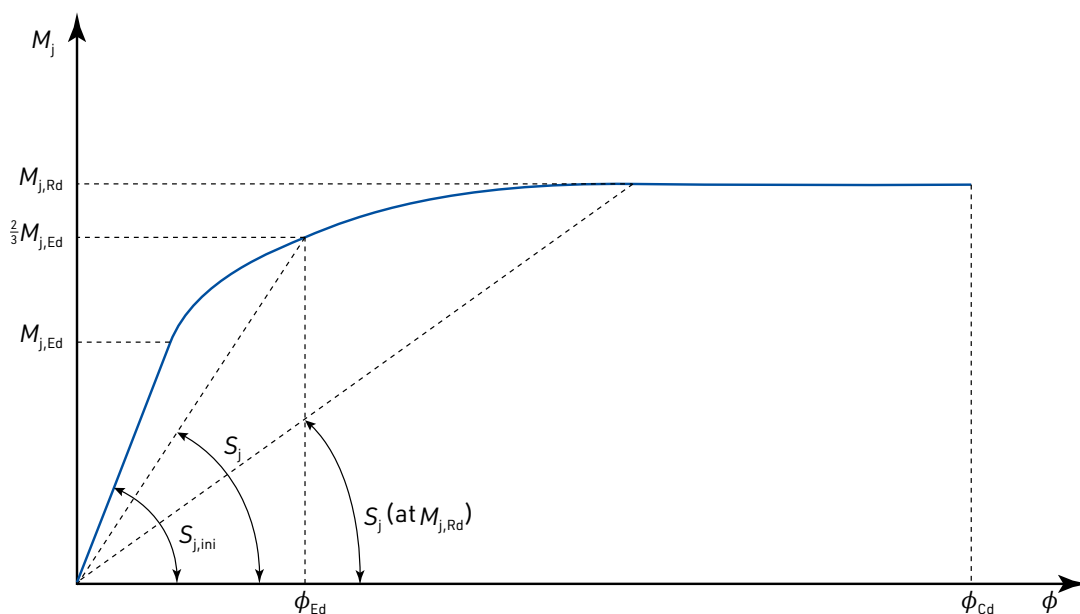


Figure 1.
Moment rotation behaviour
of a semi-continuous joint

P183 describes how to apply semi-continuous design to braced frames. The key steps are summarised below. It is based on the premise that standard joints will be used, but 25 years after it was published we should be able to offer greater design/detailing freedom.

SCI has previously published guidance on semi-continuous unbraced frames⁴, based on the so-called wind-moment method. Some readers may know that this is a very opaque way of designing frames, where assumptions are made that implicitly ‘give the right answer’, but clearly don’t represent what happens in reality. The wind-moment method was widely used in the days of ‘hand calculations’, and may still have a place as even today explicit design of semi-continuous unbraced frames is extremely complex (because of the load-unload behaviour of the joints under load reversal, which results in varying stiffnesses and ‘shakedown’). Care should be taken to ensure that a given design falls within the scope of application of the wind-moment method, with particular attention drawn to the need for fixed base columns.

How to do it?

P183 was written at a time when it was envisaged hand analysis and design would be adopted. Although today one would imagine that software might well be used for semi-continuous frame design, P183 (and the notes below) contains much useful information and guidance. Its Section 6 describes the design procedure in only two pages, with the stages summarised below. These are presented as a ‘teaser’ to show designers the ease and potential benefits, and encourage them to study the complete guidance.

Scheme design

Select column sizes to resist axial load alone in an overall buckling check. The utilisation of perimeter columns should be limited to 0.8, to allow some

reserve for applied moment. The utilisation of internal columns may approach 1.0 (a lower value should be used for columns in an unbalanced situation, for which guidance is also given in P183).

Select Class 1 or 2 beam sections, based on the following criteria and the assumption that the beams will be restrained against failure in lateral torsional buckling:

- Internal span $M_{pl,Rd} \approx 0.70 M_{Ed}$
- External span $M_{pl,Rd} \approx 0.80 M_{Ed}$

Where:

$M_{pl,Rd}$ = moment resistance of the beam

M_{Ed} = free bending moment at ULS

Final design

Joints

Select standard joints from the design tables provided in P183. Note that these are the same as the so-called wind-moment joints specified in the ‘Green Book’⁵, and, although this is a BS 5950 orientated guide, the resistances are based on the EN 1993-1-8 component method and are ‘Eurocode compatible’. The minimum joint moment resistance must satisfy the shortfall between the maximum applied moment and the moment resistance of the beam. Doing so means that no further check of the beams is required for the ULS. This highlights the interdependence of joint and member design, both of which should be carried out by the same ‘person’ for a semi-continuous frame.

The joint moment resistance should not exceed 50% (i.e. 50% partial strength) of the beam resistance for a joint to an internal column. The moment resistance of a joint to an external column should be approximately 20% of the beam resistance.

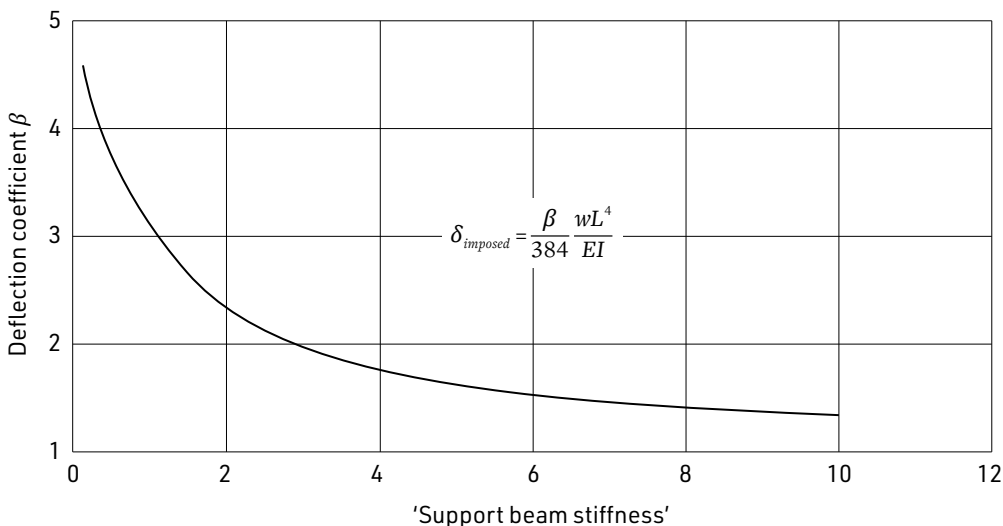


Figure 2: Beam deflection as a function of support rotational stiffness (internal span subject to UDL)

Check the joint shear resistance (using the design tables for standard joints), and add 'shear bolts' if necessary.

Beams

Calculate beam deflections under imposed (SLS) loading, using appropriate formulae and deflection coefficients as given below. Beams should be thought of as being rotationally restrained at the supports by springs. The spring stiffness represents the stiffness of the joint itself, plus that of the adjoining structure. Because of this stiffness, beam behaviour lies between 'built-in' and 'simply supported'.

The graph shown in Figure 2 is taken from P183 and shows that not a lot of support rotational stiffness is needed to significantly reduce the beam deflections.

For the analysis of orthodox frames (as defined in P183), there is no need for the designer to determine an effective support stiffness provided the standard joints are used. The formulae below may be used for beams subject to uniformly distributed loading (other load types are considered in P183). The coefficients used in these equations are a conservative approximation, but P183 includes rules that allow stiffness of a particular situation to be taken into account explicitly for more accurate results (P183 Appendix B.2 states how to combine joint, column and beam stiffnesses to determine the equivalent stiffness of the rotational support).

$$\delta_{imposed} = \frac{\beta}{384} \frac{wL^4}{EI}$$

For an internal span:

with joints having a partial strength in excess of 45%, $\beta = 3.0$

with joints having a partial strength less than 45%, $\beta = 3.5$

For an external span:

with joints having a partial strength in excess of 45%, $\beta = 3.5$

with joints having a partial strength less than 45%, $\beta = 4.0$

The relevance of using standard joints to justify the use of the relationships above is that they have a known (from testing) relationship between stiffness and strength, so can be chosen based solely on the latter. Guidance is given in P183 that would enable other joints to be used with stiffness and strength considered explicitly. The third key characteristic of a joint should not be forgotten even though rarely quantified – namely rotation capacity. Because the joints are partial strength they will normally be the location of the first plastic hinges, and so must be able to rotate sufficiently. The detailing of the standard joints means they are known, through testing, to be able to do this.

It is worth commenting on the UK National Annex to EN 1993-1-8⁶, and what this means for the use of non-standard joints. Whilst explicitly allowing the use of semi-continuous design, it states that 'until experience is gained with the numerical method of calculating rotational stiffness given in BS EN 1993-1-8, semi-continuous elastic design should only be used ... where supported by test evidence ... (or experience)'. Although this was published some time ago, it is doubtful that much experience has been gained. But of course the procedure described in P183, and here, does not use elastic design for ULS – the joints are behaving plastically well before collapse so their initial stiffness is irrelevant. Some inaccuracy when calculating deflections is part of structural engineering, not just limited to semi-continuous design. Indeed assuming joints are pinned, i.e. have zero stiffness, will always lead to an overestimation of deflections, with considerably greater inaccuracy than one would expect with semi-continuous joints designed using EN 1993-1-8!

Once deflections are determined, they are checked against limits as usual. Dynamic checks should also be performed (noting that whether joints are pinned, semi-continuous or continuous they behave as if they were rigid when subject to dynamic loading).

Columns

Check internal columns for overall buckling under the applied axial load in combination with any moment about the major axis resulting from unequal joint strengths, and any unbalanced minor axis moments. Minor axis moments should be calculated and distributed as in simple design (there are no standard details for minor-axis semi-continuous joints, though they might be envisaged), assuming eccentric beam reactions. The internal columns should also be checked for local capacity, considering axial loads and moments under pattern loading. Simplified procedures are given in P183.

Check perimeter columns for the applied axial load in combination with any major or minor axis moments. Both overall buckling and local capacity checks are required.

Check that the column sizes identified in the final design are compatible with the joint details, preferably without the need for column stiffening.

Details

Design the column bases, column splices, and the frame bracing systems as in 'simple construction'. The detailing of bases and splices, which may be pinned or chosen to provide moment continuity, must be properly reflected in the frame analysis and design assumptions. Care should be taken if semi-continuous joints are used as part of the bracing system, because the behaviour of the joints may be adversely affected by either the presence of additional axial or shear loads in the beams, or detailing to accommodate the bracing members.

What now and next?

A simple illustration of the benefits to be had from using semi-continuous design, rather than simple design, can be gained by considering the reduction in beam size that could be possible due to the use of the β values given above for deflections (as β goes down, the second moment of area can follow it and still result in the same deflection). A 457×152×82 UB has a second moment of area of 36,588 cm⁴. If used in a frame with semi-continuity such that the β value dropped from five to three, this beam could be replaced by a 406×178×60 UB (second moment of area 21,596 cm⁴). So a saving of 22 kg/m (almost 27%) for this simple comparison assuming deflection was critical. If we use beam weight as a simple representation of embodied carbon for the two cases, we can see how semi-continuous design could be highly relevant for future designs.

In 2022 SCI and BCSA will be working together to produce guidance on 'design for sustainable steel construction'. This will include a more detailed review of the semi-continuous frame design principles and rules. Unlike 25 years ago, designers today can use tools to determine joint strength, stiffness and rotation capacity, although levels of accuracy (of the last two) may be open to some question. SCI will be developing an easy-to-use tool, using Trimble's TEDDs platform, in 2022. This will allow key characteristics of 'typical' types of joint to be determined using the well-accepted EN 1993-1-8 component method. As TEDDs is part of the Trimble family this may also allow integration with their frame analysis and design software, which will really make semi-continuous design simple. ■

1. *UK structural steelwork: 2050 decarbonisation roadmap*, BCSA, 2021
2. *Design of semi-continuous braced frames*, P183, SCI, 1997
3. Dunant, C., Drewniok, M., Eleftheriadis, S., Cullen, J., & Allwood, J. M. (2018). Regularity and optimisation practice in steel structural frames in real design cases. *Resources, Conservation and Recycling*, 134 294-302
4. *Wind-moment design of low-rise frames*, P263, SCI, 1999
5. *Joints in steel construction – moment connections*, P207, SCI, 1995
6. *UK National Annex to Eurocode 3: Design of steel structures. Design of joints*, BSI, 2013



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Proposed changes to assessment of frame stability

In this second article looking at the forthcoming revisions to EN 1993-1-1, David Brown of the SCI considers the proposed guidance on frame stability with some disappointment that the requirements can be easily misunderstood. Experienced designers will no doubt use the correct approach, but those looking at the standard for the first time could be misled.

Once, there was BS 449

Designers of a certain age might comment that the first time that assessment of frame stability was highlighted occurred with the introduction of BS 5950. That is not true – BS 449 required sway stability to be considered, but gave no advice on how this was to be carried out. Clause 10c of BS 449 required that all structures “shall be adequately strong and stiff to resist sway”. There was no advice on what was considered to be adequately stiff.

In the same clause, BS 449 introduced notional horizontal forces to allow for “practical imperfections such as a lack of verticality”, which were only applied in combination with vertical loads, and not when wind or other horizontal loads were applied.

Frame stability in BS 5950

With the introduction of BS 5950, the need for structures to have adequate stiffness against sway was highlighted, but in the early versions there was still little guidance on what assessment should be completed. In the 1990 version, clause 2.4.2.3 required that to ensure adequate stiffness against sway “a separate check should be carried out for notional horizontal loads”. In the author’s opinion, confusion was introduced by this rather loose requirement in the standard, which has remained ever since.

In BS 5950, the notional horizontal forces (NHF) are used to allow for imperfections such as a lack of verticality. They appear in the load combinations with only vertical loads but, according to BS 5950, not when real lateral loads are applied. The confusion arises because in later versions of the standard, the very same notional horizontal forces are used entirely independently as part of the assessment of frame stability. With the benefit of hindsight, perhaps having some entirely different forces used in the assessment of frame stability – also with a different name – might have demonstrated the difference.

With the issue of the 2000 version of BS 5950 further clarity was added. In clause 2.4.2.4 the NHF are described as allowing for imperfections such as a lack of verticality (one wonders why “out of plumb” was not used). Separately, in clause 2.4.2.5, the need for sway stiffness is described and that secondary forces and moments must be allowed for in design if they are significant. These secondary forces and moments are described as “second order” and “PA”. This emphasis on “significance” is entirely correct – second order effects are always present, but may not be significant.

BS 5950 also included a measure of significance, λ_{cr} , which will be familiar to pre-Eurocode designers. The value of λ_{cr} is to be calculated for each storey in a structure, and is given by:

$$\lambda_{cr} = \left(\frac{h}{200\delta} \right)$$

where h is the storey height and δ is the horizontal displacement over the storey, due to the NHF (only) applied to the frame. BS 5950 clarifies that the deflection δ is due to “horizontal forces equal to 0.5% of the factored vertical dead, imposed and crane loads applied to the frame at each storey level”. Thus differently loaded storeys would have different horizontal loads, in proportion to the factored vertical load on the storey.

The value of 200 appearing in the denominator is inextricably linked to the definition of the NHF being 0.5% (or 1/200) of the factored vertical loads applied at that level. As will be seen later, the measure of frame stability (or, the significance of second order effects) can be determined using any

consistent set of forces and complementary expression for frame stability. Using a set of forces distinctly different from the NHF would perhaps have reduced the confusion referred to earlier.

One point to note is that the BS 5950 approach demanded a separate loadcase to be analysed, with only the NHF applied. This loadcase was only used to determine λ_{cr} .

2005 and BS EN 1993-1-1 arrives

The Eurocodes were available for use from 2005 – it is not surprising that revisions are being developed over 15 years later. Within the Eurocode, “PA” effects become the “Effects of deformed geometry of the structure” in clause 5.2.1 and the measure of frame stability becomes α_{cr} .

The value of α_{cr} is again calculated for each storey and is given by:

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

The value of H_{Ed} is the horizontal shear at the base of the storey and is equal to the summation of the horizontal loads applied to the structure above that level. In general, the horizontal loads are typically wind loads, plus the equivalent horizontal forces (EHF). Unlike BS 5950, the Eurocode requires that the EHF are always applied (unless the externally applied loads are very large).

