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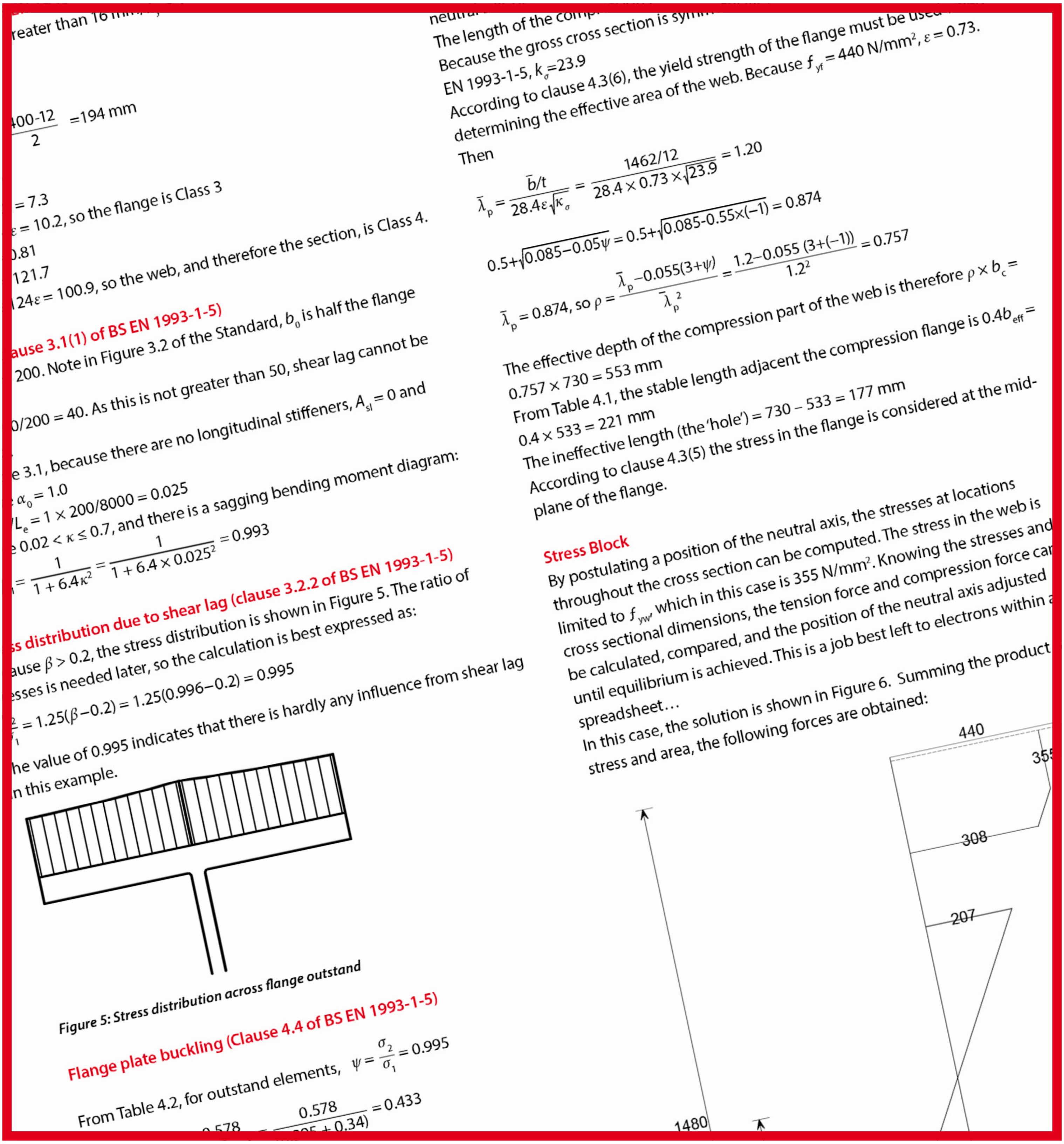


NSC

Technical Digest
2017

EDITOR
Nick Barrett Tel: 01323 422483
nick@newsteelconstruction.com

DEPUTY EDITOR
Martin Cooper Tel: 01892 538191
martin@newsteelconstruction.com

PRODUCTION EDITOR
Andrew Pilcher Tel: 01892 553147
admin@newsteelconstruction.com

PRODUCTION ASSISTANT
Alastair Lloyd Tel: 01892 553145
alastair@barrett-byrd.com

COMMERCIAL MANAGER
Fawad Minhas Tel: 01892 553149
fawad@newsteelconstruction.com

NSC IS PRODUCED BY BARRETT BYRD ASSOCIATES ON BEHALF OF THE BRITISH CONSTRUCTIONAL STEELWORK ASSOCIATION AND STEEL FOR LIFE IN ASSOCIATION WITH THE STEEL CONSTRUCTION INSTITUTE

The British Constructional Steelwork Association Ltd
4 Whitehall Court, Westminster, London SW1A 2ES
Telephone 020 7839 8566
Website *www.steelconstruction.org*
Email *postroom@steelconstruction.org*

Steel for Life Ltd
4 Whitehall Court, Westminster, London SW1A 2ES
Telephone 020 7839 8566
Website *www.steelforlife.org*
Email *steelforlife@steelconstruction.org*

The Steel Construction Institute
Silwood Park, Ascot, Berkshire SL5 7QN
Telephone 01344 636525 Fax 01344 636570
Website *www.steel-sci.com*
Email *reception@steel-sci.com*

CONTRACT PUBLISHER & ADVERTISING SALES
Barrett, Byrd Associates
7 Linden Close,
Tunbridge Wells, Kent TN4 8HH
Telephone 01892 524455
Website *www.barrett-byrd.com*

Keeping designers up-to-date

Nick Barrett - Editor

This is the second in the steel construction sector's annual series of Technical Digests. Steel construction is known for its tireless efforts in keeping engineers and architects fully up-to-date with the technical guidance that ensures they can take advantage of the numerous benefits of steel as a construction material. Among the many sources for this information is the steelconstruction.info website, the free to use first port of call for technical support. The monthly magazine New Steel Construction (NSC) is another popular source of advice, with Advisory Desk Notes and longer Technical Articles from the steel sector's own experts.

This Digest brings the Advisory Desk Notes and Technical Articles together in a separate format that is available as downloadable pdfs or for online viewing. It contains all of the AD Notes and Technical Articles from the steel construction sector published in NSC during 2017.

AD Notes reflect recent developments in technical standards or new knowledge that designers need to be made aware of. Some of them arise because a question is being frequently asked of the steel sector's technical advisers. They have always been recognised as essential reading for all involved in the design of constructional steelwork.

The longer Technical Articles offer more detailed insights into what designers need to know to do their jobs, often sparked by legislative changes or changes to codes and standards. Sometimes it is simply felt that it would be helpful if a lot of relatively minor changes, perhaps made over a period of time, were brought together in one place, so a technical update is needed.

The content of both AD Notes and Technical Articles needs to be known and understood by designers. Both can provide early warnings to designers that something has changed, and they need to know at least this much about it – further detailed information would always be available via the steel sector's other advisory routes. We hope you will continue to find this new publication of value.

Restraint to chords

BS 5950 indicates that purlins can be assumed to provide out-of-plane restraint to trusses. David Brown of the SCI discusses the intended scope of the advice and warns against straying outside the intended application.

Clause 4.10 of BS 5950 covers members in lattice frames and trusses. The clause contains a series of assumptions that designers may adopt, notably about buckling lengths, joint fixity and approximate bending moments in the rafters. The subject of this article is part (a) of that clause, which notes that the out-of-plane (buckling) lengths may be taken as the distance between purlins. It is tempting for designers to apply this guidance to all types of trusses, not appreciating that the original intent was relatively lightweight roof trusses.

In long span roofs, it is relatively common to provide a truss solution, perhaps with secondary trusses spanning onto primary trusses, so that internal column-free space is maximised. Some of the larger trusses carry significant loading and may therefore be fabricated with UC section chords (typically), or sometimes UB section chords, if other steelwork members connect to the chord. The eventual solution may be something like that shown in Figure 1. The chords are both UC members and the internal members are hollow sections. The exact details are immaterial – the key point is that there are purlins at the node points, and because of the proposed geometry and member selection, there are purlin connections at intermediate positions between the nodes.

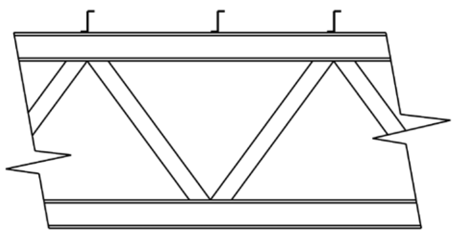


Figure 1: Assumed truss arrangement

Assuming that the top chord is in compression, the buckling resistance must be calculated, demanding an assessment of the buckling lengths in each axis. Designers may refer to clause 4.10 of BS 5950 and conclude from that clause that the out-of-plane buckling lengths may be taken as the spacing of the purlins. But is a connection to only one flange providing the assumed restraint, particularly at the intermediate location? Would the restraint be satisfactory for a UC section? Would it be equally satisfactory for a UB section, if one had been chosen?

The original intent of the clause

Colin Taylor, the primary drafter of BS 5950 has been consulted and his advice is acknowledged with gratitude. Colin comments that the clause was intended to be applied to small roof trusses (note the word "rafter" used in the clause) and similar triangulated lattices. The members themselves would have typically been angles, back-to-back angles or tees. At the time of drafting, purlins were angles, channels or even hollow sections. The use of light gauge purlins came later. Colin also notes that designers would have naturally provided restraint to the "inside" flange of compression chords.

It is interesting to look back even further, at the provisions in BS 449. Diagrams are provided giving the buckling lengths for stanchions, including those with tie beams attached to one flange only. Figure 14 from BS 449 is reproduced below as Figure 2, and the "diaphragms" shown providing restraint to the inside flange a clearly an important feature.

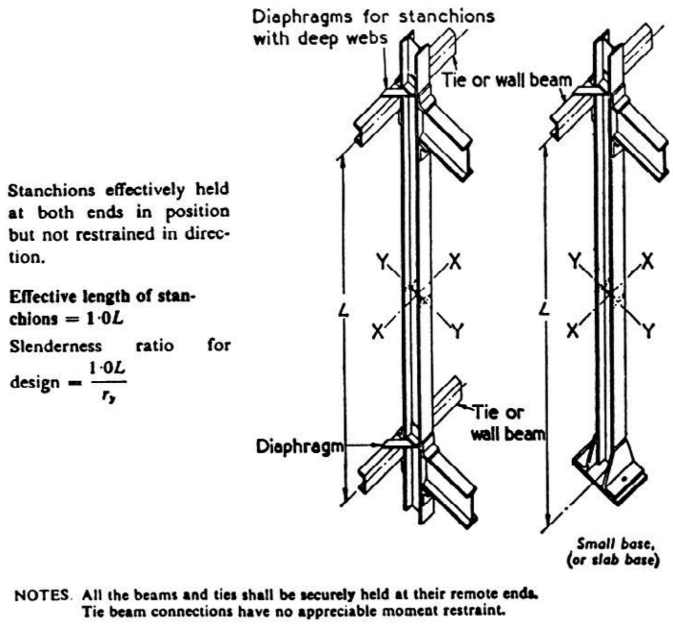


Figure 2: Figure 14 from BS 449 – Stanchion with tie beams attached to one flange

Figure 15 of BS 449 is equally instructive. In that Figure, a single storey stanchion has a number of intermediate angle side rails, attached to one flange only. The out-of-plane effective length is specified as 0.75L, where *L* is the overall height of the column, despite the intermediate angle rails.

Perhaps we might say that those provisions were unduly conservative, but it is clear that much more attention was paid to restraining both flanges, rather than assuming restraint to one side only was sufficient to produce pure flexural buckling in the minor axis. This article aims to encourage designers to think carefully about such arrangements and consider how the member will buckle.

But how *does* the member behave?

Jumping forward from BS 449 to today, designers have a range of tools which can be used to investigate structural behaviour. Colin Taylor mentions making Perspex models, but today's solution is invariably software.

For the second part of this article, the software *LTBeamN* has been used, as this tool allows restraints to be placed anywhere within (or outside) the member depth and allows the fixity (both laterally and rotationally) to be specified. To investigate the behaviour in a truss, a member has been modelled with fork ends at the nodes. At the intermediate purlin position, a lateral restraint can be modelled. Specifying full lateral and rotational fixity in the software will produce the results for a fully effective lateral and torsional restraint – the chord buckling will be minor axis flexural bending between the purlin positions. The "real" situation can also be modelled, with a lateral restraint some distance outside the flange (assumed to be the centre of the bolt group to the purlin) and a varying degree of rotational fixity. The software reports the elastic critical buckling load, $N_{cr,}$ but also gives a useful graphical output of the buckling mode.

Buckling examples

The following examples are based on a 254 UC 89, arbitrarily chosen as a typical section. The nodes are at 4 m centres, and a single restraint is provided at the mid-point.

With no intermediate restraint, the member (as expected) buckles in the minor axis, between the supports. The buckled form is shown in Figure 3

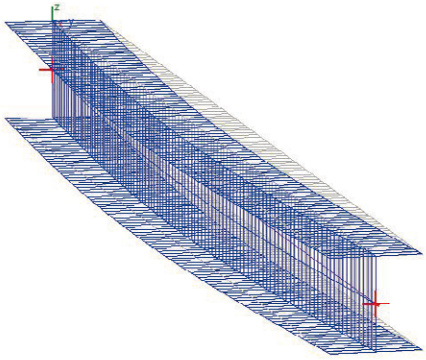


Figure 3: Buckling between supports

For the arrangement in Figure 3, N_{cr} is given as 6264 kN. For those interested, the intermediate steps and the buckling resistance in S355 are as follows: $\lambda = 0.789$; $\chi = 0.669$; $N_{b,z,Rd} = 2610$ kN

If a midspan restraint is introduced with full torsional fixity, the result is shown in Figure 4.

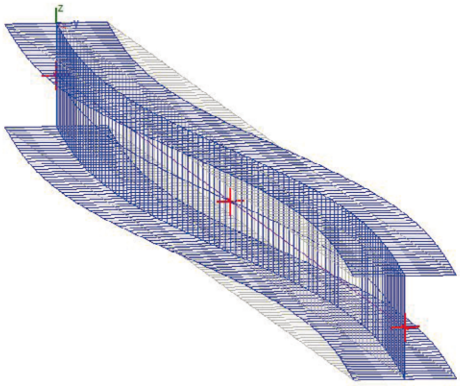


Figure 4: Buckling with lateral torsional support at midspan

For the arrangement in Figure 4, N_{cr} is given as 25069 kN. The intermediate steps and the buckling resistance in S355 are as follows: $\lambda = 0.394$; $\chi = 0.9$; $N_{b,z,Rd} = 3510$ kN

The values of 2610 kN and 3510kN can be confirmed in the Blue Book.

If a midspan restraint is provided 100 mm outside one flange only, with no torsional fixity, the result is shown in Figure 5

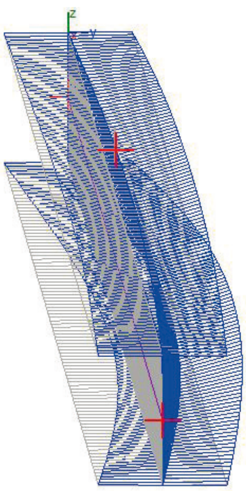


Figure 5: Buckling with lateral support at midspan, 100 mm outside the flange

In this case, both flanges have buckled laterally, not the double curvature bending shown in Figure 4 that one might have hoped for. In the case illustrated in Figure 5, N_{cr} is given as 7273 kN. The intermediate steps and the buckling resistance in S355 are as follows: $\lambda = 0.732$; $\chi = 0.705$; $N_{b,z,Rd} = 2747$ kN, which is significantly less than the resistance with an effective lateral torsional restraint.

The benefit of stiffness at the connection

The buckling form in Figure 5 resulted from a lateral restraint which was modelled to provide zero rotational stiffness. It could be argued that there is some rotational stiffness delivered by the secondary member. If this case is to be made, designers must credit the connection itself with stiffness and the ability to transfer moment, as this provides the torsional fixity to the main member. Without doing any analysis, it seems rather brave to credit a connection to a light gauge steel member with too much stiffness, as the bolts are in oversize holes and the material is thin.

With a secondary member each side of the chord, with lengths L_1, L_2 and Inertias I_1 and I_2, the stiffness can be calculated as

$$4E \frac{I_1}{L_1} + \frac{I_2}{L_2}$$

With a typical purlin length taken as 7m and a typical purlin inertia of 175 cm^4, the stiffness at the joint is calculated as 420 kNm/radian. Assuming the joint itself is infinitely stiff (which must be too optimistic, as discussed above) the midspan restraint can be credited with some rotational stiffness.

Figure 6 shows the results for the identical situation described in Figure 5, but with rotational stiffness at the restraint of 420 kNm/radian.

In fact, even with some degree of stiffness, the buckling form has not changed significantly. In this case, N_{cr} is given as 12079 kN. The intermediate steps and the buckling resistance in S355 are as follows: $\lambda = 0.568$; $\chi = 0.804$; $N_{b,z,Rd} = 3135$ kN

It may be observed that the resistance (3135 kN) appears to be approaching that when a fully effective lateral torsional restraint is provided (3510 kN). However, the rotational stiffness must be increased from 420 kNm/radian to 1660 kNm/radian before double curvature bending results. In other words, the secondary members must be around four times as stiff as is typical, before the assumption of a lateral torsional restraint is realised – and that still depends on the unlikely assumption that the connection itself is infinitely stiff.

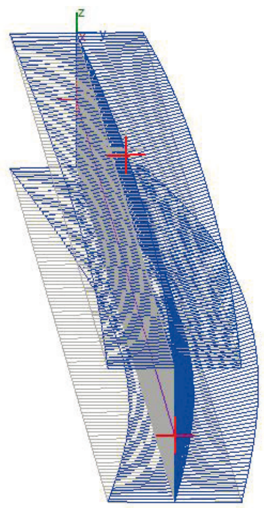


Figure 6: Buckling with lateral and rotationally stiff support at midspan, 100 mm outside the flange

Conclusion

It is hoped that this article has illustrated that restraints to only one flange of compression members should not be assumed to provide effective torsional restraint, unless carefully assessed. The advice in clause 4.10 of BS 5950 should not be used to justify such an assumption for large, heavily loaded members, as it is clear that the intended scope was limited to quite different forms of construction. If there is uncertainty about the effectiveness of the restraint, freely available software may be used to examine the behaviour of the member, modelling the location and fixity of the connecting steelwork.

M_{cr} Calculation software

David Brown and Ibrahim Fahdah of the SCI introduce the new design tool on *www.steelconstruction.info*

Most designers working in accordance with the Eurocodes will have met the elastic critical buckling moment, M_{cr}. This moment is needed as an essential step on the way to calculate the lateral torsional buckling (LTB) resistance of a member. The non-dimensional slenderness $\bar{\lambda}_{LT}$, which is needed to calculate the LTB resistance is given by:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

M_{cr} can be determined by calculation, using the following expression:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}}$$

This expression is straightforward to use, but is limited to uniform members with "fork end" supports and loads assumed to be applied at the shear centre. Other more involved expressions are available that deal with loads not applied at the shear centre (stabilising or destabilising loads) and different end conditions. The only real uncertainty is the value of C_1 which depends on the shape of the bending moment diagram. For linear and non-linear bending moment diagrams, expressions are available to calculate C_1[1].

Many real design situations become complex. An irregular loading pattern may make the bending moment diagram quite unorthodox. Some loads may be applied at the shear centre, whilst others on the same member may be applied elsewhere within (or outside) the section depth. Restraints may be intermittent, to one flange, or the other, or outside the section depth, at different locations along the member. The numerical expressions for M_{cr} are unable to deal adequately with these types of situations and software must be used.

Previous articles in New Steel Construction have discussed the use of software[2,3] and a design tool to calculate M_{cr} has been available on *steelconstruction.info* for a considerable period. The design tool on *steelconstruction.info* has been replaced with a new software, substantially increasing the scope, features and output; this article offers advice on how the software should be used.

Interface

Users will note that the interface looks markedly different from other design tools available on the *steelconstruction.info* website. The new software has a wider scope, with many opportunities for the user to define sections, beams, loading and restraints, which demands a flexible and comprehensive interface (Figure 1). In general, users work from one tab to the next, saving data at each step. The saving of data is important, as it triggers a refresh of any graphic, a re-ordering of any tabulated data and makes subsequent tabs available.

Although the software has been developed to assess beams, it may also be used to analyse steel members subjected to bending and axial loading.

Input Data

The first step when using the software is to create a project, which is a collection of one or more beams. Material types and cross sections are defined for the project, which are then used when beams are added to the project. Because the software allows a number of different beam arrangements (sections, length, loading, restraints etc) to be defined and

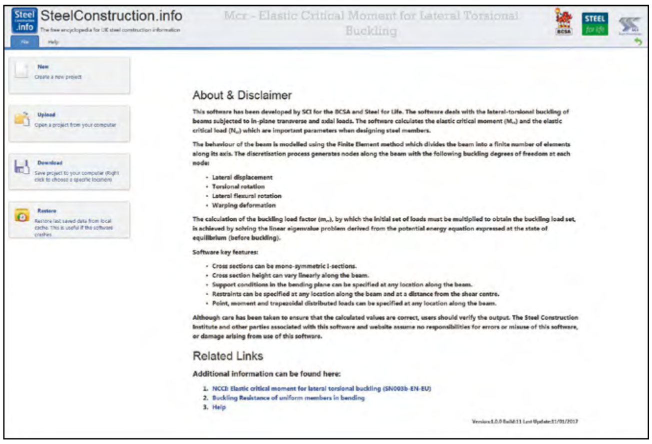


Figure 1: M_{cr} software interface

analysed at one time, perhaps grouped because they are all part of one single project, a number of sections might be selected at this stage.

The "Section Definitions" tab, shown in Figure 2, allow users to add modify and delete cross sections. The software allows users to add standard sections (UB and UC) or non-standard sections defined by their dimensions or their properties.

Figure 2: Sections Definitions – initial view

Users then visit the "Beams" tab (Figure 3) to work with the beams in the project. Beams may be added, duplicated or deleted. The currently selected beam is indicated in the drop down box at the top left corner of the tab. By default the remaining input tabs for a beam are greyed out. These tabs will be enabled as the user visits them working from the left to the right.