The Eurocode defines $\delta_{H,Ed}$ as the relative displacement when the frame is “loaded with horizontal loads (e.g. wind) and fictitious horizontal loads which are applied at each level”. Practice has assumed that these fictitious horizontal loads are the EHF elsewhere described in the standard. Use of the EHF is ideal, since they are based on a proportion of the factored vertical load applied at that level.

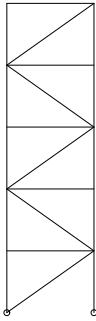
The EHF are based on the sway imperfection given in 5.3.2, which is 1/200 of the factored vertical loads, with optional fudge factors. Thus the value of H_{Ed} can be considered to be $\left(\frac{V_{Ed}}{200} + \text{wind} \right)$, which is quite the same as BS 5950 if one chooses to ignore the wind. Towards the base of an orthodox multi-storey building, the value of $\frac{V_{Ed}}{200}$ is large, and the contribution of the wind loads comparatively small.

Neglecting the relatively small influence of the wind, and neglecting the optional fudge factors in the determination of the EHF, the expression for α_{cr} becomes:

$$\alpha_{cr} = \left(\frac{V_{Ed}/200}{V_{Ed}} \right) \left(\frac{h}{\delta_{H,Ed}} \right) = \frac{h}{200\delta}, \text{ or the same as BS 5950.}$$

The advantage of completing the analysis of the frame using the “actual” loads of H_{Ed} and V_{Ed} was said to be that designers would have that load combination modelled as a matter of course, and that an additional, separate loadcase with just the EHF was inconvenient. Of course, it does not matter what forces are being used in the current Eurocode (and hence the use of the word fictional) since under an elastic analysis, the lateral deflection is proportional to the applied lateral loads – the ratio $\frac{H_{Ed}}{\delta_{H,Ed}}$ is a constant.

An analysis of a simple bracing system demonstrating the equivalence of BS 5950 and EN 1993-1-1 is shown in Table 1 (over). In the example the braced bay has been extracted from the structure for simplicity (the rest of the structure is of “simple construction” and does not contribute to the lateral stability).



	Storey	Storey height (m)	Factored load on level (kN)	BS 5950 NHF (kN)	EN 1993 ΣH_{Ed} (kN)	EN 1993 ΣV_{Ed} (kN)	Lateral movement (mm)	δ (mm)	BS 5950 $\lambda_{cr} \frac{h}{200\delta}$	EN 1993 $\alpha_{cr} \frac{H_{Ed} h}{V_{Ed} \delta}$
Roof			3200	16			13.72			
	5	3.5						1.48	11.9	11.9
Floor			10200	51	16	3200	12.24			
	4	3.5						2.02	8.7	8.7
Floor			10200	51	67	13400	10.22			
	3	3.5						2.81	6.2	6.2
Floor			10200	51	118	23600	7.41			
	2	3.5						3.17	5.5	5.5
Floor			10200	51	169	33800	4.25			
	1	4						4.25	4.7	4.7
Ground					220	44000	0.00			

Table 1: Frame stability according to BS 5950 and BS EN 1993-1-1

Looking to the future

As has been noted in earlier articles, the draft of EN 1993-1-1 is mature – significant changes are not anticipated.

The proposed measure of frame stability is called $\alpha_{cr,sw}$ (the subscript “sw” indicates “sway”) and is given by:

$$\alpha_{cr,sw} = \frac{K_{st} H_{st}}{\sum N_{Ed,i}}$$

(the subscripts “st” indicate “storey”)

K_{st} is the lateral rigidity of the storey and is given by:

$$K_{st} = \frac{H_f}{\Delta_f}$$

where

H_f is a fictitious horizontal force applied at the top of the columns of the storey;

Δ_f is the horizontal displacement at the top of the storey due to H_f relative to the bottom of the storey;

$\sum N_{Ed,i}$ is the sum of axial forces within the columns under consideration;

H_{st} is the height of the storey.

For Eurocode designers, the formula for $\alpha_{cr,sw}$ looks like a simple rearrangement of the current expression for α_{cr} , but care must be taken when evaluating K_{st} . The draft seems to imply that the lateral rigidity of a multi-storey frame can be assessed by considering each storey in isolation. Designers may be tempted to do just that and model single storeys, in isolation, with some arbitrary load applied at the top of the storey.

The correct deflections over a storey can only be determined from including the full height of the building in one analysis and calculating the differential deflections over the storey under consideration. Figure 5.1 of the current standard helpfully shows the assessment of one storey as part of a larger model – indicating that the entire stability system should be modelled, not storeys in isolation. In addition, the draft does not clarify that the fictitious load applied at the top of the columns at each storey must be consistent throughout the model. The fictitious load must be some fixed

proportion of the factored vertical load applied at that level.

Although one might feel disappointment with the proposed rules, they are not significantly different to the current standard – which similarly does not clarify the need for the lateral loads to be a fixed proportion of the vertical loads. Previous practice with BS 5950 (and the general use of the EHF to determine frame stability) has probably meant designers “knew” what was required, rather than being left uncertain by the standard.

Recommended approach

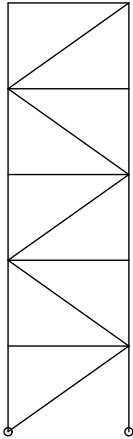
Every storey should be modelled in one analysis, exactly as was done in BS 5950 and the current version of the Eurocode. The fictional lateral loads applied at each level should be the same proportion of the factored vertical load at that level. To avoid confusion with the NHF of BS 5950 and the EHF of the Eurocode, perhaps these fictitious loads, used only to establish the measure of frame stability, could be 2% of the factored vertical load at that level. The final calculation becomes a convenient expression:

$$\alpha_{cr,sw} = \frac{H_{st}}{50\delta}$$

This equation may look different to the Eurocode, but yields the same result, as demonstrated in Table 2. The deflections are of course larger, but the end result is the same – allowing for some loss of precision.

Conclusions

The proposed clause will hopefully go a long way to clarify the confusion between the use of notional/equivalent/fictitious horizontal forces to assess frame stability, and separately the use of notional/equivalent horizontal forces to allow for a lack of verticality. The wording of the clause could be improved, in particular to advise that the fictitious forces should be a fixed proportion of the loads applied at each level. Using a fixed absolute force at each level will yield the wrong result (unless the vertical loads at each level are identical), as will attempting to model single storeys in isolation. ■



	Storey	Storey height (m)	Factored load on level (kN)	Fictitious force (2%)	Lateral movement (mm)	δ (mm)	$\alpha_{cr,sw} \frac{H_{st}}{50\delta}$
Roof			3200	64	54.56		
	5	3.5				5.81	12.0
Floor			10200	204	48.74		
	4	3.5				8.02	8.7
Floor			10200	204	40.72		
	3	3.5				11.17	6.3
Floor			10200	204	29.55		
	2	3.5				12.61	5.6
Floor			10200	204	16.94		
	1	4				16.94	4.7
Ground					0.00		

Table 2: 2% fictitious lateral loads, and resulting values of $\alpha_{cr,sw}$

Proposed Annex B

The proposed amendments to EN 1993-1-1 include Annex B which deals with semi-compact sections (the contents of the current Annex B have been promoted to the main part of the code). Richard Henderson of the SCI compares the proposals with the treatment of semi-compact sections in BS 5950.

1 Introduction

In BS 5950-1:2000, section 3.5 Classification of cross-sections includes a subsection: 3.5.6 Effective plastic modulus, dealing with semi-compact sections. This subsection allows semi-compact sections to be designed using either the elastic modulus or the effective plastic modulus (denoted Z and S_{eff} respectively in BS 5950). This approach is limited to bisymmetrical sections, circular tubes and I and H sections with unequal flanges subject to bending in the plane of the web. The value of S_{eff} depends on the limiting values of the web slenderness given in the relevant table. These values adjust the class boundaries according to ratios of axial stress and resistance in the webs of the cross section. The magnitude of S_{eff} falls between the elastic and plastic moduli and credits the cross-section with increased resistance.

Despite the presence of the established approach in BS 5950, BS EN 1993-1-1:2005 did not include a basis for using an increased modulus for the design of semi-compact sections but this omission is now to be remedied. The proposed Annex B in prEN 1993-1-1 now includes the determination of $W_{ep,y}$ the elasto-plastic section modulus for semi-compact doubly symmetric cross-sections and circular or elliptical hollow sections. This is illustrated in Figure 1.

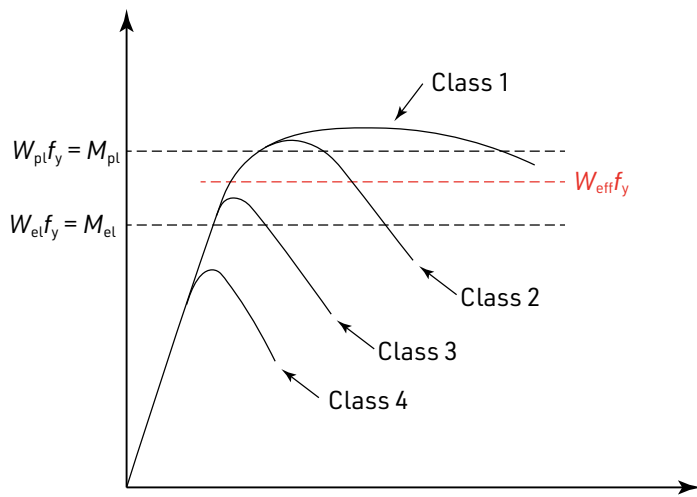


Figure 1: Illustration of effective modulus

The Annex also takes the effect of axial force into account but in this case an adjusted bending resistance is determined by applying a factor to the resistance in the absence of axial force, calculated using the elasto-plastic modulus. The factor depends on the ratio of the design axial force to the plastic axial resistance.

2 Detailed Proposals in Annex B

The proposals have been examined in detail for the case of symmetrical welded I sections. The formulae for the revised proposals are as indicated below.

$$W_{ep,y} = W_{pl,y} - (W_{pl,y} - W_{el,y}) \beta_{ep,y}$$

$$W_{ep,z} = W_{pl,z} - (W_{pl,z} - W_{el,z}) \beta_{ep,z}$$

where $W_{ep,y}$ and $W_{ep,z}$ denote the elasto-plastic moduli. The plastic and elastic moduli adopt their usual notation.

The β_{ep} values depend on the parameter ϵ which varies with the yield strength and the slenderness ratios of the elements of the cross section defined in the classification table.

The class boundaries for certain sections are to be modified. The relevant ones to this article are those relating to webs in compression and combined bending and compression, and the boundary between class 3 and class 4. In general, the slenderness ratios at the boundaries have reduced: for example, for internal compression parts, the slenderness at the class 1/class 2 boundary has reduced from $c/t \leq 33\epsilon$ to $c/t \leq 28\epsilon$.

The expressions for rolled or welded I or H sections are as follows:

$$\beta_{ep,y} = \text{Max} \left\{ \frac{\frac{c}{t_f} - 10\epsilon}{4\epsilon}; \frac{\frac{c}{t_w} - 83\epsilon}{38\epsilon}; 0 \right\} \text{ but } \beta_{ep,y} \leq 1.0$$

$$\beta_{ep,z} = \text{Max} \left\{ \frac{\frac{c}{t_f} - 10\epsilon}{6\epsilon}; 0 \right\} \text{ but } \beta_{ep,z} \leq 1.0$$

In the first expression, 10ϵ is the limiting slenderness for class 2 for outstand flanges subject to bending; 83ϵ is the limiting slenderness for class 2 for internal compression parts subject to bending and 38ϵ is the limiting slenderness for class 3 for internal compression parts subject to compression.

2.1 Resistance of cross-sections

The resistance of cross-sections that are required to sustain combined bending and axial compression is established by determining the design elasto-plastic bending resistance reduced in the presence of axial force. The section classification is established first, followed by the elasto-plastic modulus. The bending resistance is then found and reduced according to the ratio of the design compression force to the plastic compression resistance of the section.

The process is illustrated below in an example and the results compared with the equivalent outcome determined using the BS 5950:2000 procedure.

3 Example

3.1 prEN 1993-1-1 procedure

Design a fully restrained plate girder subject to a bending moment $M_{y,Ed} = 615 \text{ kNm}$ and an axial compression $N_{Ed} = 100 \text{ kN}$. Material grade: S355.

Using the Blue Book resistances for an equivalent rolled section as a starting point, try a beam 530 deep by 230 wide with 12.5 mm thick flanges and a 6 mm thick web. Assume a 6 mm leg web to flange weld.

Section properties are presented in Table 3.1.

Table 3.1 Section Properties

Property	Value	Units
Area	8780	cm ²
Plastic modulus $W_{pl,y}$	1870	cm ³
Plastic modulus $W_{pl,z}$	335	cm ³
Elastic modulus $W_{el,y}$	1696	cm ³
Elastic modulus $W_{el,z}$	221	cm ³

Classify the section

Selected class limits are presented in Table 3.2. In the table, c is the flange outstand beyond the weld toe, d_w is the web depth between weld toes.

Table 3.2 Classification

Flange	Web	Flange	Web
Slenderness	$c/t_f = 8.48$	Slenderness	$d_w/t_w = 82.2$
Class 1 limit, bending	$9\varepsilon = 7.29$	Class 1 limit, bending and axial	$d_w/t_w = 50.7$
		Class 2 limit, bending	$d_w/t_w = 67.2$
Class 2 limit, bending	$10\varepsilon = 8.1$	Class 2 limit, bending and axial	$d_w/t_w = 59.1$
		Class 3 limit, compression	$d_w/t_w = 30.8$
Class 3 limit, bending	$14\varepsilon = 11.3$	Class 3 limit, bending and axial	$d_w/t_w = 93.3$

The section therefore has class 3 flanges and a class 3 web under this loading. The semi-compact section classification means that the proposed Annex B may be used to determine an elasto-plastic section modulus W_{ep} . Substituting in the equation for $\beta_{ep,y}$,

$$\beta_{ep,y} = \text{Max} \left[\frac{8.48 - 8.1}{3.24}; \frac{82.2 - 67.2}{30.8}; 0 \right] \text{ but } \beta_{ep,y} \leq 1.0$$

Precise calculation gives a value of $\beta_{ep,y} = 0.485$ which differs slightly from the results of the above expression due to rounding. The reduction relative to the plastic modulus is therefore just under half the difference between the plastic and elastic moduli, giving a value for the elasto-plastic modulus of 1785 cm³.

The reduction factor for axial load in the major axis bending resistance for sections of class less slender than 4 is:

$$1 - n = 1 - \frac{N_{Ed}}{N_{pl,Rd}}; N_{pl,Rd} = \frac{A_f f_y}{\gamma_{M0}}$$

For the section being considered, $N_{pl,Rd} = 3117$ kN so $n = 0.0321$ and $(1 - n) = 0.968$. The major axis bending resistance in the presence of axial load is given by:

$$M_{N,ep,y,Rd} = M_{ep,y,Rd} (1 - n) = \frac{W_{ep,y} f_y}{\gamma_{M0}} (1 - n)$$

Substituting values gives a bending resistance of 614 kNm and a utilization of 1.00 to two significant figures.

3.2 BS 5950 Procedure

The corresponding procedure in BS 5950 involves determining the stress ratios r_1 and r_2 in the section. These are respectively the ratio of the applied axial compression to the web compression resistance, and the ratio of the applied axial compression to the compression resistance of the gross section, both based on the design strength of the web. In this case the web depth d is taken as the depth between flanges. The flange and web thicknesses are denoted T and t respectively. The stress ratios are:

$$r_1 = \frac{100000}{505 \times 6 \times 355} = 0.0930$$

Similarly, the value of r_2 is 0.0321. The value of ε in BS 5950 is 0.88 as it is based on $p_y = 275$ MPa. The class limits are given in Table 3.3. Some entries in the table are given only for comparison with the prEN values. The flange outstand b is taken from the face of the web.

The section is therefore class 3.