Figure 3: Beams input tabs

Figure 4 shows the "Profile" input. This screen shows the length of beam (the default is 10 m) and invites the user to define the cross-section at each end of the beam. Only the cross-sections previously defined are available. The data should be saved, which will update the graphics. In general, the cross-section will be identical at each end of the beam. The software allows a beam to be made up a number of different sections, using the "add" option. This may be used if part of the beam is tapered and part has a uniform cross-section. Tapered beams must be defined by dimensions, so the cross-section at each end of the section must be defined and selected on this "Profile" input. Only the depth of the beam can be varied on a tapered beam. The use of cross-

[1] Calculating the C_1 factor for lateral torsional buckling. NSC Nov/Dev 2013
[2] Getting the best out of LTBeam, NSC, May 2009
[3] Use of LTbeamN, NSC, January 2015

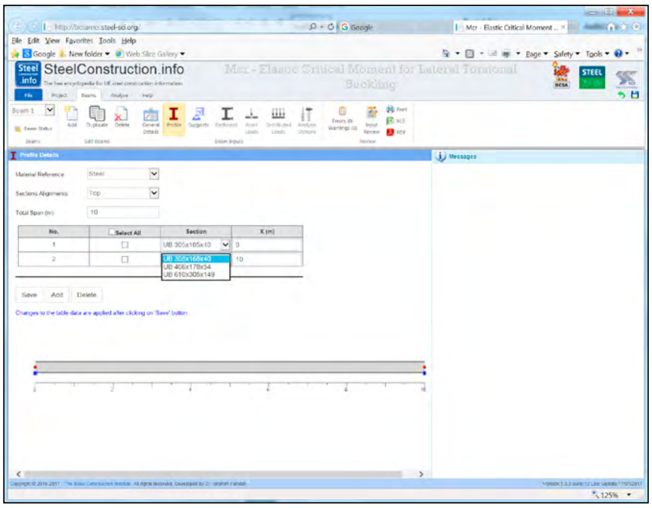


Figure 4: Profile input

sections defined by dimensions and a beam comprised of two sections allows the user to investigate beams which are tapered over part of their length, as shown in Figure 5.

Supports may now be selected, with default locations at each end of the beam. By default, both supports (shown in blue) are pinned and fixed vertically, one end is fixed laterally. These defaults can be changed by the user, and intermediate supports added if required.

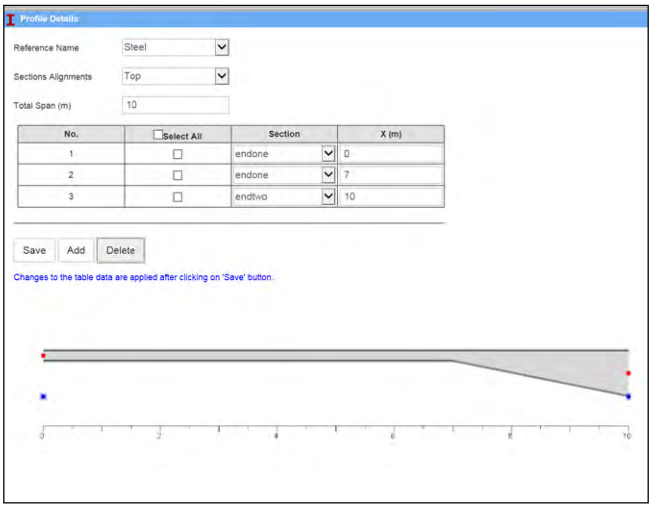


Figure 5: Part tapered beam

Restraints may now be added. The default restraint condition is of "fork ends" at each end of the beam, which is laterally fixed, warping free and the restraints are located at the shear centre. All of these defaults may be changed and additional restraints may be specified. Normally, restraints might be discreet 'point' restraints, but the user can select a length of continuous restraint if required. Restraints may be specified at any point with respect to the top of the section, bottom or shear centre. Figure 6 shows a beam with intermediate restraints (red dots), one above the top flange, and one on the top flange. Note that saving the data is always required to refresh the graphics.

Having amended (or at least viewed) and saved the restraint data, loading data is now available. The default inputs are empty, so loads are introduced by the "add" button. Point loads, point moments, axial loads, distributed loads and varying loads may all be added, with the position of load application anywhere within (or outside) the section depth.

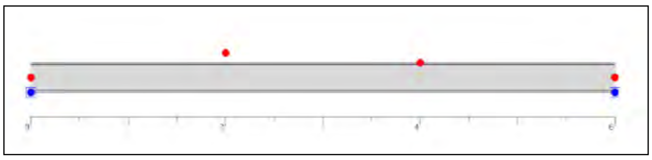


Figure 6: Illustration of restraint options

This allows users to deal readily with destabilising (above the shear centre) and stabilising (below the shear centre) loads.

Figure 7 shows some of the possibilities available to the designer when defining loads.

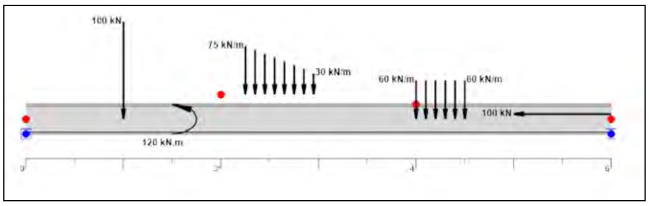


Figure 7: Load arrangements

The last view on the beam inputs group is "Analysis Options" view. The analysis options of a beam allow the user to vary the number of the finite elements used to represent the beam – more elements may improve the accuracy of the result marginally. The default of 100 elements is recommended.

The options to ignore bending effects and ignore axial effects demand some explanation. When verifying beams using expressions 6.61 and 6.62 of BS EN 1993-1-1, the lateral torsional buckling resistance is determined assuming there is no axial load, and the axial resistance is determined assuming there is no bending; the interaction is accounted for within the expressions. The opportunity to ignore one type of load allows the user to readily determine the values of M_{cr} and N_{cr} in the absence of axial force and bending respectively. The values of M_{cr} and N_{cr} will be different if calculated with both types of load applied simultaneously.

One warning

It may be very helpful to use this software to determine N_{cr} and thus the buckling resistance of a beam under axial load – particularly if the restraints are to one flange only, or perhaps to both flanges but at different locations along the beam. This version of the software only reports the value of N_{cr} in the minor axis direction. It would be possible to restrain the beam in the minor axis so effectively that major axis flexural buckling becomes the critical behaviour – which is not investigated by the software.

Finally, the analysis and output!

The "Analyse" tab allows the user to move to the analysis stage. Users can carry out the analysis and review the analysis results for the currently selected beam, or all beams in the project. The default report is a simple summary of M_{cr} and N_{cr} values, but much more detail can obtained by selecting the "Beam Report" (Figure 8). The reports can be exported to pdf, or Excel, or printed. The reported "buckling factor" is the amplifier by which the actual loads must be multiplied to obtain the elastic critical buckling moment. As a trivial example, a beam of 6 m span and 20 kN/m UDL results in a midspan bending moment of 90 kNm. If the value of M_{cr} is 109.9 kNm, the buckling factor is 109.9/90 = 1.221. A buckling factor less than 1 indicates an immediate problem – a larger beam is required.

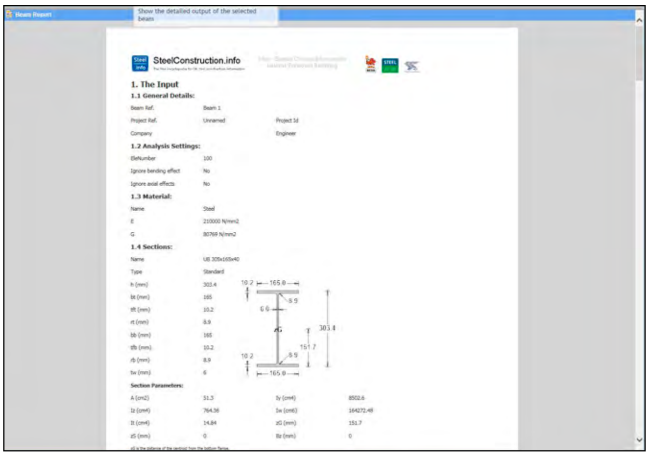


Figure 8: Detailed beam report

Steel construction with trusses

Richard Henderson of the SCI discusses the use of trusses in buildings.

1 Introduction

Trusses have been used in construction for centuries, originally manufactured from timber and used to form pitched roofs. Early truss railway bridges in the United States were constructed of timber and iron rods. With the development of wrought iron, truss bridges in this material were built in large numbers from the 1870s. The Forth Bridge was the first major steel bridge adopting truss construction and opened in 1890.

Steel trusses in buildings are used extensively to cover large clear spans and this article will mainly focus on this sort of construction.

2 Roof trusses

Roof trusses are an efficient means of supporting a roof covering for spans upwards of 20 m. Upper bound spans of 100 m are suggested on the steelconstruction.info web site. The upper limit is in fact dictated by the value and utility to the building user of the clear span and enclosed volume because examples of truss bridge construction illustrate that much longer spans are possible.

Space trusses and diagrids have been used to form two-way spanning roofs but the most common arrangement of truss roof construction uses one-way spanning elements. A common form of truss is the Pratt truss (or N frame) with vertical shear elements in compression and diagonal shear elements in tension.

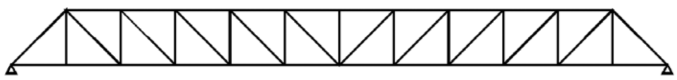


Figure 2.1: Pratt truss

Another is the Warren truss with all shear elements inclined at the same angle to the horizontal in alternating tension and compression from the support to mid-span of a simply supported span.

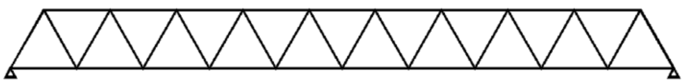


Figure 2.2: Warren truss

Primary trusses are commonly spaced at about one quarter or one fifth of their span but consideration should be given to the form of the secondary elements and roof decking when choosing the truss spacing as it is usual to have no more than two "layers" of structure supporting profiled roof sheeting. Deep-profile decking is capable of spanning five metres or more depending on the loading and can therefore be used with secondary elements spanning 20 metres or more between long-spanning primary trusses.

The span to depth ratio of trusses ranges from 10 to 25, depending on the intensity of the applied load. The lower the ratio, the longer are the shear members in the truss and the larger is the volume occupied by the roof structure. However, the load in the truss booms is lower in a deep truss so there is a trade-off between the truss booms and the members carrying shear forces. The slope of the top boom must also be considered because for a long span truss the increase in depth from eaves to mid span can be significant. The slope must also allow rainwater run-off to occur without ponding.

3 Truss modelling and analysis

The first step in modelling trusses for analysis, when designing to EN 1993-1-1 is to classify the joints in accordance with clause 5.1.2. If the joints are classified as fully pinned or fully fixed, the stiffness of the joints does not need to be taken into account in the global analysis. If the joints have an intermediate stiffness, the moment-rotation curve of the joint does affect the results. It is usual to choose fully pinned or fully fixed joints as the moment-rotation characteristics of the joints are not normally known. A common arrangement is for the tension and compression booms to be modelled as continuous with the bracing members pin ended because this matches the usual built arrangement.

Hand analysis of statically determinate trusses can easily be made if all the joints are assumed to be pinned and computer modelling can follow the same approach. The axial forces found in the members will be slightly higher following this approach than if all the joints are assumed fixed. Appropriate releases must be included in the analysis model, e.g. a roller support at one end. Where a truss boom is connected to a column which is included in the model but not intended to provide lateral stability, the connection should be released to ensure the column does not develop unintended bending moments.

Initial selection of members can be made from a hand estimate of the maximum bending moment divided by the mid-span depth and shear force at the support. The starting point can be improved by more detailed hand analysis or the choice of truss members can be refined by iterative computer analysis. If a vertical deflection criterion is to be met, it is worth noting that, unlike in solid-webbed beams, the deformation of the bracing (shear) members contributes significantly to the total deflection. If the truss is to have bolted joints, the adoption of non-slip joints will eliminate the significant additional deflections due to bolt slip. Such joints are particularly recommended for splices.

4 Choice of truss members and connections

4.1 Tubular members

Tubular members with fully-welded joints are often used for visible roof trusses because they give the cleanest appearance. In conventional steel building design and manufacture, it is usual for the structural engineer responsible for the overall design to select the members and for the steel fabricator to design and detail the connections. In the case of trusses made from steel tubes, it is important for the structural engineer to consider the design of the connections when selecting the members. It may be tempting to select a large size thin-walled element for a compression boom because of its efficient buckling performance. However it is likely that joints between such a member and shear members in the truss will require external strengthening to prevent failure of the thin wall.

A cheaper, easier to fabricate choice of member would be a smaller size, thicker walled section with joints that required no strengthening. Connection design rules and details are given in BS EN 1993-1-8.

Splices are necessary in long-span trusses for transportation: a 22 m length does not require any special arrangements for movement by road. Pipe-flange type joints are often used in truss booms and are efficient in compression. In tension, thick end plates may be required.

Spade-type joints with cover plates can be connected to tubes by slotting them. Although introducing boom splices at mid span of a truss may not initially appear sensible, for a uniform load, the reduction in forces at third points for a parallel boom truss is only 11% so the difference in the splice arrangement is not likely to be large.

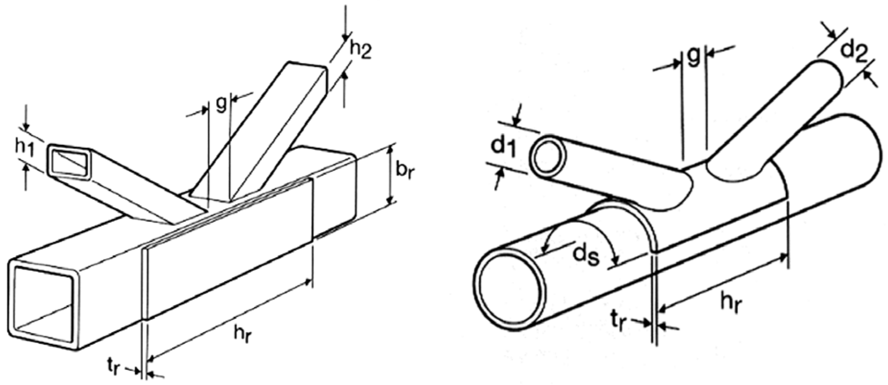


Figure 4.1: Tubes with external strengthening

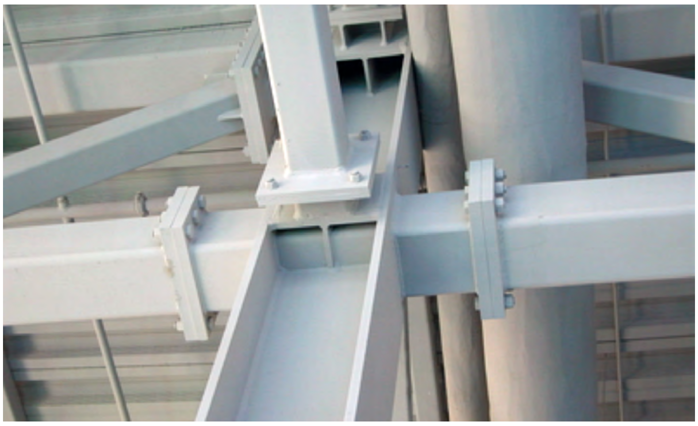


Figure 4.2: Thick end plate splices

4.2 Open sections

Open section members are utilitarian and give more scope for bolted forms of connection. Booms can be oriented with webs vertical or horizontal with different benefits for each arrangement. Vertical webs with gusset plates welded on centreline result in a planar element through which forces can flow from member to member which may not require any strengthening.

Vertical flanges provide a surface to which tension diagonals (flats or angles) can be welded in pairs with single compression members between. Top and bottom booms must be the same size however.

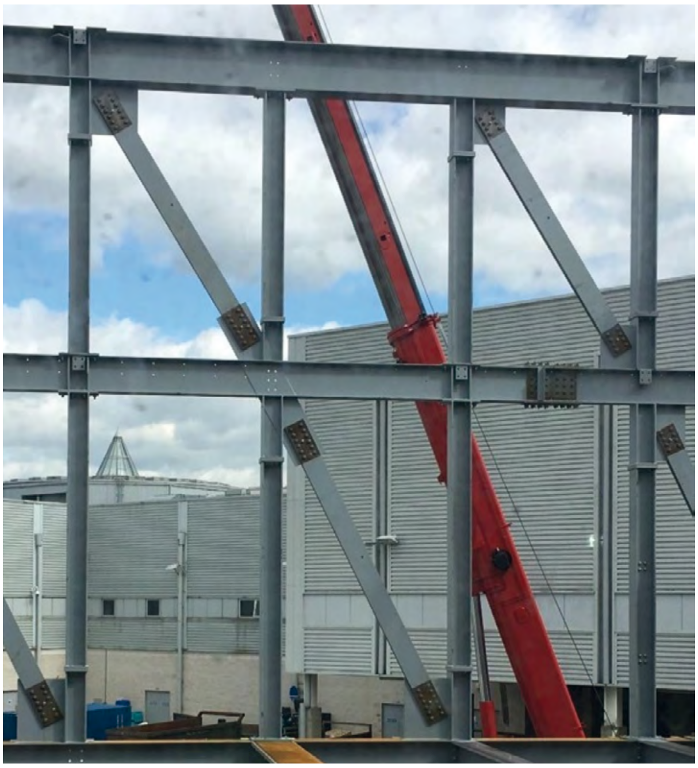


Figure 4.3: Thick end plate splices. *Photo courtsey of H Young Structures Ltd*

Figure 4.4: Vertical flanges

Open sections in compression can be orientated so that minor-axis buckling in the plane of the truss is restrained by secondary members provided for that purpose. The efficient use of material in the strut is traded off against the extra members and joints.

Gusset plate details are included in the SCI 'Green Book'.

5 Compression boom restraints

A system of restraints to the compression boom of trusses is essential to their structural performance in a roof. Such restraints are usually provided by a system of in-plane bracing connected to purlins or specially provided restraint members. As discussed in the article on restraint to chords in (NSC, January 2017), careful consideration to the effectiveness of the connections between the truss booms and restraining members must be made. Clause 5.3.3 of BS EN 1993-1-1 gives guidance on the design of bracing systems used for restraint of truss compression flanges and indicates that such restraint forces are internal forces and are not transmitted to the building foundations.

Long-span light-weight roofs may be subject to wind uplift such that the bottom boom of the truss goes into compression. If this occurs, the bottom boom must also be adequately restrained to prevent buckling.

Figure 5.1: Top and bottom boom restraints

6 Conclusion

Trusses are a common and effective way of supporting long-span roofs in buildings. The variations in possible arrangement are very wide and the results range in appearance from delightful to utilitarian.

Structural steel reuse

Michael Sansom of the SCI focuses on structural steel reuse and in particular, the work of SCI to explore the viability of more mainstream steel reuse.

Structural steel sections are inherently reusable. Reuse, as opposed to the current, common practice of recycling steel by remelting, makes good environmental sense; saving both resources and carbon emissions. It also retains more economic activity within the UK since currently around 70% of steel scrap is exported for recycling.

SCI, working together with the University of Cambridge, has recently completed two national, (Innovate UK) projects exploring the barriers to more mainstream reuse, the economics of reuse and assessing the feasibility of developing a website for trading and sharing information about reclaimed structural steel. SCI is also working on two large European projects REDUCE and PROGRESS. This article describes these projects.

Reusing reclaimed steel is not a new idea; in fact, the practice was more prevalent in the past but has declined over the last few decades. The main reasons being new development programme constraints and tougher health and safety requirements in relation to demolition activities, in particular, working at height.

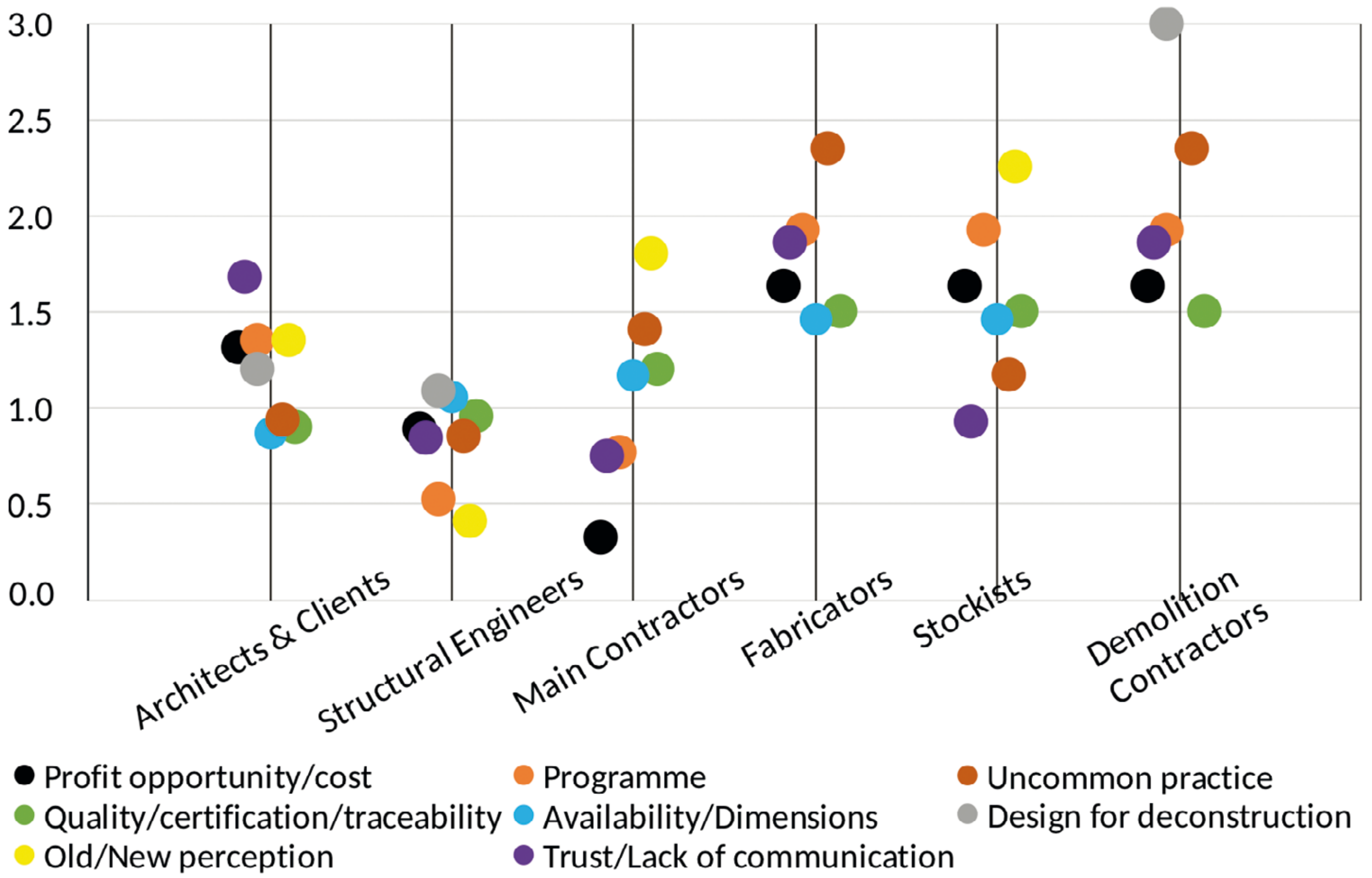
There are many steel-based temporary work systems which are highly reused; the challenge is to develop permanent work systems that are similarly reusable.