The effective section modulus is given in BS 5950 cl. 3.5.6.2 for semi-compact sections as:

$$S_{x,eff} = Z_x + (S_x - Z_x) \left[\frac{\left(\frac{\beta_{3w}}{d/t} \right)^2 - 1}{\left(\frac{\beta_{2w}}{d/t} \right)^2 - 1} \right] \text{ but } S_{x,eff} \leq Z_x + (S_x - Z_x) \left[\frac{\beta_{3f} - 1}{\beta_{2f} - 1} \right]$$

The suffices of the β values refer to the limiting values of the

Table 3.3 Classification

Flange	Web	Flange	Web
Slenderness	$b/T = 8.96$	Slenderness	$d/t = 84.2$
Class 1 limit, bending	$8\varepsilon = 7.04$	Class 1 limit, bending	$d/t = 70.4$
		Class 1 limit, bending and axial	$d/t = 64.4$
Class 2 limit, bending	$9\varepsilon = 7.92$	Class 2 limit, bending	$d/t = 88.0$
		Class 2 limit, bending and axial	$d/t = 77.2$
Class 3 limit, bending	$13\varepsilon = 11.4$	Class 3 limit, bending	$d/t = 105.6$
		Class 3 limit, bending and axial	$d/t = 99.2$

classifications of the web and flange. For example, β_{3w} refers to the limiting value of d/t for a class three web. The elastic and plastic moduli are denoted Z and S and the suffix x refers to the major axis in BS 5950 notation.

Substituting values:

$$S_{x,eff} = Z_x + (S_x - Z_x) \left[\frac{0.388}{0.651} \right] \text{ but } S_{x,eff} \leq Z_x + (S_x - Z_x) \left[\frac{0.277}{0.439} \right]$$

The quotients in square brackets are 0.599 and 0.622 respectively so the effective modulus depends on the web classification. In this case, the effective modulus takes a value about 60% of the way between the elastic and plastic moduli - i.e. 1801 cm³. This is about 1% greater than the elasto-plastic modulus calculated using the prEN. The bending resistance of the section is 639 kNm.

Using cl. 4.8.3.2 to carry out a cross-section check, the utilization is given by:

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

In the example, M_y (the minor axis moment) is zero. The utilization is therefore:

$$\frac{100}{3117} + \frac{615}{639} = 0.994$$

4 Conclusion

The procedure set out in Annex B of prEN 1993-1-1:2020 for determining an elasto-plastic modulus for class 3 sections has a similar result to that adopted in BS 5950. The approaches are different in that the approach to classification in EC3 uses factors that are related to the effect of axial force on the position of the plastic or elastic neutral axis, whereas the BS 5950 approach uses the ratios of applied axial load to the resistance of the web and gross section.

In the prEN, a reduced value of bending resistance in the presence of axial load is calculated, whereas in BS 5950 a cross-section check involves the determination of a utilization factor.

It is interesting that the BS 5950 approach uses the elastic modulus of a class 3 section as a starting point and determines what proportion of the difference between plastic and elastic moduli can be added to suit the classification. The prEN approach uses the plastic modulus as a starting point and determines what proportion of the difference must be deducted.

For the example given, the outcomes are very similar: the prEN approach indicates the section is 0.4% over-utilised, whereas the BS 5950 approach shows the section is 0.6% under-utilized.

The EU funded an RFCS project over 36 months from July 2004: *Plastic member capacity of semi-compact steel sections - a more economic design* to develop and justify the proposed procedure in Annex B. The work involved 60 physical experiments and more than 2600 FE simulations. The total budget was €674k, with an EU contribution of €404k. We can be reassured that structures designed using the procedure in BS 5950 are sound. ■

Design of Angles

David Brown of the SCI offers advice on the use of angles in bending, in response to questions received by the advisory service. Angles subject to bending are often found carrying loads over openings, but also may be found as continuous chords in trusses. Eurocode guidance follows that presented in the previous standard, BS 5950.

Using an angle in bending? Select another profile!

Questions relating to angle sections are surprisingly common at the SCI. Usually, they are not related to the use of angles in compression (typically in a truss) where the design guidance is clear, but rather concern the bending resistance of angles. Often, it becomes clear that the angle is unrestrained, so the real issue is the buckling resistance of the member. In those situations, the SCI's advice is to select a different profile. Angles in bending are often used to support brickwork over openings. Although this is a common detail in domestic applications, the member selection is fraught with potential risk. As illustrated in Figure 1, the compressed leg will wish to move out of plane, which combined with the usual eccentric application of load will lead to a twist of the member – and a dissatisfied client if the supported façade cracks.

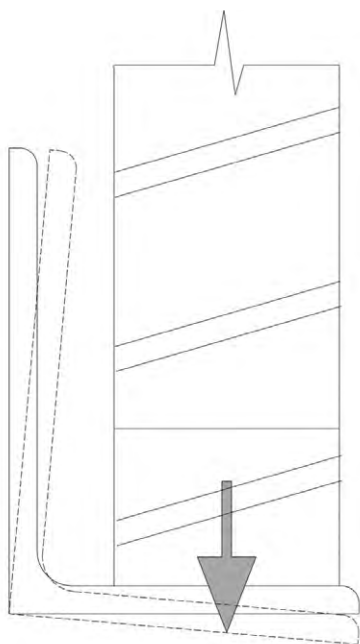


Figure 1: Behaviour of a single angle under load

As an aside to the main theme of this article, any member supporting an eccentric load will twist. When carrying eccentric loads commonly found in domestic construction (but equally applicable in all situations) thought should be given to using a hollow section which is torsionally very stiff. Although the hollow section member itself may be more expensive and the connections more involved, the risk of twist has been minimised. Figure 2 shows a member to be used in a house extension – a selection commended by the author.

Angles in compression

Here, the design guidance is straightforward. Angles do not buckle about their rectangular axes which are aligned with the angle legs, but buckle about their principal axes, u-u or v-v, as shown in Figure 3.

Following the same principle as illustrated in Figure 1, each leg wishes to buckle in its own out-of-plane direction, which causes a twist. This torsional behaviour is allowed for in the calculation of the slenderness, which considers the different axes, adjusts the slenderness for the torsional behaviour and finally allows for the restraint (or otherwise) offered by the end connections. The determination of slenderness is covered in clause 4.7.10 in BS 5950.



Figure 2: Torsionally stiff member for eccentric loading condition

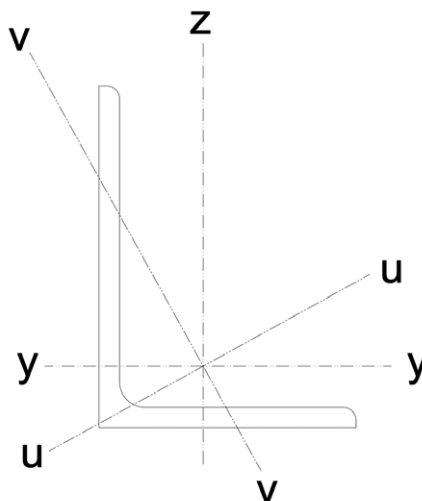


Figure 3: Axis identification for angles

Section BB.1.2 of BS EN 1993-1-1 provides equivalent (but not as comprehensive) guidance.

Angles in bending – lateral torsional buckling

BS 5950 looks hopeful, since clause 4.3.8 covers the buckling resistance moment for single angles and provides both a basic method and a simplified method. Optimism may be misguided, especially if unequal angles have been selected. As will be seen later, the designer must overcome a number of challenges.

Equal angle buckling resistance moment – BS 5950 simplified method

Assuming that the angle is being used to carry load across an opening, the heel of the angle is in tension. The simplified method of BS 5950 clause 4.3.8.3 gives the buckling resistance moment, M_b for members subject to bending about the x-x axis as:

$$M_b = p_y Z_x \left(\frac{1350\epsilon - L_E/r_x}{1625\epsilon} \right) \text{ but } M_b \leq 0.8 p_y Z_x$$

Where $\epsilon = (275/p_y)^{0.5}$

BS 5950 specifies the elastic modulus is to be used in the calculations.

Assuming the angle is $150 \times 150 \times 12$, 4 m long and S275, then $\epsilon = 1.0$ and (from section tables) $r_x = 29.5$ mm.

$$\text{Then } M_b = 275 \times 67.7 \times 10^3 \left(\frac{1350 \times 1 \times 4000 / 29.5}{1625 \times 1} \right) \times 10^{-6} = 13.9 \text{ kNm}$$

Equal angle buckling resistance moment – BS EN 1993-1-1

The Eurocode is less helpful, as no design advice is given. Designers are encouraged to consult The Institution of Structural Engineers’ “Grey Book”¹ which recommends that the applied moment be resolved about the u-u and v-v axes, and an interaction expression used to verify the member. The relative slenderness $\bar{\lambda}_{LT}$ is given by:

$$\bar{\lambda}_{LT} = 0.72 v_a \sqrt{\frac{f_y}{E}} \phi_a \lambda_v$$

This is a rearrangement of the expression given for λ_{LT} in B.2.9.2 of

BS 5950, determined by dividing the BS 5950 slenderness by $\lambda_i = \pi \sqrt{\frac{E}{f_y}}$.

ϕ_a is the equivalent slenderness coefficient and is given in the “Blue Book” as 3.77 for this particular angle.

The value of v_a is more complicated, and is given by:

$$v_a = \frac{1}{\sqrt{\left(\sqrt{1 + \left(\frac{4.5\psi_a}{\lambda_v} \right)^2} + \frac{4.5\psi_a}{\lambda_v} \right)}}$$

This is the same presentation as found in B.2.9.3 of BS 5950.

ψ_a is the monosymmetry index, found in the Blue Book for unequal angles.

For equal angles, $\psi_a = 1$.

For the selected angle, $\lambda_v = 4000/29.5 = 135.6$

$$v_a = \frac{1}{\sqrt{\left(\sqrt{1 + \left(\frac{4.5 \times 1}{135.6} \right)^2} + \frac{4.5 \times 1}{135.6} \right)}} = 0.984$$

Therefore:

$$\bar{\lambda}_{LT} = 0.72 \times 0.984 \times \sqrt{\frac{275}{21000}} \times 3.77 \times 135.6 = 0.580$$

From Table 6.4 of BS EN 1993-1-1, curve *d* must be selected, and $\alpha_{LT} = 0.76$ from Table 6.3.

According to expression (6.56), $\chi_{LT} = 0.724$

In the “Grey Book”, the applied moment is resolved into moments about the u-u and v-v axes and the buckling resistance moment calculated about the u-u axis. The moment resistance about the v-v axis is the resistance of the cross section. Using the elastic modulus about the u-u axis poses an immediate problem as this property is not given in published tables. The second moment of area about the u-u axis is given as 1170 cm⁴ and the dimension *c* to the centroid given as 41.2 mm. The distance to the extreme fibre at the angle toe is 106 mm. The minimum modulus, W_u is given by:

$$W_u = \frac{1170 \times 10^4}{106} = 110 \times 10^3 \text{ mm}^3$$

The modulus about the v-v axis can be calculated as

$$\frac{303 \times 10^4}{58.3} = 52 \times 10^3 \text{ mm}^3$$

The lateral torsional buckling resistance is then given by:

$$M_b = \frac{0.724 \times 275 \times 110 \times 10^3}{1.0} \times 10^{-6} = 21.9 \text{ kNm}$$

This is the LTB resistance about the u-u axis, so cannot be compared directly with the buckling moment calculated according to BS 5950.

If the applied moment was 14 kNm in the major axis, the resolved moments in the u-u and v-v axes are both 9.9 kNm

According to the “Grey Book”, the interaction to be satisfied is:

$$\frac{M_{v,Ed} \gamma_{M0}}{W_{vfy}} + \frac{M_{u,Ed} \gamma_{M1}}{\chi_{LT} W_{u, \min} f_y} \leq 1.0$$

Substituting:

$$\frac{9.9 \times 10^6 \times 1.0}{52 \times 10^3 \times 275} + \frac{9.9 \times 10^6 \times 1}{21.9 \times 10^6} \leq 1.14 \text{ which is unsatisfactory.}$$

The approach set out in the “Grey Book” is following the advice given in clause I.4.2 of BS 5950, described as the “Basic Method”, which requires the applied moments to be resolved about the principal axes u-u and v-v, and an interaction check for biaxial moments to be completed. BS 5950 refers the designer to clause 4.8.3.3.1 for the biaxial check, but using the moments and resistances about the principal axes in the same way as the “Grey Book”. The only notable difference is that the LTB resistance according to BS 5950 is 26 kNm, compared to the value of 21.9 kNm computed above. According to BS 5950, the interaction result is 1.07, lower than the Eurocode result, due solely to the increased LTB resistance.

The “Simplified method” of I.4.3 uses the “Simplified method” of clause 4.3.8.3 to calculate the buckling resistance moment, M_b , about the x-x axis. This value is then used in the interaction expression of I.4.3, but using the moments and resistances about the rectangular axes. With an applied moment of 14 kNm about the major axis only and a resistance $M_{bx} = M_b = 13.9$ kNm as calculated above, the interaction result is 1.01.

Buckling of unequal angles – more complexity

BS 5950 does not permit the “Simplified method” to be used for unequal angles – the “Basic method” of clause 4.3.8.2 must be used. This is going to be painful for designers, as an unequal angle is probably preferable – at least by intuition, to have the longer leg vertical if spanning over an opening. In Both BS 5950 and the “Grey Book”, applied moments are to be resolved into the u-u and v-v axes. The position of the centroid and the angle between the principal axes and the rectangular axis is given in the Blue Book, so this is not overly difficult. The Blue Book also gives the second moment of area about the u-u and v-v axes, so with some trigonometry, the distances to the extreme fibres and the modulus about each principal axis can be determined. The monosymmetry index is given, so the calculation of v_a given above can be completed.

The complexity is not over, especially if the angle is not at least Class 3 (semi-compact in BS 5950). According to Table 11 of BS 5950, the Class 3 limit for a single angle when the compression is due to bending is 15 ϵ , so a 150 × 10 leg in S275 would be satisfactory, but nothing more slender. According to BS EN 1993-1-1 and Table 5.2, the limiting value is 14 ϵ , but based on the dimension *c*. For a 150 × 10 leg, the dimension *c* is around 128 mm, so *c/t* = 12.8 and the limiting value is 12.9, meaning the same conclusion is reached.

If the angle is Class 4 (and many are), the Eurocode method of calculating effective properties, or the BS 5950 alternative of using a reduced design strength adds more complexity. If the member is used under combined bending and axial (for example, as a continuous chord in a truss), the design effort involved with a Class 4 angle is likely to be too much to be worthwhile.

Conclusions

This article has identified two messages. Firstly and most importantly, an angle may be cost-effective but is not suitable for carrying significant moments. Under bending, and under axial loads, there is torsional behaviour causing a significant twist, which may be very detrimental if an angle is supporting brittle materials. The second message is that the verification of an unequal angle in bending is complicated – more so if the member is Class 4 and even more so if the member is subject to combined axial load and bending. If faced with this design situation, an equal angle of at least Class 3 cross section is recommended. In general, the author’s advice remains that if faced with an angle subject to unrestrained bending, substituting an alternative profile is a much better solution. ■

1. *Manual for the design of steelwork building structures to Eurocode 3: October 2010 ISE, 2010*

Bolted connections to hollow sections and column webs

David Brown of the SCI reviews design models – including simple analysis, resistance formulae and FE-based software.

Connections to planar elements

Designers occasionally wish to develop a bolted connection to the wall of a hollow section, or to the web of a member – generally a column, as shown in Figure 1. Whilst fin plates, extended plates and plates across the toes of the supporting member can be used (Figure 2) particularly when the loading is shear only, situations do arise when end plate type connections are required. If the member is subject to an applied moment, or to an axial tension, the bolts are in tension and the planar element of the connected member (the hollow section wall or member web) is subject to tension applied by the bolts. The out of plane resistance of those planar elements is the subject of this technical article.