Reuse is commercially and technically viable, as demonstrated by isolated projects and in certain niche markets. Reusing simple structures, such as portal frames, is relatively common particularly in the agricultural and industrial building sectors. The SEGRO case study is a good recent example.

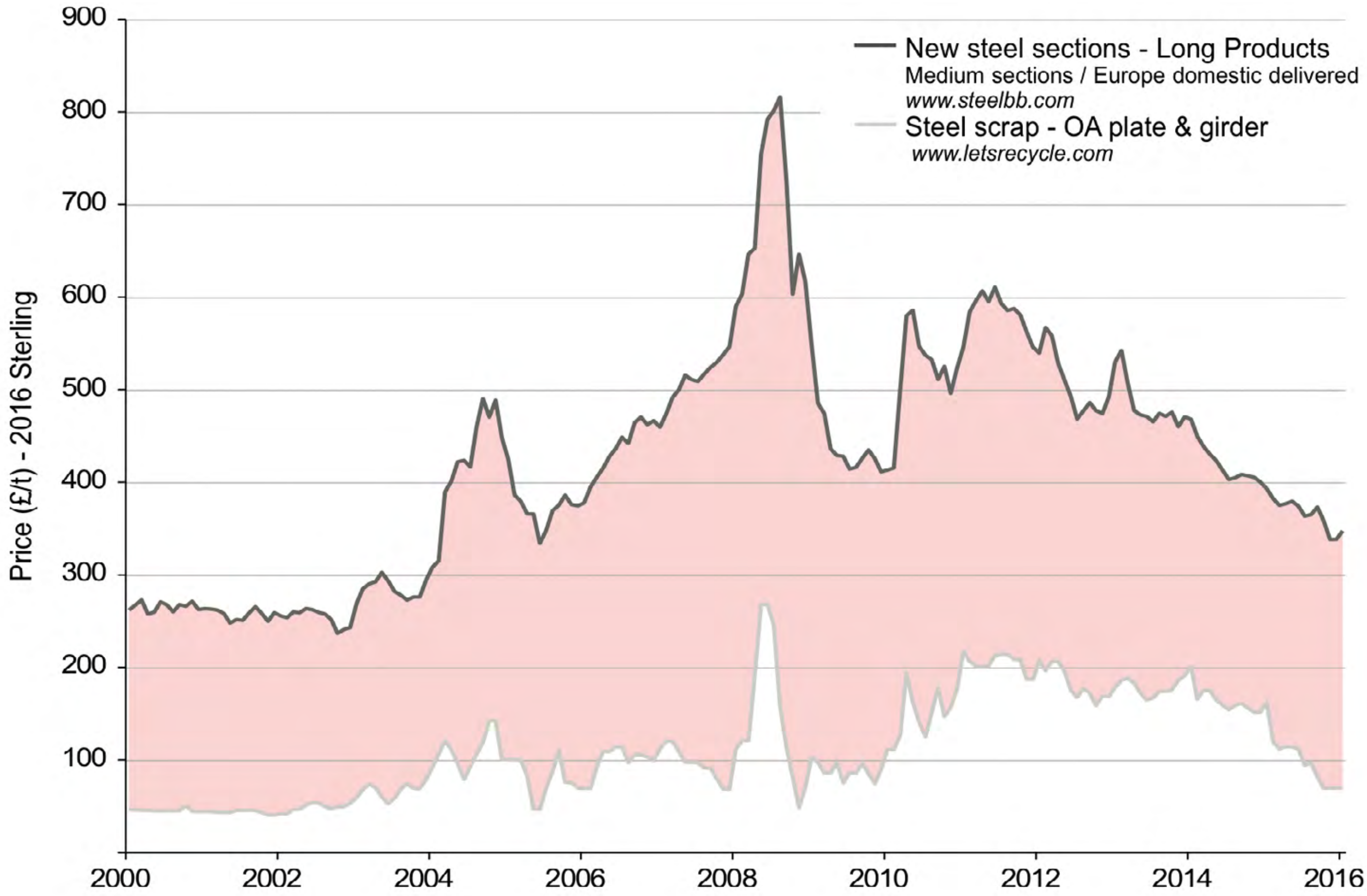
Consultations with the steel construction supply chain, confirm the technical viability but also identify the many, real and perceived, barriers to reuse. These include barriers across the supply chain, notably the additional cost and longer procurement and construction programmes involved.

Based on consultations (interviews and on-line survey) with the supply chain, the overall ranking of barriers in descending order of importance, is:

1. Availability of reclaimed sections; particularly of the desired size, volume and in the right location
2. Issues relating to the quality, traceability and certification of reclaimed sections
3. Additional cost associated with using reclaimed sections
4. (Lack of) supply chain integration; particularly communication and sharing information through the supply chain and trust (and risk sharing) between companies
5. Additional time required within construction programmes to allow for using reclaimed steel; in general, additional time incurs addition cost
6. Reclaiming and reusing structural steel is a relatively uncommon practice and many organisations simply do not have the skills or experience to do it
7. The perception that reclaimed steel is somehow inferior to new steel sections.



Barrier ranking by actor – the higher the score the higher the perceived importance[1]



Price differential between new steel sections and scrap grade OA (2000-16) [2]

The historical (2000-16) price range between new steel sections and scrap sections (grade OA scrap) reveals an average price difference of £313 per tonne over this timeframe. This differential represents the profit opportunity for reuse before the additional deconstruction costs (testing and certification, storage and refabrication costs) are taken into account.

The economic case for widespread reuse today is marginal. Under current UK economic and legislative conditions, the conclusion is that, other than some small-scale and niche markets and under certain project specific circumstances, mainstream structural steel reuse is not viable today. The lack of economic incentive is compounded by the lack of any legislative drivers. We conclude that, in the short-term, this situation in the UK is unlikely to change dramatically.

Although more mainstream structural steel reuse is unlikely under current UK economic and legislative conditions, BIM technologies overcome several of the barriers to steel reuse by providing certainty about material properties, traceability and provenance and eliminating the need for testing. Looking ahead therefore, structural steel (BIM) models offer a cost-effective means of enabling future reuse.

Steelwork contractors have been using BIM models for years and routinely offer their clients as-built structural models on building handover. By storing such models in a secure database, this will future-proof UK steel structures by enabling:

- efficient refurbishment and structural extension of existing structures
- safe deconstruction
- a detailed inventory of reclaimed steel sections for future use (with full traceability and all relevant material properties)
- optimising the recycling process through knowledge of the metallurgy of the steel.

SCI has developed a prototype website and database to facilitate trading of reclaimed steel sections and for securely storing structural steel information to enable future deconstruction and reuse.

Design for deconstruction

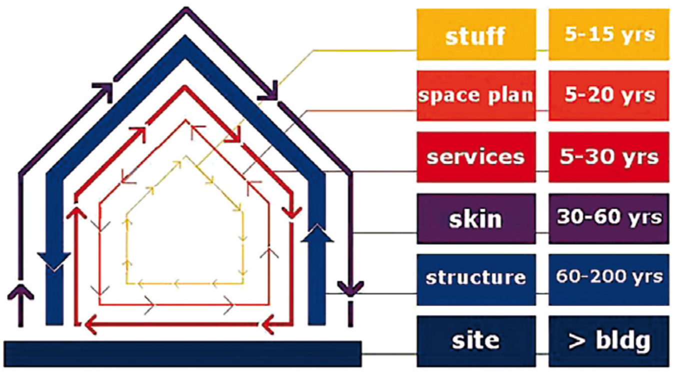
Design for deconstruction (and reuse) is central to the circular economy. Current practice, is generally to demolish buildings with little thought about preserving the integrity and value of components for reuse. Only by designing buildings for deconstruction can we make reuse of buildings and building components more commonplace and commercially viable.

The ability to reuse building components is, to a large extent, dependent on how buildings have been constructed in the first place. Although designers routinely consider the constructability of buildings, generally little thought is given to their deconstruction and how elements and components could be reclaimed and reused. At its simplest level, there are two main considerations:

1. The types of materials and components used; some products, like structural steel, are inherently more reusable than other structural materials and systems
2. The way the materials and components are put together (thus able to be taken apart) and deconstructed.

Key principles to follow in design for deconstruction are:

- Simplicity – design building systems and interfaces that are simple to understand, with a limited number of different material types and component sizes
- Standardisation and regularity – design building systems and materials that are similar throughout the building and laid out in regular, repeating patterns. Where possible, standardise elements
- Simplify and separate building systems – use a 'layering approach' to keep elements of the building (with different anticipated lifetimes) separate
- Minimise the number of different types of materials and components; fewer larger elements which are more durable and easier and quicker to remove are more likely to be reused
- Use lightweight materials and components
- Use reusable materials: chose materials that are inherently durable and reusable and retain their value through reuse
- Identify points of disassembly/connections and ensure they remain accessible
- Simplify and standardise connection details: This allows for efficient construction and deconstruction and facilitates reuse without modification after deconstruction
- Use mechanical fasteners in preference to chemicals such as sealants and adhesives
- Record as-built conditions, i.e. what was built not just what was designed
- Provide a deconstruction plan outlining general concepts where the load path for the self-weight of structure and deconstruction loads follow conventional paths. Provide specific detailed plans where load paths are not conventional. All load transfer systems should be identified
- Record adaptations to the building over its life
- Ensure information is securely stored and remains accessible.



Layering approach to design for deconstruction in which different layers (with different lifetimes) are separated to facilitate deconstruction and minimise waste © SEDA

The Circular Building, London 2016

Specifically in relation to structural steel:

- Provide clear documentation of all steel members used in the structure including, size, grade, length, and connection details
- Keep records of the steel supplied, specifically mill test certificates including manufacturer, production date and standard
- Ideally steel members should have a permanent marking or tagging to assist in traceability and to identify their chemical and mechanical properties.

SCI is coordinating a collaborative EU-funded project called REDUCE (Reuse and Demountability using Steel Structures and the Circular Economy).

The overall objective of this three-year project is to provide practical tools and steel-based technologies to be able to design steel and composite structures for deconstruction and reuse. A specific objective is to develop and test composite, steel-based flooring systems which are demountable and the components reusable.

The project will investigate applications of the developed systems in commercial and residential buildings and will explore options for greater standardisation. In the context of demountable composite construction systems, the shear connector systems with the greatest potential will be tested and analysed using numerical modelling so that design guidance can be developed following the principles of Eurocode 4.

In addition, REDUCE will review methodologies to quantify the benefits of demountable buildings and reuse including life cycle assessment methodologies, e.g. CEN TC350 standards and developing metrics for quantifying circularity, e.g. those developed by the Ellen MacArthur Foundation. The project will also explore how BIM can be used to provide information to enable the building to be easily adapted during use, and/or deconstructed and the components reused at end-of-life.

SCI is also a partner in a new EU project (PROGRESS). The focus of PROGRESS is the deconstruction and reuse of elements of single-storey buildings. The project will address both the structural and envelope elements and their interfaces, and will also consider both the reuse of existing buildings and how new single-storey buildings can be designed and constructed to facilitate future reuse.

SEGRO warehouse and office building deconstructed and relocated on the Slough Trading estate in 2015

Structural steel is uniquely placed to deliver buildings that are flexible, adaptable and ultimately reusable. Although current UK commercial and legislative conditions are not conducive to more widespread reuse, the realisation that current, global consumption patterns are unsustainable leads to the conclusion that it is only a matter of time before reuse of buildings is mandatory. SCI is leading the UK steel construction sector to ensure that it is able to capitalise on these opportunities.

References

1 Cyrille F. Dunant, Michał P. Drewniok, Michael Sansom, Simon Corbey, Julian M. Allwood, Jonathan M. Cullen (2017) Real and perceived barriers to steel reuse across the UK construction value chain, *Resources, Conservation & Recycling*, under review.

2 Cyrille F. Dunant, Michał P. Drewniok, Michael Sansom, Simon Corbey, Jonathan M. Cullen, Julian M. Allwood (2017) Cost and Risk Distribution Across the Value Chain: Why Steel Reuse Is Not Profitable, *Construction & Building Materials*, under review.

Building Consequence Classes and the link to Execution Classes

Richard Henderson of the SCI discusses the evolution of Building Consequence Classes and the link between them and Execution Classes of structures.

Figure1: Ronan Point Flats

The Building Regulations

Building Consequence Classes have their origin in Approved Document A to the Building Regulations and the provisions for disproportionate collapse in section A3. The need for such provisions was exposed by the partial collapse of the 23-storey Ronan Point flats in 1968 following an accidental gas explosion in a kitchen on the 18th floor. (see Figure 1).

Section A3 of Approved Document A (AD-A) gives the requirement in the Building Regulations itself and its limitations and guidance as to how the requirements may be met. In the 1985 Building Regulations, requirement A3 stated "The building shall be so constructed that in the event of an accident, the structure will not be damaged to an extent disproportionate to the cause of the damage". The requirement was applicable only to: (a) a building having five or more storeys (each basement level being counted as one storey); and (b) a public building, the structure of which incorporates a span exceeding nine metres between supports. The guidance in AD-A advised the Building Regulations could be met for steel buildings by designing to BS 5950: Part 1: 1985 where recommendations for the provision of horizontal ties for all buildings were made. Additional recommendations were made for "tall" buildings for vertical ties and, where necessary, localization of damage. The latter recommendation resulted in the provision of key elements where notional removal of an element resulted in damage over an unacceptably large area.

The 1991 Building Regulations amended the wording but not the sense of the requirement. The limits on application removed the reference to a public building including a span of over nine metres. Guidance given in BS 5950 was that horizontal and vertical ties were to be provided but where effective vertical ties were not feasible, a check on the potential area of collapse was to be made by considering notional removal of the element and limited to a maximum of 70 m^2 or 15% of the floor area, whichever is the smaller. If this check failed, the element was to be designed as a "protected member" or key element.

The Building Regulations 2000 included the same requirement A3 as the 1991 Regulations: "The building shall be constructed so that in the event of an accident, the building will not suffer collapse to an extent disproportionate to the cause". No limits on application were made. Instead, guidance depending on the class of the building was provided in AD-A on appropriate approaches to ensure sufficient robustness of a building to limit the extent of damage or failure without collapse. The building classes were presented in Table 11 in AD-A. The primary function of the building classification was to establish what structural arrangements were to be made by the structural engineer for a particular building to ensure satisfactory robustness.

The Building Regulations 2010 (the current edition) includes exactly the same requirement. In the guidance in AD-A, building classes are renamed building Consequence Classes (CC) and are shown in Figure 2 (Table 11 from AD-A). The guidance in AD-A states the requirement will be met if horizontal ties are provided for CC2a; vertical ties are added for CC2b and a systematic risk assessment is carried out for CC3 buildings. The alternative notional removal of columns and provision of a key element where the area at risk of collapse is too extensive is also included. The maximum area is increased from 70 m^2 to 100 m^2.

Table 11 Building consequence classes

Consequence Classes	Building type and occupancy
1	Houses not exceeding 4 storeys
	Agricultural buildings
	Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height
2a Lower Risk Group	5 storey single occupancy houses
	Hotels not exceeding 4 storeys
	Flats, apartments and other residential buildings not exceeding 4 storeys
	Offices not exceeding 4 storeys
	Industrial buildings not exceeding 3 storeys
	Retailing premises not exceeding 3 storeys of less than 2000m² floor area in each storey
	Single-storey educational buildings
	All buildings not exceeding 2 storeys to which members of the public are admitted and which contain floor areas not exceeding 2000m² at each storey
2b Upper Risk Group	Hotels, blocks of flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys
	Educational buildings greater than 1 storey but not exceeding 15 storeys
	Retailing premises greater than 3 storeys but not exceeding 15 storeys
	Hospitals not exceeding 3 storeys
	Offices greater than 4 storeys but not exceeding 15 storeys
	All buildings to which members of the public are admitted which contain floor areas exceeding 2000m² but less than 5000m² at each storey
	Car parking not exceeding 6 storeys
3	All buildings defined above as Consequence Class 2a and 2b that exceed the limits on area and/or number of storeys
	Grandstands accommodating more than 5000 spectators
	Buildings containing hazardous substances and/or processes

Notes:

1. For buildings intended for more than one type of use the Consequence Class should be that pertaining to the most onerous type.
2. In determining the number of storeys in a building, basement storeys may be excluded provided such basement storeys fulfil the robustness requirements of Consequence Class 2b buildings.
3. BS EN 1991-1-7:2006 with its UK National Annex also provides guidance that is comparable to Table 11.

Figure 2: Approved document A Table 11

The number of storeys and floor area are the principal determinants of the building Consequence Class. The potential number of people harmed in the event of a collapse is also an important factor.

The National Structural Steelwork Specification and EN 1090-2

In 1989, the first edition of the National Structural Steelwork Specification for Building Construction (NSSS) was published for the execution of steelwork by people with full knowledge and understanding of BS 5950. It coincided with the introduction of the British Constructional Steelwork Quality Assurance Certification Scheme (now SCCS), based on the requirements of BS 5750 Parts 1 and 2, precursor to ISO 9001. The NSSS was conceived to achieve greater uniformity in contract specifications issued with tender and contract documents. The first edition covered outline requirements for the design, materials, preparation of drawings, fabrication, erection and protective treatment of structural steelwork which is to be used for all types of building construction. The (current) Fifth Edition CE Marking Version was published in October 2010.

The clauses of the latest version fully complies with BS EN 1090-2: Technical requirements for steel structures, introduced in 2008. This standard introduces the concept of Execution Class (EXC), a significant addition to the original requirements of the NSSS. It is a classified set of requirements specified for the execution of the works as a whole, of an individual component or part thereof. The stated reason to differentiate between the classes is to provide a level of reliability against failure or malfunction of the structure that is matched to the consequences of failure.

Execution Classes

The default Execution Class to which BS EN 1090-2 is applicable is EXC2 (cl. 4.1.2). The NSSS scope states the specification is based on the execution of structures in EXC2 and it is not intended for structures which are subject to fatigue or seismic loading. The requirements of the NSSS are in general more onerous than those of EXC2 in BS EN 1090-2. Additional requirements of EXC3 over EXC2 are principally aimed at reducing defects in the material, producing a higher quality of welding and increasing the level of inspection of the work. The National Foreword to BS EN 1090-2 suggests that as a default basis, EXC2 could be specified for structures/components/details used in buildings, and EXC3 could be specified for structures/components/details used in bridges where fatigue needs to be considered in design. The additional requirements in EXC3 may therefore be considered to be at least part of those necessary for structures subject to fatigue.

BS EN 1993-1-1:2005+A1:2014 was issued in June 2015. Amendment 1 involved the inclusion of Annex C which places the basis for the selection of Execution Class in the design standard. The UK National Annex issued at the same time sets out the UK approach to selecting the Execution Class. Structures in Consequence Classes one and two are to be minimum EXC2. Structures in Consequence Class three are to be EXC3. The NSSS is therefore applicable un-amended to structures in CC1 and CC2 but structures in CC3 require additions to the NSSS, as outlined in BS EN 1090-2.

Effect of the Consequence Class as a differentiator

The use of the Consequence Class to differentiate between Execution Classes is a blunt instrument. As in all circumstances where there is a step change in requirements on either side of a boundary between regions of a continuum, instances which just fall on the more onerous side of the boundary may be considered to be unreasonably penalized. This issue is illustrated in the following two figures which show hypothetical building arrangements. Both buildings are of conventional simple construction with a regular grid, with braced frames providing stability. Neither building is subject to fatigue.

Figure 3 is a stick diagram of the structure of a 15-storey office building with 9 m by 9 m bays. The floors are 8 bays by 7 bays giving a floor area of 4,536 m^2 and a total area over 15 floors of about 68,000 m^2. At an occupancy rate of one person per 10 m^2 of net floor area, the number of occupants could well exceed 5000 people. This structure does not exceed 15 storeys or 5000 m^2 per storey and therefore meets the requirements for Consequence Class 2b. The execution class for the structure is therefore EXC2.

Figure 4 is a stick diagram of the structure of a 2-storey shopping centre, also with 9 m by 9 m bays. The floor is 8 bays square giving a floor area of 5,184 m^2. The floor area exceeds 5,000 m^2 and therefore falls outside the requirements for Consequence Class 2b and the structure is Consequence Class 3. The Execution Class for the structure is therefore EXC3.

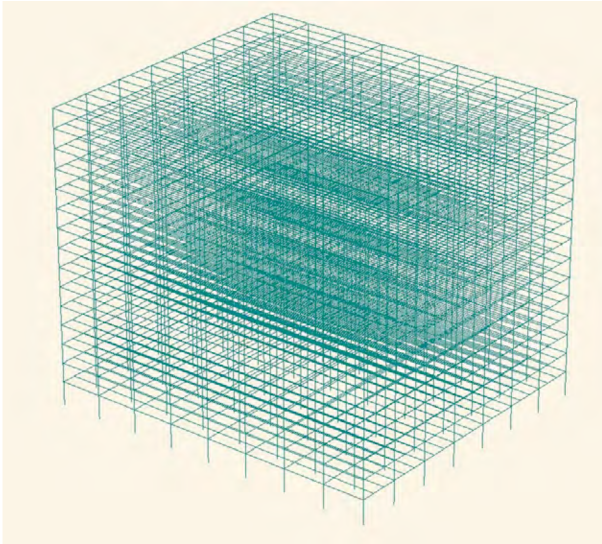


Figure 3: 15 Storey office building

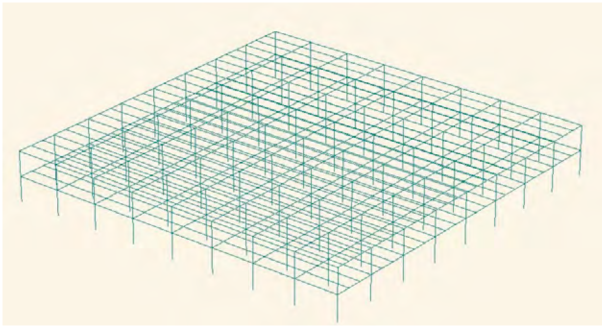


Figure 4: Two storey shopping centre

The office building requires horizontal and vertical ties. The shopping centre requires a systematic risk assessment to determine the provisions for robustness. Such an assessment may well conclude that no special provisions for robustness are required and in practice, horizontal ties alone will be sufficient. As the columns are all likely to be continuous over the two storeys, no consideration of vertical tying is necessary. Nevertheless, the office building is in EXC2 and the shopping centre is in EXC3.