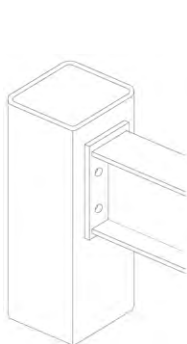


Figure 1: Bolted connection to hollow section

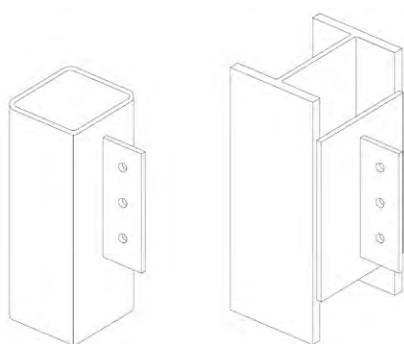


Figure 2: Alternative bolted connections

“Blind” connectors

For connections to hollow sections and other situations with one-sided access, various proprietary fixings are available¹. Among these fixings are bolts with a slotted sleeve which is designed to flare out and provide the anchor on the side with no access. The deformation of the sleeve may be by an internal mandrel driven through the bolt, or by a threaded cone which rides along the bolt shank as the bolt is tightened. Other fixings include bolts with a pivoted anchor initially lying within a slot in the bolt shank. After inserting and rotating the bolt the anchor pivots into position.

Some details adopt ordinary bolts, with an access hole in the side wall of the member, though this is not common in the author’s experience. Some designers propose forming threaded holes, by drilling and tapping the member. The UK has generally advised against drilling and tapping holes as the result is very different from the use of a high strength nut. Nuts have an ultimate strength of 800 N/mm² or 1000 N/mm² (Property Class 8.8 and 10.9 respectively) so the performance of a bolt threaded in material with an ultimate strength of perhaps 500 N/mm² will be rather different. In contrast to the UK view, the proposed revisions to EN 1993-1-8 include a table giving the minimum length of thread engagement in a threaded hole in S235, S355, S460 and stronger material. A note to the table allows the minimum thread engagement length to be set by the National Annex, so this will be the opportunity for the UK to prohibit this approach if required.

Critical design checks

In the typical details illustrated in Figure 1, the critical check is not the resistance of the fixing, which may be selected to accommodate the design forces. The critical check is very likely to be the resistance of the supporting member to the out-of-plane forces, particularly with the relatively thin walls of hollow sections and some webs, depending on the section. The ultimate resistance will obviously be important, but the deformation at working loads should also be considered as any rotation of the joint will contribute to the overall deformation of the

supported element – for example if a parapet handrail had a base connection of this form.

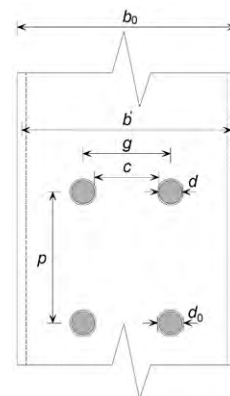
Design models

Many designers will know of the CIDECT Design Guides, covering all aspects of construction with hollow sections. Design Guide 9² provides an expression for the resistance of a hollow section face in equation 6.27 of the guide. The resistance expression covers a group of four bolts in tension, as would commonly be found around the tension flange of a beam. The resistance expression is reproduced below, but with the nomenclature changed to Eurocode terms:

$$F_{Rd} = f(n) \frac{f_y t^2}{(1-c/b')^2} \left[2 \frac{p-d}{b'} + 4(1-c/b')^{0.5} \right]$$

Where:

- p is the vertical pitch of the bolt group
- g is the horizontal gauge of the bolt group
- d is the bolt diameter
- $b' = b_0 - t$
- $c = g - d$
- t is the thickness of the hollow section wall
- $f(n) = 1 + n \leq 1.0$
- $n = \text{column stress} / \text{yield stress}$



Designers should note that in the CIDECT design guide, compression is negative. This is the reverse of the sign convention in BS EN 1993-1-8.

Gomes *et al*³ developed a formula for resistance which allowed for the relative width of the bolt group within the hollow sections wall (i.e. is the bolt group relatively narrow or wide with respect to the width of the wall?).

The formula developed by Gomes *et al* is:

$$F_{Rd} = k \frac{\pi f_y t^2}{1 - \frac{b'}{b_0}} \left[\left(1 - \frac{b'}{b_0} \right)^{0.5} + \frac{2c}{\pi b_0} \right]$$

Where:

If $\frac{b'+c'}{b_0} > 1$ then $k = 1.0$

Otherwise $k = 0.7 + \frac{0.6(b'+c')}{b_0}$

In the above expressions the quite different definitions of b' and c' should be noted:

$b' = g + 0.9d_{ba}$ and $c' = p + 0.9d_{ba}$ where d_{ba} is the effective diameter of the bolt clamping area (the average of the dimensions across the flats and across the points of the bolt head).

Finally, a formula is given in P358⁴ used for the tying resistance of a bolted connection to a hollow section wall. The expression is reproduced below but adopting the yield strength rather than the ultimate strength and using nomenclature previously defined.

$$F_{Rd} = \frac{8M_{pl}}{(1-\beta_1)} [\eta_1 + 1.5(1-\beta_1)^{0.5}(1-\gamma_1)^{0.5}]$$

Where:

$$M_{pl} = \frac{f_y t^2}{4} (n_1 - 1) p - \frac{n_1 d_0}{2}$$

$\eta_1 = \frac{(n_1 - 1) p - \frac{n_1 d_0}{2}}{b_0 - 3t}$, which is equivalent to $\frac{p-d_0}{(b_0-3t)}$ for a group of four bolts

$$\beta_1 = \frac{g}{(b_0 - 3t)}$$

$$\gamma_1 = \frac{d_0}{(b_0 - 3t)}$$

d_0 is the diameter of the hole

n_1 is the number of rows of bolts

The general similarities between the expressions can be seen. The multiplier of 1.5 in the P358 expression is included to allow for axial compression in the column, but there is no indication of the stress ratio assumed. It should be noted that if the multiplier were larger, the resistance increases, so the value should decrease with increasing compression.

Comparison of results

Results are presented in Table 1 for the three design approaches, for two arrangements in a S355 SHS. Wang *et al*⁵ undertook physical tests and completed the same calculations – their values are shown for comparison. Whilst generally good agreement is seen for the CIDECT and SCI calculated resistances, there is clearly a significant difference when calculating the resistance according to Gomes for the 90 mm gauge. The test resistance indicated as “yield” is based on a limiting deformation of the chord face, equal to 3% of the SHS face, or 4.5 mm for the 150 SHS tested. This limiting deformation is recommended by CIDECT² and reflected in the resistance formulae given in the design guides.

Resistance calculations	$b_0 = 150 \text{ mm}; t = 8 \text{ mm}; d = 16 \text{ mm}; g = 60 \text{ mm}; p = 100 \text{ mm}$			$b_0 = 150 \text{ mm}; t = 8 \text{ mm}; d = 16 \text{ mm}; g = 90 \text{ mm}; p = 100 \text{ mm}$		
	Resistance (kN) according to design model					
	Gomes	CIDECT	P358	Gomes	CIDECT	P358
SCI	189	148	144	291	187	222
Wang <i>et al</i>	201	149	139	444	189	215
Test result (“yield”)	174			242		

Table 1: Calculated resistances for bolted connections to SHS face

There are many uncertainties in Table 1. Only one test was completed for each arrangement, so no statistical analysis is possible. It is not clear if the presented results allow for the measured material properties. The results for CIDECT assume no compression in the column (which is unreasonable; introducing compression reduces the resistance considerably) whilst the P358 calculation has an allowance for some (unspecified) compression. The P358 expression is for tying resistance, when irreversible permanent deformation is anticipated.

The paper by Wang *et al* describes three modes of failure when testing the expanding anchor type fixing. In the first mode, which happened in every test when the SHS wall was 5 mm, the fixings deformed and pulled through the SHS. In mode 2, failure was by a combination of deformation of the SHS wall and tensile failure of the fixings. Mode 3 was characterised by failure of the fixings. Pull-through is a very variable mode of failure and should be avoided. Assessment and evaluation standards for blind fasteners, such as EAD 330001⁶ insist that the failure mode cannot be pull-through. Reference 1 reports this behaviour in wall thicknesses below 8 mm.

FE models

Modelling the connection in a widely-used FE-based software yielded a maximum tension of around 200 kN for the bolts at 60 mm gauge, applying the 3% deformation limit to the SHS. This contrasts with the CIDECT value of 148 kN and P358 value of 144 kN. A check of the second connection with bolts at 90 mm gauge could not be completed – the software reported that the bolts would clash with the internal radius of the SHS.

With an axial compression of 300 kN the maximum tension reduced slightly. With an axial compression of 600 kN, the maximum tension reduced to around 185 kN. At this level of compressive stress, the CIDECT resistance drops from 148 kN to 92 kN.

Including deformation in compression

When bolting to an SHS wall, or to a column web, whilst the tension zone deforms in one direction, the compression zone will deform in the opposite direction, contributing to the overall rotation of the joint. Reference 3 offers advice on the calculation of the resistance in this situation.

Simple alternatives

In real life, plenty of connections will have to be made where the bolts cannot be located symmetrically to the supporting member. In these cases, a much simpler model may be appropriate, analysing a “beam” spanning between “supports”, with point loads at the positions of the fixings. A traditional assumption is that the width of the “beam” is defined by considering a 45° spread back to the support (but not double counting with an adjacent “beam”). Some assumptions need to be made about fixity at the “supports”. The development of a simple design model is shown in Figure 3 (adapted from Figure 4.9 in reference 2).

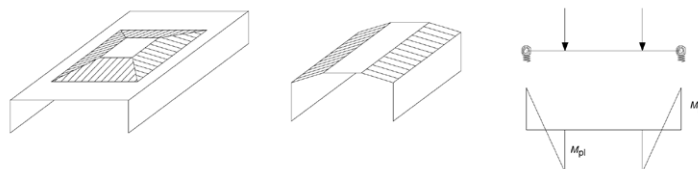
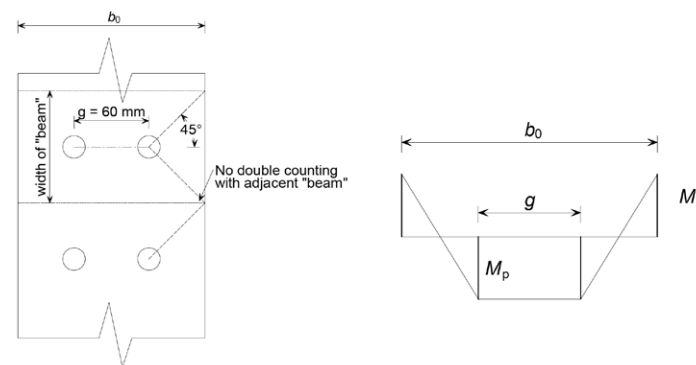


Figure 3: Development of design model

If the bolts are symmetrically placed at 60 mm gauge, as shown in Figure 4 the distance to the side walls is 45 mm. If dispersion in each direction is 45°, then the width of the “beam” is 90 mm (which does not double count the adjacent “beam”). With four plastic hinges, each 90 mm long, the resistance of the 8 mm wall is given by:

$$\frac{4 \left[\frac{355 \times 90 \times 8^2}{4} \right]}{45} \times 10^{-3} = 45 \text{ kN}$$

The resistance is significantly less than the CIDECT formula, and much lower than FE, but simple and conservative.



Determination of “beam” dimensions
Figure 4: Simple design model

Conclusions

For connections to thin elements such as hollow section walls or column webs, the fixing is generally not the critical component – it can be sized to suit. Several design models are available, with significant variations in the results. Simple models represent one end of the range, and FE the other. Whilst ultimate resistance is critical, designers should not forget the deformation of the components at serviceability loads which contribute to overall joint rotation. Finally, fixings should be specified to ensure that pull-through is not the failure mode, which may govern in thin material¹. ■

1. Tizani, W. Nethercot, D.A.; *The practice of blind bolting connections to structural hollow sections: A review*; Steel and composite structures, March 2001
2. Kurobane, Y; Packer, J.A; Wardenier, J; Yeomans, N.; *Design Guide for structural hollow section column connections*; CIDECT Design Guide 9, CIDECT, 2004
3. Gomes, F. C. T.; Jaspart, J. P; Maquoi, R.; *Moment capacity of beam-to-column minor axis joints*; Proceedings of IABSE International Colloquium on semi-rigid structural connections, Turkey 1996, IABSE, 1996
4. *Joints in steel construction: Simple joints to Eurocode 3 (P358)*; SCI & BCSA, 2014
5. Wang, Z-Y; Wang, Q-Y.; *Yield and ultimate strength determination of a blind bolted end plate connection to square hollow section column*; Elsevier, 2015
6. EAD 330001-00-0602; *Expanding structural bolting assemblies for blind fasteners*, EOTA 2017

More from the history of steel design code development

David Brown of the SCI looks at the work of Professor J. F. Baker, author of *The Steel Skeleton*, finding many issues of direct relevance to today's designers and originally presented in a highly engaging style.

Certain publications in the SCI library have an elevated status, indicated by the note “Important historically – keep for reference”. One of these special publications is *The Steel Skeleton* by Professor Baker, which comes in two volumes. Volume 1 (1954) covers elastic behaviour and design. Volume 2 (1956) is co-authored with Horne and Heyman (all from the University of Cambridge) and addresses plastic behaviour and design.

Of particular interest – at least to the present author – is the writing style found in Volume 1. In addition to the technical guidance, Professor Baker offers a commentary on the changes to BS 449, the design standard of the time. The Professor had proposed revised approaches which were not adopted – the text reveals his disappointment often in an amusing style rather different to most sterile discussions found in text books. Although references were made to Professor Baker's comments in the *New Steel Construction* article of September 2020, this article looks at some of the background and the sometimes pointed comments found in Volume 1.

The Background

Back in 1929 a Steel Structures Research Committee was established to develop the existing design rules which were felt to be too conservative. The Committee produced a *First Report* in 1931, which was embodied in BS 449. This report did not include the results of any new research, but was based on current practice. The Committee continued its work, including tests on existing buildings and showed that the method of design inherent in BS 449 was “almost entirely irrational and therefore incapable of refinement”. Professor Baker did not mince his words!

In 1936, the Committee produced a *Final Report* containing completely new “rational” design rules. When BS 449 was revised some 12 years later, in 1948, the Committee's recommendations were ignored. Professor Baker postulates one reason for this neglect may have been that the store of *Final Report* was destroyed by enemy action in World War II. Professor Baker does not hide his disappointment in Volume 1, describing the guidance in BS 449 as “irrational rules”. Part of Professor Baker's introduction expresses his view of the Committee's objectives:

“While the pure scientist, secure in the ‘disinterested’ status of his research, can often follow the path of least resistance, skirting some nasty jagged rocks, the applied scientist must concentrate on those difficult places. He must get to the top of a particular rock. Not content with that achievement, he must prepare an easy road so that others, with less time and less elaborate equipment, can get there without difficulty whenever they wish”.

One of the founding objectives of SCI was to make steel easy to choose and use. Not so elegant as Professor Baker, but the same ideal.

Effective lengths

Professor Baker is most exercised by column design as presented in BS 449 (notably the very same requirements are followed in BS 5950 and presented in NCCI for design to EN 1993). When deciding column effective lengths, designers are directed to consider the restraint offered by beams connected to the column. Professor Baker describes the clauses as “irrational because no mention is made of the stiffness of the beams or the rigidities of the connexions on which the restraint at the end of the stanchion length must depend. Worse than this, such argument that there is for the assumption of an effective length less than the actual length is only tenable for the condition of

pure axial load which is found nowhere but in that rare member, a symmetrically loaded internal stanchion”.

Ahead of his time?

Professor Baker also notes that as a column deflects under axial load, “restraining moments are introduced at the end of each storey length by the beams, which frame into the stanchion through connexions of some rigidity, offering resistance to the change of slope. The presence of these restraining moments means that a member of given section can carry a greater axial load before the yield stress is developed than would have been the case had it been pin-ended”. Professor Baker points out the inconsistency of designing a beam as pin-ended, then “remembering” the rigidities of the connections when determining an effective length, and then conveniently forgetting the rigidity of the connections when calculating applied moments introduced by eccentric reactions.

Some 40 years later, Gibbons *et al*¹ considered the same issue. The research investigated non-sway frames, aimed at quantifying the adverse effect of the moment transferred through simple beam to column connections on the column capacity. The study showed that in many cases, the benefit of the rotational restraint offered by the connection outweighs the detrimental effect of the moment transferred through it – exactly as Professor Baker had observed in the 1930s. Gibbons *et al* also concluded that the present methods of simple frame design (still used today in 2022) are highly conservative, sharing that view with Professor Baker. A second shared opinion is that “this approach”... (the verification of columns in braced frames in clause 4.7.7 of BS 5950) ... “does not assist in the understanding of true behaviour and is not therefore likely to facilitate innovative design. Indeed, reliance on such approaches can be seen as a definite barrier to progress as the designer is not encouraged to develop a physical understanding of the way in which the structure actually ‘works’”. Not as colourful language as Professor Baker, who wished to stop designers “blindly applying irrational rules”, but the same sentiment!

Choice of a strain gauge

The Committee wished to undertake testing of physical buildings – and required a reliable strain gauge which would deliver a high order of accuracy “maintained under conditions of dirt, vibration, exposure and hurry which would seem intolerable to the laboratory worker”. The extensive reporting of the Committee's efforts to obtain a strain gauge meeting those demands are entertaining and entirely unexpected in a highly technical publication. Professor Baker reports that “Every known type of strain gauge was examined to see if one could be modified to fit the rigorous specification”. A brochure was obtained describing an instrument, the Maihak extensometer, of German origin, as shown in Figure 1 (over page).

We read that “The claims made for it seemed so fantastic that the brochure was relegated to the waste-paper basket. When all other hope had gone, the waste-paper basket was searched and the brochure again studied”. The instrument involved an audible comparison between the note from a stressed wire (on the structure) and a gauge wire in the instrument. The initial tests were disappointing – “Other observers were called in, some with an ear for music, some without. The results were uniformly bad, and so with regret, the agent, who had brought the instrument to the test house, was dismissed”.

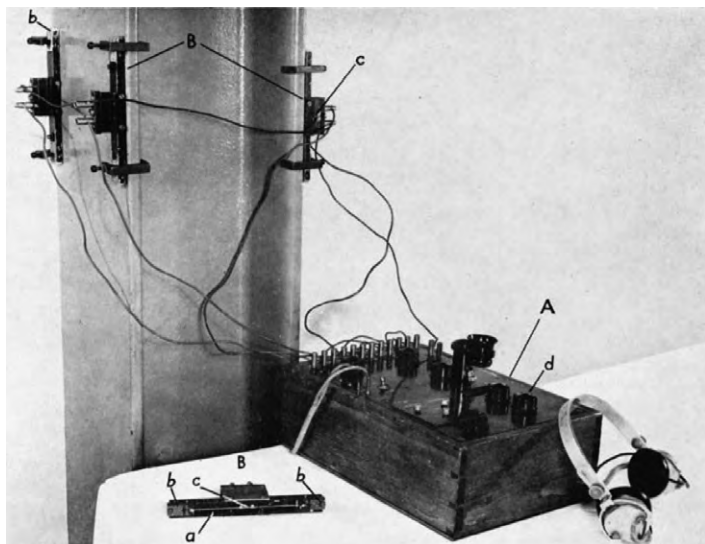


Figure 1: Maihak extensometer

The reader then learns that on reflection, the Committee decided that perhaps greater amplification of the sounds was needed. The agent was called (who had conveniently mentioned where he was staying) and his departure delayed. This second test took place in the agent’s bedroom, with the instrument clamped to the agent’s brass bedstead. It is reported that “the scene was set – the morse key was pressed and a strong sustained note was heard. The investigator, full of hope, proceeded to match the notes, easily detected the beats and attempted to eliminate them by turning the micrometer head. Just when success seemed to be within his grasp, the beats would mysteriously appear again then disappear and so on.” They were about to condemn the instrument, but noticed that the agent was leaning over to look at the micrometer reading – “in doing so his weight came on the bed rail and so subjected it to a not inconsiderable strain which the gauge reported”. We go on to read that the party returned to the test house “triumphantly”. The prose in our modern reports, technical papers and publications appears rather bland in comparison.

Vibrating wire strain gauges using the principles of the Maihak extensometer are still used today – as seen in Figure 2.



Figure 2: Vibrating wire strain gauges installed on steelwork (courtesy Sisgeo.com)

Neglected recommendations

In summary, the Committee’s proposals for a more rational design method recommended using the beneficial effects of connection stiffness for beam design. For column design, rigid connections are assumed, but the moments in the column are determined from the stiffnesses of the beams and columns. The Committee’s recommendations were not accepted. The additional effort in the proposed design method did not deliver an advantage “which the average client would appreciate”. The proposed method would mean that a structure designed as a residential building, on change of use “might well prove strong enough for use as a light warehouse”. Again in eloquent language, Professor Baker envisaged a spreading of suburbia, meaning that urban flats might be reused as warehouses. In recent times, the opposite is true – urban structures are converted for residential use. At the time, Professor Baker commented that “Though the representatives of local authorities delight to argue, when faced with the request that a block of flats be designed for the intensity of load actually to be experienced in them, that the west-end of every city is continually moving farther westward so that the fashionable flats today will be the warehouses of tomorrow, such considerations are unlikely to weigh heavily with the prospective building owner”

BS 449 of 1948

There is understandable bitterness when Professor Baker comments on the 1948 revision of BS 449 which declined to adopt the Committee’s recommendations. He comments that the Committee’s work has been disregarded “and the designer has taken a step in the dark. If his position could be viewed dispassionately, it would be interesting to see if there was a tendency for the ground to crumble under his feet, for nothing has such a tonic effect as a threatened disaster”. Strong language indeed! Experience since 1948 suggests that the design methods remain safe, if conservative.

One of Professor Baker’s comments was that the recommendations failed “first because they did not lead to certain economy of material and second because they were complex”. It is interesting to reflect on the transition from BS 449 to BS 5950, the latter being considered more involved but producing some economy. One wonders what would be thought of the subsequent transition to the Eurocodes, with numerous Parts, National Annexes and NCCI. ■

1. Gibbons, C. Nethercot, D. A. Kirby, P.A. Wang, Y. C. *An appraisal of partially restrained column behaviour in non-sway steel frames* Proceedings of the Institution of Civil Engineers. Structures and Buildings, 1993

Composite slabs with in-plane loading

Although composite slabs are normally designed as one-way spanning and subject to (out-of-plane) gravity loading only, they often need to do more than this. Graham Couchman of the SCI discusses aspects of composite floor behaviour when subject to in-plane loading. The use of shear studs on non-composite beams is explained and justified, and guidance is given on how to design slabs subject to large in-plane forces.

Introduction

Two aspects of composite floors that feature behaviour that is sometimes not fully considered relate to edge beams that are designed non-compositely but are provided with shear studs, and the effect of in-plane forces in the slab due to diaphragm, or similar, action. These behaviours are considered below.

Edge Beams

At the SCI Advisory Desk we often get asked about the provision of shear studs on edge beams, where the studs are not needed to provide composite action for the beam, but rather to provide a structural connection to tie the slab to the beam. This makes sense, with detailing rules identifying any special provisions for slab geometry and reinforcement – U-bars must pass around the studs when the distance to the edge of slab is less than 300 mm, because straight bars would have insufficient anchorage.

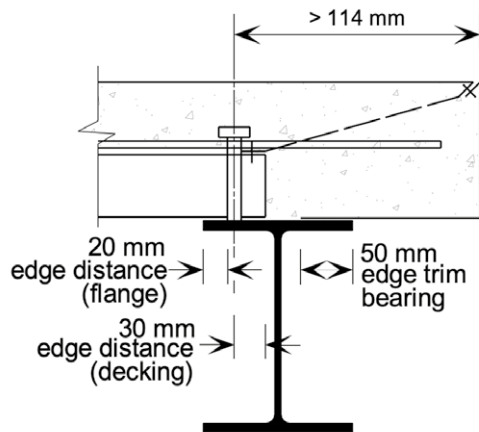


Figure 1: U-bar reinforcement wrapped around shear studs on an edge beam

Of course, even though the beam is designed to be non-composite, it doesn't know that. Because the shear studs have some stiffness they cannot avoid attracting force as the concrete slips over the top of the downstand beam, so some composite action is going to be developed. A question then arises, do the rules for minimum degree of shear connection need to be satisfied?

To recap, the reason why all composite beams must have a minimum degree of shear connection is to ensure that there are enough studs (with a high enough collective stiffness) to limit the slip at the slab to beam interface. Rather than bothering designers with a need to explicitly consider stud stiffness, which incidentally will be between 50 kN/mm and 100 kN/mm depending on stud size and the slab, the rules for minimum degree of connection assure the provision of a certain level of resistance, to indirectly ensure the total stiffness of the studs present is sufficient. The rules presented in EN 1994-1-1¹, and previously BS 5950-3.1², were based on

ensuring that the slip at the beam ends (where it is at a maximum) did not exceed 6 mm at ULS. More recent work, for example the more relaxed rules in SCI publication P405³, recognise that in some situations the stud resistance can be maintained to higher levels of slip. For example, in the presence of transverse trapezoidal decking 10 mm can be achieved. An ability to accommodate greater slip means that fewer studs are needed, because their collective stiffness can be lower. It is worth adding that in addition to needing slip capacity, studs must be stiff enough so that the slip at SLS levels of loading is sufficient to mobilise the shear studs, and therefore justify use of their resistance in design. A shear connector that lacked initial stiffness might have a high resistance, and indeed high slip capacity, but slips would never be high enough to make it 'work' properly. Generation 2 EN 1994⁴ addresses this particular aspect by introducing a number of different ductility classes, compared to the previous ductile/non-ductile 'switch'.

All this would make one believe that the minimum degree of shear connection rules should be satisfied, even for beams that are designed to work non-compositely. Failure to do so would result in the studs 'unzipping' as the end slip exceeded their slip capacity as load was increased. However, consideration of the load-slip behaviour for a shear stud, typically determined by a push-out test (Figure 2a), shows us why this is not in fact the case.



Figure 2a: Push out test specimen

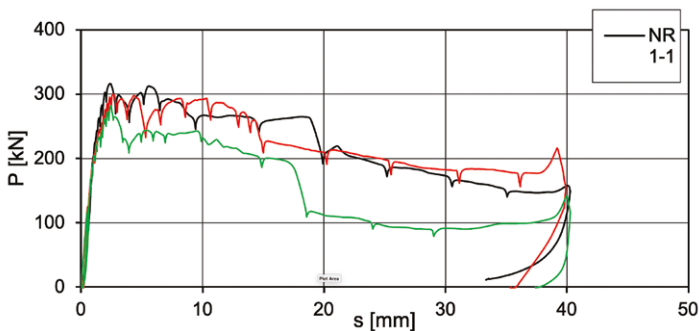


Figure 2b: Load-slip curves determined by a push-out test (courtesy University of Luxembourg)

Figure 2b shows that although there is a value of slip beyond which the stud resistance reduces (for these curves, which were for specimens with transverse trapezoidal deck, a slip of between 10 and 15 mm), it is not a brittle fall-off. The curves in Figure 2b show that the test set-up only allowed slips of up to 40 mm to be recorded (this would have been the distance that the steel section initially protruded above the slabs, see Figure 2a). EN 1994-1-1 defines behaviour from a load-slip curve in terms of:

- Characteristic resistance is the minimum failure load (from a group of three tests) reduced by 10%
 - Provided the deviation of any single result does not exceed the mean of all tests by more than 10%.
- The slip capacity of an individual test is the maximum slip at the characteristic resistance, and the characteristic slip capacity is the lowest slip from a set of three tests, reduced by 10%.

Beyond the characteristic slip capacity, the characteristic (and therefore design) resistance of the stud cannot be assured. But of course if the beam has been designed to work non-compositely, then it doesn't matter if the stud cannot provide its design resistance. All that matters is that it remains attached to the beam, and so continues to tie the slab to the beam. For the example curves shown in Figure 2b this would remain the case for slips up to 40 mm. Taking the extreme case of zero collective stud stiffness, slip would not be expected to exceed such a value. This qualitative conclusion has been quantified by researchers in the past, for example reference 5 used numerical analysis to predict 15 mm slip for a typical size beam over a 1.5 m span. This finding can also be confirmed by considering the elastic behaviour of a beam, given the radius of a beam in bending $R = M/EI$, and using this radius to predict end slope and thereby the shortening of the top of the beam. It can also be understood by the analogy of bending two rulers, one placed on top of the other. For 'one foot' rulers one would find approximately 1 mm slip – zero shear connection stiffness does not result in infinite slip.

The studs on an edge beam that tie the beam to the slab transversely will be subject to a combination of shears in two directions, and some interaction that would potentially reduce the shear resistance can be envisaged. For beams that are designed to be non-composite this is irrelevant. For the alternative case where the edge beams (or others) are designed to be composite, EN 1994-1-1 6.6.4.3 provides an interaction equation:

$$\frac{F_{t, Rd}^2}{P_{t, Rd}^2} + \frac{F_{r, Rd}^2}{P_{r, Rd}^2} \leq 1.0$$

For beams supporting composite slabs with trapezoidal decking this may be conservative, given that in the direction of the decking ribs the concrete near the base of the studs is critical, but in the orthogonal direction failure is normally due to failure of a concrete surface passing over the studs.

In-plane diaphragm action

Turning to the slab, there may be a concern that a slab that is loaded in-plane, in whatever direction, may experience different stresses at the interface between the decking and concrete compared to a slab loaded purely in bending. It is easy to imagine that a slab that was lozengeing would have a tendency to break the shear bond. Provided in-plane stresses and strains are small, practically it is suggested that such effects can be ignored. The tests used to determine shear bond include a cyclic element to destroy chemical adhesion, so the shear bond used in design is solely due to mechanical interlock and one would expect this to be sufficiently robust.

In extreme cases a slab may be subject to significant in-plane, and coincident out-of-plane, loading. The former could be due to thermal movements. Little is known on how composite slabs would then behave. For example, the shear bond would presumably be adversely affected if the concrete was in tension, and therefore cracked, throughout its depth. In a standard test to determine the level of shear bond that can be generated by a given deck, whilst the extreme (lower) fibres of the concrete will indeed be in tension, those above the deck neutral axis will not. Evidence suggests that much of the shear bond generated by trapezoidal decks is due to the embossments and form of the top flange of the decking. In the absence of any evidence from testing or numerical modelling, the most robust approach in such a situation could be to:

- Design the slab as a reinforced concrete element carrying coincident in-plane and out-of-plane loading
- Use the decking as permanent formwork (ignore any shear bond)

If necessary the decking could also be used to add to the vertical shear resistance of the slab. Its ability to contribute has recently been recognised during the process of developing the Generation 2 EN 1994. The relevance of it being able to do so would be particularly high in slabs that had in-plane tension, the presence of which reduces the shear resistance of the concrete (which is the only contributing element according to the current EN 1994 rules, which are taken from EN 1992).

Conclusions

As engineers we often use our judgement to make pragmatic decisions, and one common example has been the use of apparently insufficient shear studs on non-composite edge beams. No matter what we assume, in this case that the beams are non-composite, the physics of elements with stiffness attracting load cannot be ignored. The discussion above shows why the approach of seemingly adding too few studs can indeed be justified. Consideration is also given to composite slabs in which significant in-plane forces coincide with out-of-plane loading, with a conclusion that such slabs should not be designed compositely (using available shear bond data) without justification. A traditional reinforced concrete approach should be adopted. ■

1. BS EN 1994-1-1:2004. *Eurocode 4: Design of composite steel and concrete structures. General rules and rules for buildings*. BSI, 2005
2. BS 5950-3.1:1990+A:2010. *Structural use of steelwork in buildings. Design in composite construction. Code of practice for design of simple and continuous composite beams*. BSI, 2010 (Superseded).
3. P405. *Minimum Degree of Shear Connection Rules for UK Construction to Eurocode 4*. Graham Couchman. SCI, 2015.
4. prEN1994-1-1. *Project Team Final draft April 2021* (this document is not publicly available)
5. *Composite Construction in Steel and Concrete V*. Edited by Roberto Leon and Jorg Lange, ASCE, 2004.

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Advisory Desk 2022

AD 471:

Second order moments for splice design

AD notes 243 and 244 were produced when BS 5950:1990 was current. The present AD note provides an update for designers using BS EN 1993-1-1. The same principles are followed as those set out in the approach in BS 5950. The general advice on the design of splices in AD 243 is still relevant.

Splices in compression members designed in accordance with BS EN 1993-1-8 sub-para. 6.2.7.1(15) that are not close to points of restraint are subject to second order effects that must be taken into account. AD 470 identified these effects as the strut moment and, if design bending moments are present in the member, such moments are amplified by the axial force.