In the case of the shopping centre with a regular grid and conventional structure, what benefit is there of requiring that the Execution Class is 3? Does either the employer or the user gain from greater reliability? Compared with the office building, the potential number of occupants is likely to be less so fewer people are likely to be affected by any loss of a column.

Approved Document A does offer an alternative approach to using Table 11 to determine the Consequence Class of a building, set out in documents referred to. These documents give the background to the building classes in Table 11. The approach is specific to a building and may result in its allocation to a lower Consequence Class than Table 11.

Conclusion

Execution Classes can be applied to a whole structure, a part or specific detail. If a transfer structure was included in a CC2b building, it is likely to be designed as a key element and EXC3 can be specified for this part at the discretion of the structural engineer. It seems equally appropriate in the case of a regular, conventional structure which falls outside the requirements of CC2b, such as the shopping centre illustration above, that the structural engineer be given the ability to choose Execution Class 2. Perhaps a more appropriate approach would be to place building structures not subject to fatigue in EXC2 and require significant parts such as transfer structures to be in EXC3.

The design of hybrid fabricated girders

David Brown of the SCI discusses the design of hybrid fabricated girders. In the first part of the article, some background is presented, and a worked example taken as far as the moment resistance. Shear resistance is covered in Part 2.

Why hybrid?

Hybrid girders are plate girders with flanges of higher strength than the web. Conceptually, one might say that the web merely keeps the flanges apart, so why not use a lower steel strength for the web? The web must carry the shear force, but this is generally low in a beam designed for bending or deflection, so high strength webs are not required. The low demand for shear resistance coupled with the desire to keep the flanges far apart means that webs in fabricated plate girders are often deep and thin – making a stiffened web likely and triggering a visit to BS EN 1993-1-5 to determine properties for Class 4 sections.

Shear Lag

Shear lag may affect both compression and tension flanges. Ordinarily, it is assumed that the stress distribution across a flange is uniform, as shown in Figure 1. In fact, the longitudinal stresses are transmitted through the web-to-flange junction. It may readily be imagined that the flange local to the web is compressed more than the flange tips, as indicated in Figure 2. The tips of the flanges "lag" in that they do not take the assumed evenly distributed share of stress. The phenomena is managed in BS EN 1993-1-5 by calculating a reduced effective width of the flange.

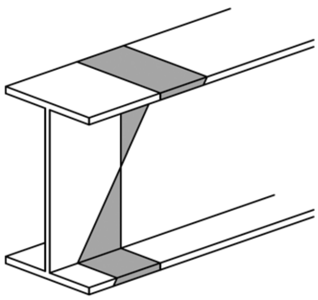


Figure 1: Commonly assumed stress profile

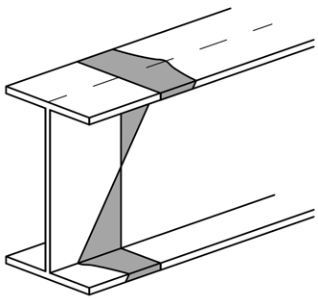


Figure 2: Shear lag in flanges

Plate buckling - flanges

All elements in compression share an enthusiasm to buckle – so the tips of relatively thin, wide flanges wish to buckle locally and do not carry load as assumed. Plate buckling only applies to the compression flange and is managed in BS EN 1993-1-5 by reducing the effective area of the flange.

Plate buckling – web

The compression zone of a thin web will suffer from local buckling. This is managed by a "hole in the web" approach where the ineffective portion of the web is neglected. The Standard specifies the stable lengths of web attached to the flange and attached to the tension zone of the web.

Stress distribution

Combining a lower strength web with higher strength flanges and assuming an ineffective portion of the compression zone of the web, the resulting stress distribution may look something like that shown in Figure 3. It is not possible to determine a modulus directly, so the position of the neutral axis is found by equating the tension to the compression. The hole in the web adds complication to the process, because the stable parts are a proportion of the

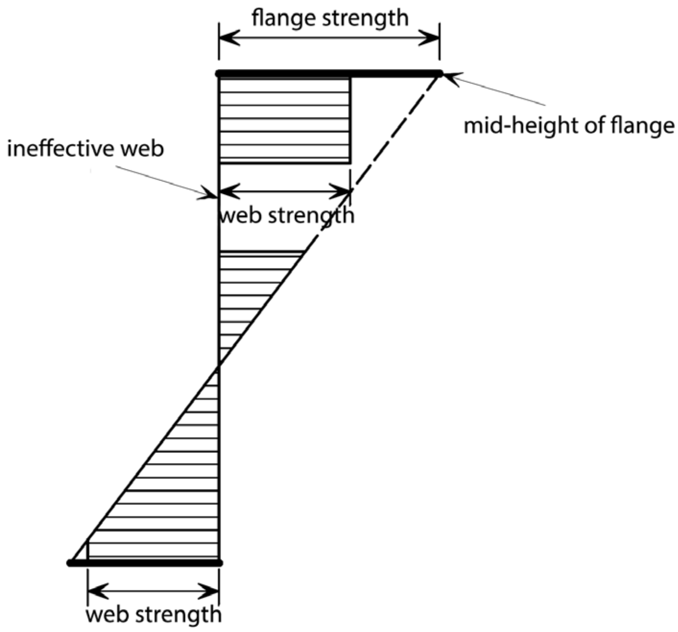


Figure 3: Typical stress profile for a hybrid girder

compression zone – and therefore change as the neutral axis moves. Others designers may have a clever way to determine when equilibrium of force is reached – the SCI approach is to move the neutral axis a (small) step at a time, check the resulting forces, and repeat as necessary until the solution is found. This is a task for a spreadsheet, or VBA.

Once the stress distribution has been determined, the moment resistance of the cross section $M_{cy,Rd}$ may be calculated, being the product of stress, area and lever arm.

Worked example

The cross section to be verified is shown in Figure 4. The flanges are S460 and the web S355. The beam span is 8 m.

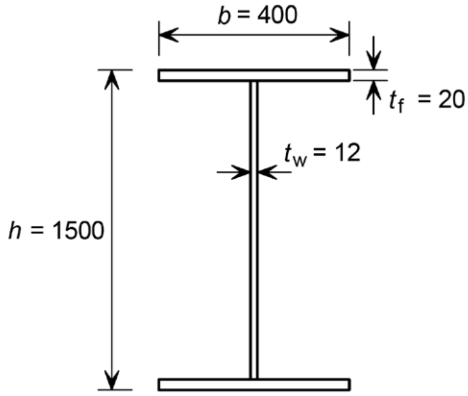


Figure 4: Cross section dimensions

Material strengths (BS EN 1993-1-1) and classification

Because the flange is greater than 16 mm, $f_y = 440$ N/mm²

$$\varepsilon = \sqrt{\frac{235}{440}} = 0.73$$

Flange outstand = $\dfrac{400\text{-}12}{2}$ =194 mm

$c/t = 194/20 = 9.7$
Class 2 limit is $10\varepsilon = 7.3$
Class 3 limit is $14\varepsilon = 10.2$, so the flange is Class 3
for the web, $\varepsilon = 0.81$
$c/t = 1460/12 = 121.7$
Class 3 limit is $124\varepsilon = 100.9$, so the web, and therefore the section, is Class 4.

Shear Lag (clause 3.1(1) of BS EN 1993-1-5)

$b_0 = 400/2 = 200$. Note in Figure 3.2 of the Standard, b_0 is half the flange width.
$L_e/b_0 = 8000/200 = 40$. As this is not greater than 50, shear lag cannot be neglected.
From Table 3.1, because there are no longitudinal stiffeners, $A_{sl} = 0$ and therefore $\alpha_0 = 1.0$
$\kappa = \alpha_0 b_0/L_e = 1 \times 200/8000 = 0.025$
because $0.02 < \kappa \leq 0.7$, and there is a sagging bending moment diagram:

$$\beta = \beta_1 = \frac{1}{1 + 6.4\kappa^2} = \frac{1}{1 + 6.4 \times 0.025^2} = 0.993$$

Stress distribution due to shear lag (clause 3.2.2 of BS EN 1993-1-5)

Because $\beta > 0.2$, the stress distribution is shown in Figure 5. The ratio of stresses is needed later, so the calculation is best expressed as:

$$\frac{\sigma_2}{\sigma_1} = 1.25(\beta - 0.2) = 1.25(0.996 - 0.2) = 0.995$$

The value of 0.995 indicates that there is hardly any influence from shear lag in this example.

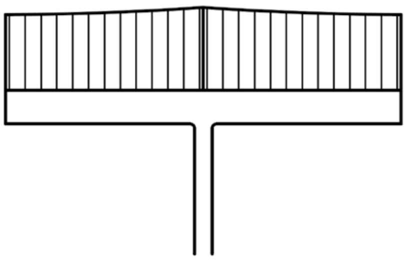


Figure 5: Stress distribution across flange outstand

Flange plate buckling (Clause 4.4 of BS EN 1993-1-5)

From Table 4.2, for outstand elements, $\psi = \dfrac{\sigma_2}{\sigma_1} = 0.995$

therefore $k_\sigma = \dfrac{0.578}{\psi + 0.34} = \dfrac{0.578}{(0.995 + 0.34)} = 0.433$

then $\bar{\lambda}_p$ is given by clause 4.4(2) as:

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{194/20}{28.4 \times 0.73 \times \sqrt{0.433}} = 0.711$$

note that $\bar{b} = c$ for outstand flanges. $c = (400 - 12)/2 = 194$ mm
because $\bar{\lambda}_p < 0.748$, ρ = 1.0
effective^p area $A_{c,eff} = 1.0$ Ac = 400 × 20 = 8000 mm²
The superscript ^{p} indicates this is the effective area when considering plate buckling.

Combined effects of shear lag and buckling (clause 3.3(1), Note 3, of BS EN 1993-1-5)

The effective area of the compression flange considering both shear lag and plate buckling is given by:
$A_{eff} = A_{c,eff}\beta^{\kappa} = 8000 \times 0.996^{0.025} = 7999$ mm²
There is therefore no reduction due to the effects of shear lag and plate buckling.

Web buckling

Because (in this case) there is no reduction of the compression flange due to the combined effects of shear lag and plate buckling, and no reduction of the tension flange due to shear lag, the gross cross section is symmetrical. The neutral axis of the gross section is at mid-height of the web.
The length of the compression part of the web b_c is 1460/2 = 730 mm
Because the gross cross section is symmetrical, $\psi = -1$, and from Table 4.1 of BS EN 1993-1-5, k_σ=23.9
According to clause 4.3(6), the yield strength of the flange must be used when determining the effective area of the web. Because $f_{yf} = 440$ N/mm², $\varepsilon = 0.73$. Then

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{1462/12}{28.4 \times 0.73 \times \sqrt{23.9}} = 1.20$$

$$0.5 + \sqrt{0.085 - 0.05\psi} = 0.5 + \sqrt{0.085 - 0.55 \times (-1)} = 0.874$$

$$\bar{\lambda}_p = 0.874,\ \text{so}\ \rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^{\,2}} = \frac{1.2 - 0.055\,(3+(-1))}{1.2^2} = 0.757$$

The effective depth of the compression part of the web is therefore $\rho \times b_c$ = 0.757 × 730 = 553 mm
From Table 4.1, the stable length adjacent the compression flange is $0.4b_{eff}$ = 0.4 × 533 = 221 mm
The ineffective length (the 'hole') = 730 – 533 = 177 mm
According to clause 4.3(5) the stress in the flange is considered at the mid-plane of the flange.