The complete list of internal second order moments that may be present in a splice connection is as follows:

- 1) The strut moments $M_{y,FB}$ and $M_{z,FB}$ due to the initial bow in the compression member amplified by the axial force (FB denotes flexural buckling);
- 2) A minor-axis moment $M_{z,LTB}$ produced by resisting lateral-torsional buckling due to major axis moments (LTB denotes lateral torsional buckling);
- 3) An additional major-axis moment $M_{y,Amp}$ due to applied major-axis moments amplified by the axial force;
- 4) An additional minor-axis moment $M_{z,Amp}$ due to applied minor-axis moments amplified by the axial force.

The second order moments listed above are assumed to vary as the sine over the length between points of inflexion in the buckled shape (the effective length). The maximum second order moment (at the mid-span of an unrestrained segment) is calculated and adjusted to the appropriate value at the splice location, positioned at a distance x along the length L between points of inflexion.

The external design forces and moments are N_{Ed} , $M_{y,Ed}$ and $M_{z,Ed}$ (axial force, major-axis bending and minor-axis bending, respectively) where the bending moments are the applied moments at the splice position. The bending moments used to determine the additional second order moments (items 3 and 4 above) are the maximum values of the applied moments about the relevant axes in the unrestrained buckling segment being considered (where the splice is located). The shape of the bending moment diagrams between points of inflexion can be accounted for by considering equivalent uniform moment factors.

Columns subjected to axial compression:

The second order effects described in Item 1 above need to be considered. The strut moments $M_{y,FB}$ and $M_{z,FB}$ should be considered about each axis but only about one axis at a time (the column cannot buckle about both axes at the same time). The second order bending moments due to strut action at the mid-span between points of inflexion can be calculated as follows:

$$M_{i,FB,max} = N_{Ed} \cdot e_{P\delta,i} \cdot \gamma_{M1}$$

where:

$e_{P\delta,i}$ is the bow imperfection accounting for the second-order effects equal to:

$$e_{P\delta,i} = e_{0,i} \cdot k_{amp,i} \text{ but } e_{P\delta,i} \geq L/200 \text{ if } N_{Ed} \geq 0.90 N_{b,Rd,i}$$

$e_{0,i}$ is the initial bow imperfection about axis 'i' equal to:

$$e_{0,i} = \frac{W_{el,i}}{A} \cdot \alpha \cdot (\bar{\lambda}_i - 0.20)$$

$k_{amp,i}$ is the amplification factor equal to:

$$k_{amp,i} = \frac{N_{cr,i}}{N_{cr,i} - N_{Ed}}$$

$N_{b,Rd,i}$ is the design flexural buckling resistance of the column according to EN 1993-1-1 section 6.3.1 about axis 'i';

γ_{M1} is the material partial factor for buckling resistance given by the relevant EN 1993 part;

$W_{el,i}$ 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 related to flexural buckling about axis 'i';

$\bar{\lambda}_i$ is the non-dimensional slenderness according to EN 1993-1-1 section 6.3.1.2 related to flexural buckling about axis 'i'

$N_{cr,i}$ is the elastic critical buckling load for flexural buckling (Euler load) about axis 'i'.

Note that the amplification factors $k_{amp,i}$ are equivalent to the Eurocode presentation in equation 5.4. For an individual element, $\alpha_{cr} = N_{cr,i} / N_{Ed}$, meaning that:

$$\frac{1}{1 - \frac{1}{\alpha_{cr}}} = \frac{1}{1 - \frac{1}{N_{cr,i}/N_{Ed}}} = \frac{N_{cr,i}}{N_{cr,i} - N_{Ed}}$$

Whilst the model described above can suitably address the phenomenon in most cases, studies have shown that for columns with low slenderness loaded close to their buckling capacity ($N_{Ed} \geq 0.90 N_{b,Rd}$), the second order effects could be underestimated. The recommended minimum value of $L/200$ for the bow imperfection accounting for the second-order effects ($e_{P\delta,i}$) ensures that the model gives safe results for such cases.

The strut moment at the splice position between points of inflexion ($M_{i,FB,sp}$) is given by:

$$M_{i,FB,sp} = \sin\left(\pi \cdot \frac{x}{L}\right) \cdot M_{i,FB}$$

If second order effects in frames have been accounted for using equivalent members (i.e. increasing the buckling lengths of columns) in accordance with 5.2.2(c), the moment at the splice location should be taken as $M_{i,FB}$.

The combined design actions to design the splice are:

$$1: N_{Ed} + M_{y,FB,sp} \text{ or } 2: N_{Ed} + M_{z,FB,sp}$$

Unrestrained beams subjected to bending moments:

The second order effects described in Item 2 above need to be considered for elements subjected to major axis moment. The second order moment due to lateral-torsional buckling at the mid-span between points of inflexion can be calculated as follows:

$$M_{z,LTB,max} = \left(\frac{1}{\chi_{LT}} - 1\right) \cdot \frac{W_{el,z}}{W_{el,y}} \cdot M_{y,Ed,max} \cdot \gamma_{M1}$$

$$\text{but } M_{z,LTB,max} \geq \frac{6 \cdot E \cdot I_z}{125 \cdot L} \text{ if } M_{y,Ed,max} \geq 0.90 M_{b,Rd}$$

where:

$W_{el,z}$ is the minor axis elastic modulus;

$W_{el,y}$ is the major axis elastic modulus;

$M_{y,Ed,max}$ is the maximum major-axis design bending moment along the length L between points of inflexion;

χ_{LT} is the buckling reduction factor for the relevant buckling curve according to EN 1993-1-1 section 6.3.2;

$M_{b,Rd}$ is the design lateral torsional buckling resistance according to EN 1993-1-1 section 6.3.2;

I_z is the second moment of area about the minor-axis z .

For cases where section EN 1993-1-1 sub-para. 6.3.2.3 (2) is applied, $\chi_{LT,mod}$ may be used to evaluate $M_{z,LTB,max}$.

Studies have also shown that beams with low slenderness loaded close

to their buckling capacity ($M_{y,Ed,max} \geq 0.90 M_{b,Rd}$), the second order effects could be underestimated. The recommended minimum second order bending moment allows for a minimum out-of-plane bow at failure of $L/200$, conveniently expressed as a minimum bending moment.

The second order moment at the splice position between points of inflexion is given by:

$$M_{z,FB,sp} = \sin\left(\pi \cdot \frac{x}{L}\right) \cdot M_{z,LTB,max}$$

The assessment of the location of the points of inflexion may not be apparent to the designer and difficult to determine for some bending moment diagram shapes. Therefore, the definition of the the second-order bending moment at the splice location (from a distance x along the length L between points of inflexion) can be challenging. As a simplification, designers may wish to assume $M_{z,LTB,sp} = M_{z,LTB,max}$. For segments with a reasonably uniform bending moment or for simply supported beams with uniformly distributed loads, the problem is simplified and the equation above can be used.

The combined design actions to design the splice are:

$$M_{y,Ed} + M_{z,Ed} + M_{z,LTB,sp}$$

Beam-columns subjected to axial compression and bending moments:

The second order effects described in Items 1 to 4 above need to be considered for elements subjected to axial compression and bending. The additional second order moments due to amplification of the applied moment about the axis ‘i’ by the axial force are given by:

$$M_{i,Amp,max} = (k_{amp,i} - 1) \cdot C_{mi} \cdot M_{i,Ed,max}$$

where:

$M_{i,Ed,max}$ is the maximum design bending moment along the length L between points of inflexion (about axis ‘i’);

C_{mi} is the equivalent uniform moment factor, given by EN 1993-1-1 Table B.3 about axis ‘i’ (C_{my} or C_{mz}).

The second order moment at the splice position between points of inflexion ($M_{i,Amp,sp}$) is given by:

$$M_{i,Amp,sp} = \sin\left(\pi \cdot \frac{x}{L}\right) \cdot M_{i,Amp,max}$$

The combined design actions at the splice position are:

1: $N_{Ed} + M_{y,Ed} + M_{z,Ed} + M_{y,FB,sp} + M_{y,Amp,sp} + M_{z,Amp,sp} + M_{z,LTB,sp}$

2: $N_{Ed} + M_{y,Ed} + M_{z,Ed} + M_{z,FB,sp} + M_{y,Amp,sp} + M_{z,Amp,sp} + M_{z,LTB,sp}$

The splice should be verified for both combinations.

If $\frac{N_{Ed}}{N_{b,Rd,i}} + \frac{M_{Ed}}{M_{b,Rd}} > 0.9$ then:

$M_{i,FB,sp}$ should be calculated as if $N_{Ed} > 0.9N_{b,Rd,i}$ as given above for columns, and

$M_{z,LTB,sp}$ should be calculated as if $M_{Ed} > 0.9 M_{b,Rd}$ as given above for

unrestrained beams.

Complementary information

Further detail and calculation examples are provided in References 1 and 2. Details of the studies completed to establish the minimum values of $e_{P\delta,i}$ and $M_{z,LTB,max}$ referred to will be described in an NSC paper to appear at a later date.

Contact: **Richard Henderson**

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1. Henderson R, *Bearing splice in a column*, NSC, March 2020
2. Pimentel R, *Design of beam-column splice connections according to Eurocode 3*, NSC, October 2020

AD 475:

Buckling of cruciform columns

The SCI’s Advisory Desk has been asked about the design of cruciform columns to Eurocode 3. This structural shape is sometimes adopted for architectural reasons. Its behaviour is unusual when the section is made of plates without flanges and has rotational symmetry of four, in the respect that the likely critical buckling mode is torsional, not flexural.

Unusually, the formula for the elastic critical torsional buckling force for an I section is given in BS EN 1993-1-1 and appears in para BB.3.3.1:

$$N_{crT} = \frac{1}{i_s^2} \left(\frac{\pi^2 EI_z a^2}{L_t^2} + \frac{\pi^2 EI_w}{L_t^2} + GI_T \right)$$

In this formula, $i_s^2 = i_y^2 + i_z^2 + a^2 =$ and a is the distance between the axis of rotation and the shear centre of the section. L_t is the length between torsional restraints. The intersection of the rectangular elements that form the cross section is its shear centre and when rotation occurs about this axis, the value of a is zero. As the section is bi-symmetric, the shear centre coincides with the geometric centroid of the cross section.

The absence of flanges at the ends of the plates remote from the shear centre results in a zero value for the warping constant I_w .

Making these simplifications means that the formula for N_{crT} reduces to:

$$N_{crT} = \frac{A}{I_y + I_z} GI_T$$

For a cruciform column with end moments, a lateral torsional buckling check can be carried out using the general formula for lateral torsional buckling in BS EN 1993-1-1. The value of M_{cr} can be determined using the same formula as that for a flat plate:

$$M_{cr} = \frac{\pi}{L} \sqrt{EI_z GI_T}$$

This formula is relevant to a uniform moment. Useful references are Design of cruciform sections using BS 5950-1:2000¹, AD391² and Timoshenko and Gere³.

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1. Charles King, *Design of cruciform sections using BS 5950-1:2000*, NSC, April 2006
2. AD391: *Lateral torsional buckling of rectangular plates in accordance with BS EN 1993-1-1*, SCI
3. Timoshenko, SP and Gere, JM, *Theory of elastic stability, 2nd Edition*, Dover Publications Inc, 2009

AD 477:

Transverse bending of composite slabs subjected to point loads

AD note 450¹ provides guidance on the design of composite slabs subjected to concentrated point loads. The present AD note provides further guidance on the topic. The method to calculate the applied transverse bending moment proposed by AD 450 is a simple approach². The first reference to such a

method may be traced back to 1994³, where the author acknowledges that the method is conservative. The present note proposes a more sophisticated approach, as a supplement rather than a replacement for the approach in AD 450. This more advanced approach is particularly relevant for the UK market, where typically we rely on a single layer of reinforcement with a minimal cover to the top surface of the slab, for which the simplified method proposed by AD note 450 may result in an onerous requirement for the area of reinforcement required to resist transverse bending. Two layers of reinforcement, or a single layer placed lower in the slab, are typical in most other European countries. As we are seeing more use of deeper slabs in the UK, adding a second layer may be a sensible option.

Despite the fact that high point loads on completed composite slabs are normally only found in particular cases (such as a car park), during the construction stage of a building they are much more common, as the slabs may be required to resist high point loads due to temporary equipment such as mobile elevating work platforms (MEWPs). However, the coincident load level on the slab under such conditions is relatively low (self-weight of the slab, live load due to the construction stage and the equipment itself). Spare capacity is therefore expected, resulting in a low level of utilisation for the longitudinal slab design, for this temporary condition.

The applied transverse bending moment ($M_{Ed,t}$) is given in AD 450 as a function of b_{em} , a_m , b_m and Q_{Ed} as shown in Equation (1). The variables involved in the method are illustrated in Figure 1. The two main topics of discussion of this AD note are the variables b_{em} and a_{em} . The former has a direct influence on the level of applied transverse moment, and the latter determines the resistance to transverse bending, as will be seen below.

AD note 450 guidance:

AD 450 proposes a value for the applied moment $M_{Ed,t}$, which is supported by a transverse strip of width a_m . Variables are as defined below:

$$M_{Ed,t} = \frac{Q_{Ed} (b_{em} - b_m)}{8 a_m} \text{ (kNm/m)} \quad (1)$$

$$a_m = a_p + 2(h_c + h_t) \quad (2)$$

$$b_m = b_p + 2(h_c + h_t) \quad (3)$$

For a simply supported composite slab:

$$b_{em,max} = b_m = 2L_p \left[1 - \frac{L_p}{L} \right] \leq \text{slab width} \quad (4)$$

b_{em} :

Eurocode 4 provides maximum admissible values for the effective width b_{em} of the longitudinal strip of slab that is assumed to support a point load (as the Eurocode rules are based on maximizing the resistance of the slab in the longitudinal direction). The width b_{em} is a function of the transverse stiffness of the slab, the distance from the point load to the nearest support and the point load contact area. The ENV (so-called pre-standard) version of Eurocode 4⁴, explicitly stated that if transverse bending of the slab is ignored, the width b_{em} could be taken as simply b_m (and no transverse reinforcement would then be needed). Intermediate values for the effective width of the longitudinal strip of slab b_{em} , between b_m and the value reported by Eurocode 4 (denoted from here on as $b_{em,max}$), may also be considered. Reducing the width $b_{em,max}$ given by Eurocode 4 would make the longitudinal slab design more onerous, but benefit the transverse slab design when compared with applying the simplified method proposed by AD 450. This benefit arises because the width of the longitudinal strip is also the span of the transverse strip, and the greater that span the more reinforcement is needed to resist a given load. Varying the assumed width of the longitudinal strip may be particularly helpful for the cited cases of composite slabs subjected to point loads during the construction stage, where the designer has very limited options.

a_{em} :

AD 450 assumes that the effective width of the transverse strip of slab over which the point load is carried is limited to a_m . The cited conservatism in the design method is mainly related to this assumption. The effective width of the transverse strip that may actually be mobilized (denoted from here on as a_{em}) is greater than a_m . Given that the transverse stiffness of the slab is assumed to be sufficient to spread a point load over a longitudinal strip of width b_{em} , it is clear that a width greater than a_m can be mobilized for transverse bending. Equation (5) proposes a maximum value for the effective width of the transverse strip of slab subject to transverse bending ($a_{em,max}$). The applied moment per metre

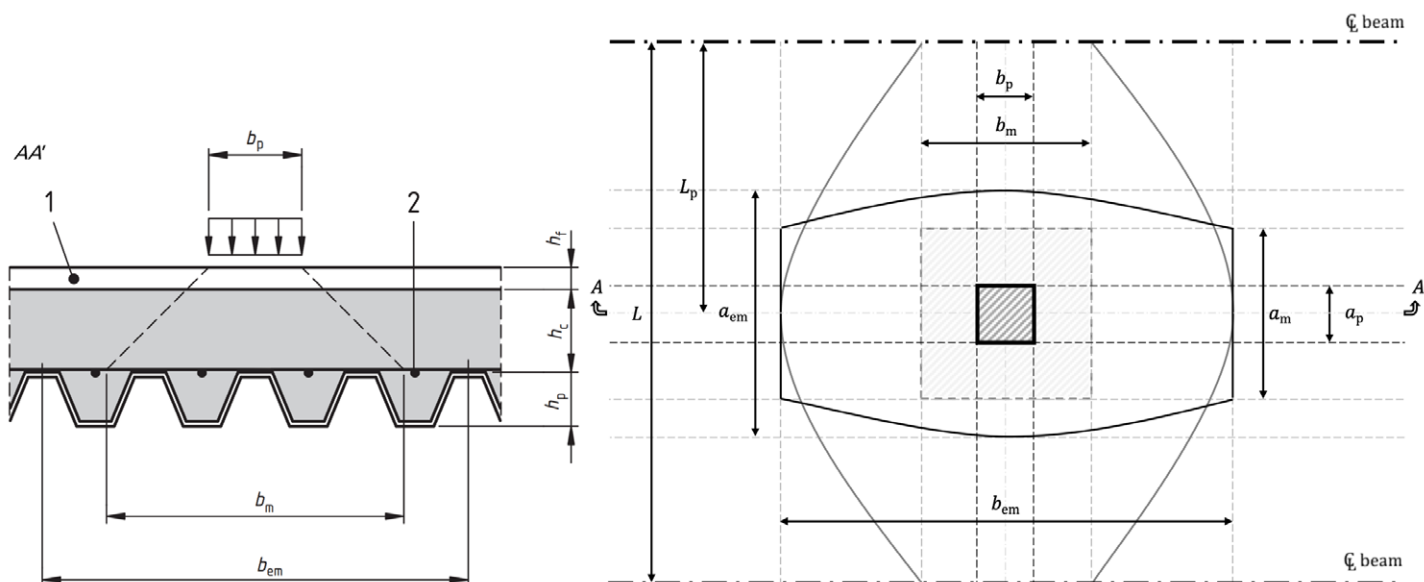


Figure 1: Widths associated with a concentrated load (1 indicates topping, 2 the slab reinforcement, noting that in typical UK practice it would not be placed as shown)

may therefore be determined using Equation (6). This leads directly to a specification of the area of transverse reinforcement needed per metre.

$$a_{em,max} = a_m + \frac{b_{em}}{3} \text{ but } a_{em,max} \leq \text{Slab span} \quad (5)$$

$$M_{Ed,t} = \frac{Q_{Ed}(b_{em} - b_m)}{8 a_{em}} \text{ (kNm/m)} \quad (6)$$

The transverse bending moment given by Equation (6) is applicable to composite slabs with $I_t/I_1 \leq 0.50$, where I_t and I_1 are the uncracked transverse and longitudinal second moment of areas of the slab, respectively.

Comments

Using the limiting value of $a_{em} = a_m$ has the advantage that any other point load present in the direction of the slab span will not overlap (assuming a minimum point load spacing of a_m , which is sensible for practical cases). So, the design of transverse reinforcement for each point load can be treated independently. When assuming a_{em} greater than a_m , the designer may need to consider the overlapping effect of adjacent point loads that are trying to mobilise the same part of the slab. Reinforcement requirements would need to be additive. The designer should also consider the proximity of other point loads in the transverse slab direction, which may result in an overlapping of the attributed effective slab widths for longitudinal bending b_{em} . The definitions of $b_{em,max}$ and $a_{em,max}$ in Equations (4) and (5), respectively, are intended to provide a sensible compromise between competing demands.

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1. AD 450: *Resistance of composite slabs to concentrated loads*, SCI, October 2020.
2. Johnson, R. P, Wang, Y. C., *Composite Structures of Steel and Concrete*, Fourth edition, 2019; Wiley Blackwell.
3. *Composite structures of steel and concrete* R.P. Johnson, Blackwell Publishing, 2nd edition 1994.
4. 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.

AD 480: Correction to P399 Appendix F Worked Example

It has been brought to our attention that in the worked example presented in SCI publication P399 Design of steel portal frame buildings to Eurocode 3, the position of the intermediate lateral restraint determined in section F3.5.4 *Plastic verification of the haunch*, has been miscalculated.

In section F3.5.4 the necessary steps in the verification of the haunched part of the portal frame rafter are presented, assuming a plastic hinge is present in the rafter at the sharp end of the haunch. The verification assumes a torsional restraint at the plastic hinge and finds the position of a second torsional restraint to the haunch at a distance L_s from the plastic hinge. An intermediate lateral restraint to the top (tension) flange is required between the torsional restraints at a distance L_m from the plastic hinge. Further tension flange restraints may also be required.

Finding L_m involves satisfying equation BB.9 in BS EN 1993-1-1 para. BB.3.2.1:

$$L_m = \frac{38i_z}{\sqrt{\frac{1}{57.4} \left(\frac{N_{ed}}{A}\right) + \frac{1}{756G_1^2} \left(\frac{W_{ply}^2}{AI_T}\right) \left(\frac{f_y}{235}\right)^2}}$$

Parameter A is defined as the cross sectional area in mm² at the location where the quotient

$\left(\frac{W_{ply}^2}{AI_T}\right)$ is a maximum of the tapered member (ie in the length L_s).

$\left(\frac{W_{ply}^2}{AI_T}\right)$ is defined as the maximum value in the segment (ie in the length L_m).

Finding L_m involves iteratively assuming a trial length to determine the parameters in the equation and comparing the calculated value with the trial value. In the example, in determining the length L_m , the quotient $\left(\frac{W_{ply}^2}{AI_T}\right)$ was taken as the value at the position defined by L_s in error. The value of the quotient should be taken as that at the trial length L_m . Adopting the value at L_s results in a smaller value of L_m which is on the safe side.

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AD 481: Composite beams with deep composite slabs

It has come to SCI's attention that composite slabs with an overall depth/thickness in the range of 200 mm to 250 mm are becoming increasingly common, particularly in cases where significant concentrated loads are involved, or where serviceability requirements (e.g. floor vibration) or other criteria (e.g. acoustic performance) that are specific to a particular project need to be satisfied. This has reportedly caused problems from a composite beam design perspective, and designers are often finding it difficult to satisfy the minimum degree of shear connection requirements. This is because the scope of SCI P405¹ limits the overall depth of a composite slab to 180 mm and requires that the depth above the profile does not exceed 100 mm, therefore not allowing its use for deeper slabs. The question then is whether the above limits within P405 could be relaxed (or removed), and whether there are any other reasons to limit the depth of a composite slab.

SCI P405 and minimum degree of shear connection requirements

The scope of P405, and more specifically the slab depth limitation, was based on the range of configurations investigated (geometries used in the finite element analyses). Also, it was based on what we thought (at the time) was the range that covered most practical cases. Since then, the SCI has carried out additional investigations on simply supported composite beams with deep composite slabs. The effect of the slab depth in terms of the minimum degree of shear connection requirements is explained in the following paragraphs.

By increasing the depth of the composite slab, the bending resistance of the composite beam also increases. This increase is relatively modest, particularly when the degree of shear connection is low (i.e. cases that would benefit more from P405), but it can become more significant for higher degrees of shear connection that allow greater force to be transferred

to the concrete flange. An increase of the bending resistance suggests that the minimum degree of shear connection requirement should also be increased, at least for a beam that is going to be highly utilised in bending. On the other hand, any additional slab depth means more load is applied at the construction stage (due to the additional weight of concrete), which can be beneficial in terms of the shear connection requirements for beams constructed unpropped (for which there is no demand on the shear connection as a result of concrete self-weight).

To demonstrate the above, the example of Figure 1 is considered. For the composite sections shown, finite element analyses were carried out and the results are presented in Table 1 in terms of slip at the reference value of $0.95M_{pl}$, where M_{pl} is the plastic resistance moment of the composite section. This is the level of loading that was used for the calibration of the minimum degree of shear connection rules in both BS EN 1994-1-1² and SCI P405.

The degree of shear connection provided in the example of Figure 1 is 0.33, compared to 0.61 required by BS EN 1994-1-1 and 0.25 (or 0.28 for propped) required by SCI P405 (the slip capacity is limited to 10 mm for this case with transverse trapezoidal decking). As shown in Table 1, although this is sufficient for both beams and the assumed slip capacity of the studs is not exceeded, the slip is higher for the case with the 250 mm deep slab when the beam is propped at construction stage (which suggests that the required $\eta = 0.28$ by P405 may not be, quite, sufficient in this case). However, when the beams are constructed unpropped, the slip is actually lower for the beam with the 250 mm slab than that for the 160 mm slab. As explained, this can be attributed to the higher proportion of load that is applied at the construction stage.

Table 1: Maximum values of slip at $0.95M_{pl}$ for the configurations of Figure 1

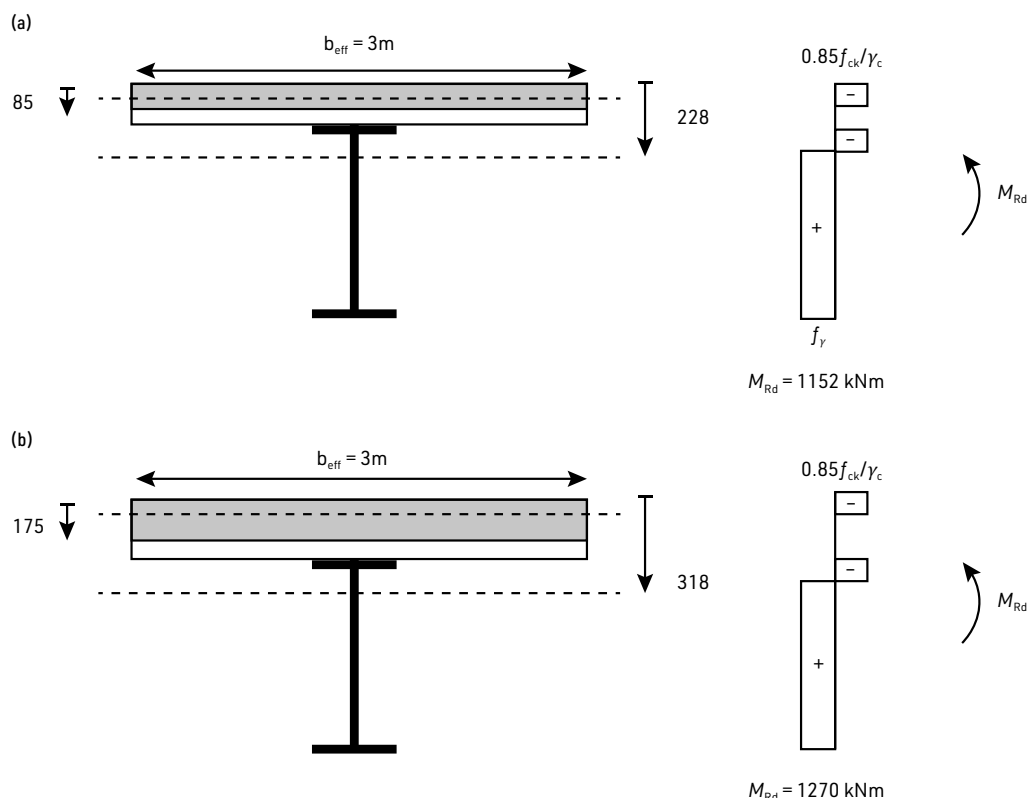
Slab depth	Propped	Unpropped
160 mm	8.5 mm	6.3 mm
250 mm	9.2 mm	5.0 mm

Plastic vs elastoplastic stress analysis of cross-section

The bending resistance of composite beams is normally determined from plastic analysis of the cross-section. This assumes that the effective areas of the steel section and the concrete flange can reach their design strengths before the concrete begins to crush. Depending on the cross-section considered, this assumption may become invalid. The current version of BS EN 1994-1-1 accounts for this effect in 6.2.1.2(2), through the introduction of a reduction factor β applied to the plastic resistance moment for steel grades greater than S355 (i.e. S420 and S460). This is because for such grades greater strain, and therefore more cross-section curvature, is needed to yield the steel. However, another aspect of the cross-section should also be considered; when the slab is deeper, it will experience greater compressive strain in the upper fibres of concrete for a given curvature. This will be reflected in the revision to BS EN 1994-1-1, with anticipated β values that also take into account the depth of the plastic neutral axis in the cross-section (β values will also be included for other steel grades, not just S420 and S460). In the meantime, care should be taken when assuming full plastic resistance for slabs that are atypically deep, although when the degree of shear connection is low the interface slip will reduce the compressive strains in the concrete.

Figure 1
Composite beam with
(a) 160 mm composite slab and
(b) 250 mm composite slab

Configuration details:
12m span beam
3m effective width
Fabricated steel section (IPE 500 equivalent dimensions)
S355 steel
C30/37 concrete
Transverse trapezoidal deck (60 mm deep with 15 mm stiffener at the top)
Degree of shear connection $\eta = 0.33$



Resistance and ductility of headed studs in composite slabs

BS EN 1994-1-1, 6.6.5.1 requires that the underside of a headed stud should extend at least 30 mm (clear distance) from the ‘bottom’ reinforcement. This statement clearly refers to slabs with two reinforcement layers, and is directly linked to the requirement for studs to resist slab-steel beam separation (i.e. studs also subjected to pull-out forces).

Common practice in UK construction is to provide a single layer of mesh reinforcement, which is often placed near the top of the slab at nominal cover to also control cracking and ensure an adequate performance in the event of fire. NCCI PN001a³ concluded that the detailing requirement of BS EN 1994-1-1, 6.6.5.1 does not need to be satisfied. This was based on the results of push-out tests, which showed that adequate resistance and ductility could also be achieved with the single layer mesh at nominal cover. The majority of these tests was on composite slabs with either 60 mm or 80 mm deep profiles, where the total slab depth did not exceed 160 mm, and the mesh was at the level of the stud head or slightly above (about 10 mm to 15 mm).

Therefore, the effect of having a deeper slab with a single layer mesh at nominal cover, i.e. at a greater distance above the head of the stud than investigated in the push-out tests, is not adequately known or understood. Smith and Couchman⁴ reported two series of push-out tests where the slab was 225 mm deep. Six tests were carried out (three per series). The specimens for the first series included a single stud per trough, while for the second series a pair of studs was used. The sheeting used was 60 mm deep, while the single layer of mesh reinforcement was placed at nominal cover, i.e. 25 mm from the top of the slab. Other test series were also included in the same test programme, where the slab depth was 140 mm. For the latter, the position of the mesh was either at 25 mm from the top of the slab or it was simply resting on the steel profile. The testing arrangement used, included the application of lateral load during the test. Comparison between the results for the deep and the shallow slab specimens did not reveal any influence of the slab depth on the stud shear resistance. There was actually a small increase of the resistance in the case of the deep slab specimens, which was significantly more pronounced for pairs of studs. However, these results were inconclusive in terms of the role of the slab depth.

Longitudinal shear resistance of beams

A composite beam must be verified for longitudinal shear, and adequate transverse reinforcement must be provided according to BS EN 1994-1-1, 6.6.6.4. The verification needs to be carried out considering different potential shear (failure) surfaces, as shown in Figure 6.16 of the Code, which is partly reproduced in Figure 2.

For the case shown on the right of Figure 2 where the sheeting is discontinuous over the beam (and the ends are not connected to the beam with through-deck welded studs), and also in the case where the sheeting is orientated parallel to the beam, the sheeting cannot be assumed to be effective against longitudinal shear. Therefore, mesh reinforcement should be placed below the head of the stud in order to be able to assume that it contributes to the longitudinal shear resistance of the beam for shear surfaces b-b and c-c of Figure 2, which is typically not the case in UK construction. For composite slabs of normal depth, where the mesh is positioned at around the same level as the head of the stud (or slightly above), failure as described by surface/plane b-b (or even c-c), would not be expected to occur and one could argue that the reinforcement would still be effective. For example, for the surface b-b shown in Figure 2, a different failure mode such as concrete splitting above the stud head (i.e. over the full depth of slab) would be expected to precede, in which case the mesh reinforcement would be able to resist. However, this may not be

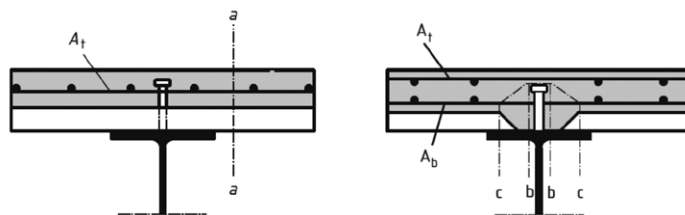


Figure 2 Shear surfaces considered in BS EN 1994-1-1 for the verification of longitudinal shear

true for deeper slabs where the distance between the mesh and the head of the stud is greater.

Conclusions / recommendations

Based on the above discussion, for slabs deeper than 180 mm, SCI advises the following:

- The minimum degree of shear connection requirement can still be based on SCI P405 for unpropped beams. For propped beams, further analysis is required, as the current rules in SCI P405 may become unconservative.
- It is advised that a second layer of mesh is placed locally over the beams near the top of the steel profile (or simply resting on it). This second layer of mesh needs to be properly anchored on both sides in the transverse direction, therefore it should extend a distance at least equal to the design anchorage length calculated according to BS EN 1992-1-1⁵.

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1. SCI P405: *Minimum degree of shear connection rules for UK construction to Eurocode 4*, 2015.
2. BS EN 1994-1-1:2004, *Eurocode 4: Design of composite steel and concrete structures. General rules and rules for buildings (together with the UK National Annex)*, BSI.
3. NCCI PN001a-GB: *Resistance of headed stud shear connectors in transverse sheeting*, SCI, 2010.
4. A.L. Smith, G.H. Couchman (2010) *Strength and ductility of headed stud shear connectors in profiled steel sheeting*, *Journal of Constructional Steel Research* 66, 748-754
5. BS EN 1992-1-1:2004+A1:2014, *Eurocode 2: Design of concrete structures - General rules and rules for buildings*, BSI.

AD 484:
Snow design situations

This advisory note aims to offer clarity when considering snow loading – in particular the drifted snow design situations. Snow loads are covered by BS EN 1991-1-3 and the associated National Annex. The most important advice is that NA.2.2 clarifies that the design situations to be considered in the UK are given in Case B2 of Table A.1 of BS EN 1991-1-3. Clause NA.2.3 specifies that Annex B of the core Eurocode should be used to determine the exceptional snow drift loads. If this advice is followed, it should stop designers even considering several design situations in the core Eurocode, as these are not relevant for structures constructed in the UK.

Case B2 of Table A.1 identifies two persistent design situations and one accidental design situation. The accidental design situation covers drifted snow, which is considered in valleys, behind parapets, behind obstructions and on lower roofs abutting taller structures. As noted above, Annex B is

used to determine the loading in these accidental design situations. As an accidental case, the design combination of actions should be calculated in accordance with expression 6.11b of BS EN 1990.

Perhaps somewhat confusingly, the second persistent case is also described as a drifted snow design situation - although it is treated as a persistent case and attracts the normal partial factors used in expressions 6.10, 6.10a and 6.10b of BS EN 1990. This second drifted case is the removal of all the snow from one roof slope, so it might be better described as an asymmetric case. In the core Eurocode, clause 5.3.3 and Figure 5.2 indicate that half the snow is removed from one roof slope, but this is amended by clauses NA.2.13, NA.2.18 and Figure NA.3 of the UK National Annex to specify that all the snow is removed from one roof slope.

Some designers appear to ignore this second persistent case. Designers may recall snow loads determined from BS 6399-3 which had exactly the same asymmetric load with no snow on one slope, but this situation only needed to be considered for roof slopes greater than 15°. There is no limiting roof slope in the Eurocode, so designers are reminded of this design situation.

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AD 486: NSSS Annex J amendment

The Sustainability Specification for structural steelwork, which is the new Annex J to the National Structural Steelwork Specification for Building Construction (NSSS), comes into force on 1st June 2022.

Before Annex J goes 'live', BCSA has amended the first paragraph of clause J.3.4.1 relating to fabrication waste management. The revised clause reads:

Where possible, the Steelwork Contractor should consider ordering sections "cut to length" to minimise off-cuts either by the mill, the Stockholder or the Steelwork Contractor. Alternatively, if stock lengths are ordered, these should be used efficiently by the Steelwork Contractor to minimise waste. Splice locations in steel members, where possible, should be coordinated to fall within standard stock length sizes.

The intention is to not preferentially source sections via a particular route rather to encourage suppliers and Steelwork Contractors to optimise section lengths and to minimise off-cuts, for example using section nesting software.

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AD 489: Height of shear studs

AD 380 entitled "What height of shear stud should be used in Eurocode 4?" provided interim advice on the height/length of shear studs to be used in design calculations and referred to a programme of tests/analysis that was in progress. On completion, the programme of tests provided the input to SCI publication

P405¹ but AD380 was not subsequently updated. This AD note now supersedes AD380 and reconfirms and supplements the guidance it contained.

BS EN 1994-1-1² defines h_{sc} as the 'overall nominal height' of a stud connector in the list of notation, but elsewhere the same variable is defined as simply 'the overall height'. Moreover, a stud that is for example 105 mm long when manufactured would typically have "length after welding" (LAW) of 100 mm when welded directly to a beam flange, or 95 mm when welded through decking. It would generally be described as a nominal 100 mm stud.

Resistances

Stud resistance values are a function of h_{sc} , because the solid slab resistance P_{rd} (clause 6.6.3.1(1)) may be reduced using the factors k_1 (clause 6.6.4.1(2)) and k_t (clause 6.6.4.2(1)) used to allow for the presence of decking. SCI's advice in P380 was that LAW should be used when determining reduction factors, not least because although the code itself is not clear, the ICE Designers' Guide to Eurocode 4 by Prof. Roger Johnson used the LAW in the examples.

In addition, BS EN 1994-1-1, clause 6.6.5.8(1) suggests that the minimum embedment length, i.e. length of stud extending above the top of the decking, should be $2d$ (where d is the stud diameter). Although considering the LAW when verifying this would be consistent, it would also suggest that a nominal 100 mm stud of 19 mm diameter could not be used with 60 mm decking. Experience and tests have shown that such a conclusion would be incorrect. It is therefore reconfirmed that, although apparently inconsistent, the nominal stud length may be used when establishing this lower bound length.

Results of test programme and analysis

As well as considering what values to use for the variables in code rules, the test programme referred to was undertaken to confirm what resistances and slip capacities, can be achieved when studs are used with modern forms of profiled decking. The tests also investigated the justification for the detailing requirement given in BS EN 1994-1-1 that any mesh local to the studs should be placed at least 30 mm below the head of the studs. Note that this criterion is not satisfied with the very common situation in the UK of a 60 mm trapezoidal deck used with nominal 100 mm studs.

The conclusions from this test programme are provided in reference 1, and guidance given therein confirms the recommendation that the as-welded height (LAW) is used in the BS EN 1994-1-1 reduction formulae.

More significantly, the results also show that when the decking is transverse to the beam the rules given in BS EN 1994-1-1 may be unconservative. To allow for this, additional reduction factors to those given in BS EN 1994-1-1 associated with the decking geometry are required as described:

- when double studs are present additional reduction factors are introduced as follows:
 - When mesh is placed below the heads of the studs an additional multiplication factor k_{mod} of 0.9 should be applied. The mesh does not need to be 30 mm below the head, merely underneath it.
 - When mesh is placed at nominal cover (assumed to mean above the head given typical slab geometries) the values derived from EN 1994 should be reduced using an additional multiplication factor k_{mod} of 0.7. For studs with transverse decking:

$$P_{rd} = k_{mod}k_tMIN[P_{rd}Solid\ slab]$$

- no further reduction is necessary when single studs are used, even if mesh is placed at nominal cover.

In addition, much research has been carried out in continental Europe as part of the background to new rules that will be presented in the so-called Generation 2 EN 1994-1-1. These suggest that some current UK practice may be unconservative, although this may simply be because of conservatism of the mechanical models that have been developed to predict resistance. The

Generation 2 document will also allow the use of test derived values.

One final point for designers to be aware of is that studs come in standard lengths (of which 100 and 125 mm are the most common). A designer may consider increasing the length of a stud to (potentially) increase resistance, but only standard lengths should be specified.

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1. Couchman G C, *Minimum degree of shear connection rules for UK construction to Eurocode 4*, SCI P405, 2015
2. BS EN 1994-1-1:2004 *Eurocode 4 Design of composite steel and concrete structures. General rules and rules for buildings (incorporating corrigendum April 2009)*

AD 491: Hydrogen embrittlement in structural bolting assemblies - effects and remedies

Introduction

Hydrogen embrittlement is a mode of failure that can affect high strength structural steel bolts. It is a reduction in the ductility of steel due to absorbed hydrogen making the steel less able to support the imposed stresses which can lead to the development of micro cracking and eventually failure.

This mode of failure is not well understood by engineers and specifiers, as all too often high strength steel bolts are specified without considering the implications of hydrogen embrittlement. This technical note briefly explains the three factors that need to be present to trigger this mode of failure and how this can be avoided.

What triggers hydrogen embrittlement failure?

For this mode of failure to occur the following three factors must be present:

- The steel must be a high strength steel, typically above 1000 N/mm², this includes property class 10.9 bolts and above, and
- There must be a tensile stress in the steel (due to the preload in a bolt or externally applied loads), and
- The steel must have absorbed atomic hydrogen. This is explained in more detail in the next paragraph.

A more detailed explanation of hydrogen embrittlement in structural fasteners is given in *Hydrogen Embrittlement - Its effect on Structural Bolting Assemblies*, which is available at: www.steelconstruction.info/Fabrication#Resources.

Sources of absorbed hydrogen

Absorbed atomic hydrogen can come from two sources:

- From the manufacturing process – i.e. Internal hydrogen embrittlement
- From the environment – i.e. Environmental hydrogen embrittlement

Studies have shown that hydrogen can be absorbed during manufacture, e.g. certain types of heat treatment and surface coatings. To avoid this, the manufacturing process must be carefully controlled. The necessary controls

and tests are given in the 'BCSA Model Specification for the Purchase of Structural Bolting Assemblies and Holding Down Bolts' (MPS), and high strength bolts should be specified in accordance with this specification. The MPS is also available at: www.steelconstruction.info/Fabrication#Resources.

Environmental hydrogen occurs when the steel is subject to corrosion from the environment. This can be avoided by designing the connections of a structure in such a way that they do not put high tensile strength bolting assemblies into areas where water or other electrolytes are allowed to collect and remain. Both coated and uncoated fasteners are susceptible to environmental hydrogen embrittlement.

Conclusion

By following the recommendations below, the risk of hydrogen embrittlement can be significantly reduced:

- Structural bolting assemblies should be obtained from approved suppliers certified to National Highways Sector Scheme 3 (NHSS3) and that bolts conform to the BCSA MPS. A list of Approved Suppliers can be found on the LANTRA Schedule of Suppliers website and a list of the BCSA suppliers of structural fasteners complying with NHSS3 and the MPS can be found in the 'Industry members' listing at the back of New Steel Construction or on the BCSA website www.steelconstruction.org and,
- Ensure that the design of the connections does not put high strength steel bolting assemblies into areas where water or other electrolytes are allowed to collect and remain. This applies to both coated and uncoated fasteners

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AD 493: Steel grade based on tested samples

The SCI has recently received a number of questions relating to the sampling of existing steelwork to determine the steel grade. In each case, no records of the steelwork exist, so samples have been obtained and tested. For understandable reasons, only a limited number of samples have been obtained, taken from different steel members within the structure. The testing has determined the yield strength and ultimate strength of each sample. The purpose of this Note is to remind engineers that the steel grade designation is related to the characteristic yield strength – which is significantly lower than the average yield strength.

If a large number of samples in one steel grade were obtained and measured, the results would be expected to follow a normal curve as shown in Figure 1.

The minimum yield strength (for example 355 N/mm²) for S355 steel grade is at the extreme left of the distribution. Note that there is always a very small probability that a result might be less than the specified minimum.

If samples of existing steelwork are taken, it should be expected from Figure 1 that generally the tested strengths should be higher (and sometimes much higher) than the specified minimum for any particular grade.

Engineers are directed to Appendix C of SCI publication P427, which gives the minimum and mean values for test results which may be used to identify the likely steel grade. As an example, if the anticipated steel grade is S355, then according to Table C.1, the minimum and average yield strength of the tested samples should exceed 391 N/mm² and 426 N/mm² respectively.

In a small population of tested results, it is not sufficient to determine the steel grade simply based on all samples exceeding the specified minimum strength for that steel grade. Unless the average of the test results is

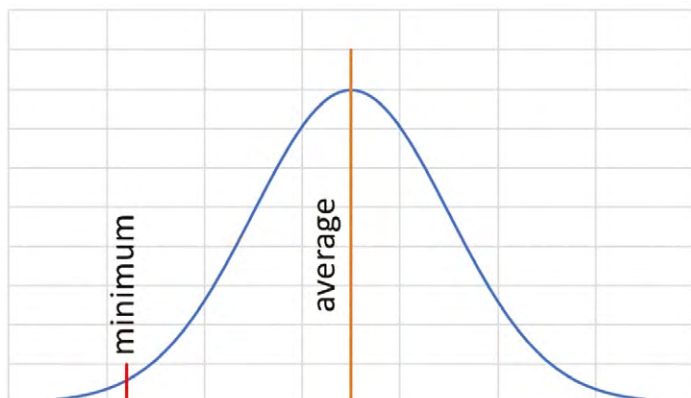


Figure 1: Typical normal distribution

significantly higher than the specified minimum, as demanded in P427, the form of the normal curve implies a high probability that other samples will be lower than the specified minimum.

Section C.3.3 of P427 covers a more comprehensive testing regime of members within the same group (same serial size, same function and detailing, as described in section 6.1 of P427). If this more extensive testing is undertaken, the calculated characteristic strengths should be compared to the minimum yield strength and ultimate strength tabulated in the appropriate material standards.

P427 is appropriate for steel used after 1970. The strength values tabulated in P427 are taken from Annex E of the proposed revisions to EN 1993-1-1

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AD 496: Toughness testing of bolts

Bolts used in environments subject to low temperatures that fall outside the range of temperatures usually encountered, such as in cold stores, may require toughness testing at the service temperature. SCI has recently been contacted regarding testing bolts that are smaller than the limiting diameter for normal Charpy impact tests. The purpose of this note is to advise how

impact testing can be carried out.

AD 332: *Toughness of bolts* advised that non-preloaded and preloaded bolts supplied to BS EN 15048:2007 and BS EN 14399-3:2005 would be tested at -20°C and have a Charpy V-notch impact strength of at least 27 joules.

Bolt materials comply with BS EN ISO 898-1:2013¹ which indicates in clause 1 Scope, Note 1 that “Fasteners conforming to the requirements of this part of ISO 898 are used in applications ranging from -50°C to + 150°C. ...”. Notwithstanding this, clause 9.14.1 indicates that impact tests are carried out only if required by a product standard or agreed between manufacturer and purchaser. The standard indicates tests are to be carried out in accordance with ISO 148-1 (Charpy V-notch test) at -20°C and are required to achieve a Charpy V-notch impact strength of at least 27 joules, for bolts of size M16 or greater (see Table 3). No tests are carried out to demonstrate impact strength at temperatures below -20°C unless specified but the standard indicates that other test temperatures and impact strength values can be called for.

The bolt size limitation appears to be indicated so that standard Charpy test pieces can be produced from the bolts. (M16 bolts have a tensile stress area of 157 mm² and a corresponding diameter of 14.1mm. The diagonal dimension of a 10mm square Charpy test piece is 14.1mm). Tests specified in ISO 898-1 are applicable to machined test pieces made from bolts, screws and studs of diameter at least 16mm. The total length of the test pieces is at least 55mm.

If impact tests are required on bolts of smaller size than M16, these can also be carried out in accordance with ISO 148-1, which allows for 7.5 mm, 5 mm and 2.5 mm square samples of 55 mm length. Such tests are not strictly in accordance with ISO 898-1 but will allow the impact properties of fasteners of smaller size than M16 to be determined at temperatures required by the purchaser. There is however no published basis for correlating test results from the smaller test pieces with results from standard ones so acceptance criteria should be agreed before supply. Test pieces with the standard length, depth and notch size but reduced width can also be used. The impact energy of such test pieces can be adjusted pro rata with the cross-sectional area at the notch for comparison with standard test requirements and would allow the impact properties of M12 bolts to be determined.

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1. BS EN ISO 898-1:2013 *Mechanical properties of fasteners made of carbon steel and alloy steel Part 1: Bolts, screws and studs with specified property classes – coarse thread and fine pitch thread* (ISO 898-1:2013), BSI



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