Stress Block

By postulating a position of the neutral axis, the stresses at locations throughout the cross section can be computed. The stress in the web is limited to f_{yw}, which in this case is 355 N/mm². Knowing the stresses and cross sectional dimensions, the tension force and compression force can be calculated, compared, and the position of the neutral axis adjusted until equilibrium is achieved. This is a job best left to electrons within a spreadsheet…
In this case, the solution is shown in Figure 6. Summing the product of the stress and area, the following forces are obtained:

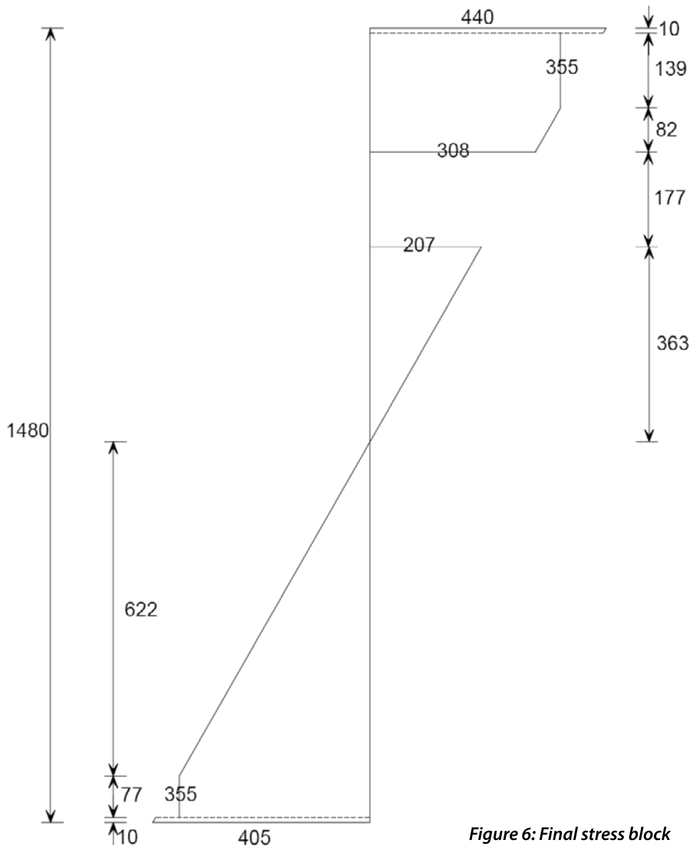


Figure 6: Final stress block

Compression flange	440 × 400 × 20	= 3520000 N
web "plateau"	139 × 355 × 12	= 592140 N
web above "hole"	0.5 × (308+355) × 82 × 12	= 326196 N
web below "hole"	0.5 × 207 × 363 × 12	= 450846 N
	Summation	= 4890 kN
Tension flange	405 × 400 × 20	= 3240000 N
web "plateau"	355 × 77 × 12	= 328020 N
web	0.5 × 622 × 355 × 12	= 1324860 N
	Summation	= 4890 kN

Equilibrium of force has been achieved.

Moment resistance

Once equilibrium has been found, the moment resistance is simply the summation of the force in each element, multiplied by the lever arm.

3520000 × 771 = 2.71 × 10^9 Nmm
592140 × 692 = 409 × 10^6 Nmm
326196 × 581 = 189.5 × 10^6 Nmm
450846 × 2/3 × 363 = 109 × 10^6 Nmm
3240000 × 709 = 2.30 × 10^9 Nmm
328020 × 661 = 217 × 10^6 Nmm
1324860 × 2/3 × 622 = 549 × 10^6 Nmm
Moment resistance = 6485 kNm

Conclusions to Part 1

Despite how it may appear at first glance, this process is not overly onerous and is suited to a spreadsheet application – perhaps with some VBA to determine the neutral axis. Different solutions can then be readily examined and resistance calculated. In Part 2, the lateral torsional buckling resistance and the shear resistance will be calculated.

In Part 2 of the article, David Brown of the SCI discusses the lateral torsional buckling resistance and shear resistance of hybrid sections.

Cross sectional moment resistance – made easy

In Part 1, the cross section was classified as Class 4 (due to the web), leading the designer to BS EN 1993-1-5 and the challenge of calculating effective section properties. There is an easier approach, but the penalty is a conservative resistance.

Clause 5.5.2(12) of BS EN 1993-1-1 allows the class of the cross section to be based on that of the flanges alone, as long as it is assumed that the web is designed for shear alone and does not contribute to the bending resistance. This simple approach does not relieve the designer of considering shear lag and plate buckling, as clause 6.2.1(2) of BS EN 1993-1-1 makes clear.

In the example presented in Part 1, the flanges were Class 3. If the web is assumed to make no contribution to the bending resistance, then the resulting stress diagram is shown in Figure 1.

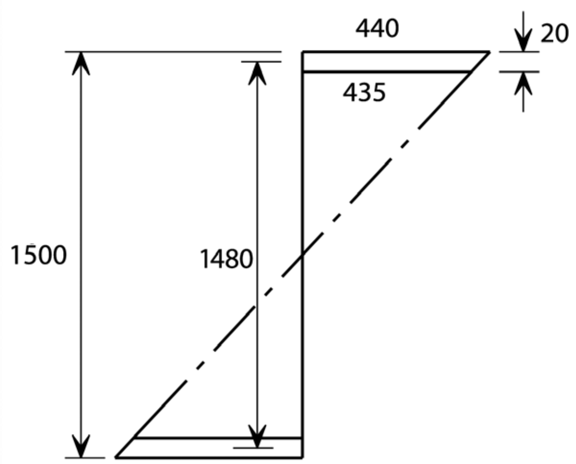


Figure 1: Class 3 stress diagram, neglecting the web

As demonstrated in Part 1, the flange does not suffer from shear lag effects or plate buckling effects.

Thus the bending resistance of the cross section = 400 × 20 × 437.5 × 1480 × 10^{-6} = 5180 kNm

The resistance calculated in Part 1 was 6485 kNm; an increase of 25% over the simple approach.

Lateral-torsional buckling

There is no real challenge in LTB resistance. The section properties must be calculated, or the on-line M_{cr} tool on *steelconstruction.info* can be used to calculate the section properties – and to compute M_{cr}, of course.

For those who prefer to see the calculations:

$$I_z = 2 \times \frac{20 \times 400^3}{12} + \frac{1600 \times 12^3}{12} = 213 \times 10^6\ \text{mm}^4$$

$$I_w = \frac{I_z \times h_0^{\,2}}{4} = \frac{213 \times 10^6 \times (500 - 20)^2}{4} = 1.17 \times 10^{14}\ \text{mm}^6$$

$$I_t = \frac{2}{3}\, b_f t_f^{\,3} + \frac{1}{3}\, h_w t_w^{\,3} = \frac{2}{3} \times 400 \times 20^3 + \frac{1}{3} \times 1460 \times 12^3 = 2.97 \times 10^6\ \text{mm}^4$$

(the calculation for I_t is a simplification; online tools give 2.92 × 10^6 mm^4)

Assuming the loading is uniformly distributed, then C_1 = 1.13. At this point, the value of M_{cr} can be calculated using the steel designer's favourite (or maybe not?) expression:

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z}}$$, which computes to 5966 kNm.

It does seem rather odd to use online tools to calculate section properties and then compute M_{cr} by hand, when the same software will calculate M_{cr} to be 5970 kNm.

The general case given in clause 6.3.2.2 of BS EN 1993-1-1 is used for fabricated girders.

h/b = 1500/400 = 3.75, so curve *d* is used and α_{LT} = 0.76

Working through the expressions in clause 6.3.2.2, the reduction factor χ_{LT} = 0.447 and M_b = 0.447 × 6485 = 2899 kNm.

Web resistance

The web must resist shear, of course, but must also prevent the flange buckling in the plane of the web (a possibility for very tall, thin webs). When calculating the shear resistance, the presence of stiffeners (or not) makes a significant difference, as will be demonstrated. The shear resistance comprises a contribution from the web, but also an additional contribution from the flanges. The flanges can span between stiffeners and mobilise a tension field mechanism (see Figure 2) – essentially like a tension member in a truss. The contribution from the flanges is generally small and can only be used if the flanges are not fully utilised in carrying moment – so a simple solution is to neglect the additional resistance.

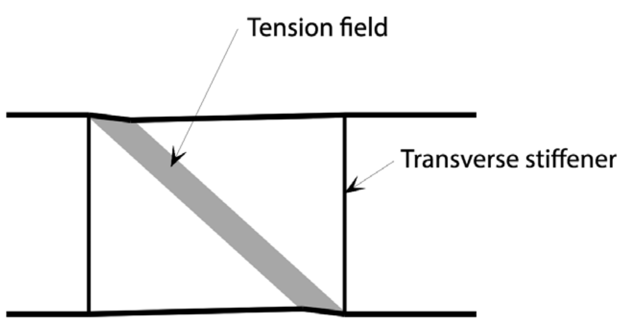


Figure 2: Flange contribution to web shear resistance (from Hendy and Murphy)

Shear resistance

The consideration of shear resistance starts in BS EN 1993-1-1, with an expectation that for tall thin webs, shear buckling will be critical (clause 6.2.6(6)).

For the web, $\varepsilon = \sqrt{\dfrac{235}{355}} = 0.81$. With η=1, then

$$\frac{h_w}{t_w} = \frac{1460}{12} = 121.7; \quad 72\frac{\varepsilon}{\eta} = 72 \times \frac{0.81}{1} = 53.3$$

A check of shear buckling is therefore required (the web would need to be 25 mm thick before a check of shear buckling is not needed), which takes designers to BS EN 1993-1-5 clause 5.2. Initially, the resistance of an unstiffened web will be calculated.

Unstiffened section - contribution from the web – clause 5.3

A series of intermediate values are required, determined from Annex A.3 and Annex A.1 of BS EN 1993-1-5

From Annex A.3, if the unstiffened span is 8 m, then

$\dfrac{a}{h_w} = \dfrac{8000}{1460} = 5.48$. Because this value is greater than 1.0, the shear buckling coefficient, k_τ is given by: $k_\tau = 5.34 + 4\left(\dfrac{h_w}{a}\right)^2 + k_{\tau sl}$

$k_{\tau sl}$ relates to longitudinal stiffeners, so is zero in this example. Therefore, $k_\tau = 5.47$

From Annex A.1, $\sigma_E = 190000\left(\dfrac{t}{b}\right)^2 = 190000 \times \left(\dfrac{12}{1460}\right)^2 = 12.84$

From clause 5.3(3), $\tau_{cr} = k_\tau \sigma_E = 12.84 \times 5.47 = 70.2$

Then $\bar{\lambda}_w = 0.76\sqrt{\dfrac{f_{yw}}{\tau_{cr}}} = 0.76\sqrt{\dfrac{355}{70.7}} = 1.71$

Because $\bar{\lambda}_w > 1.08$ (From Table 5.1), $\chi_w = \dfrac{0.83}{\bar{\lambda}_w} = \dfrac{0.83}{1.71} = 0.49$

Thus the contribution from the web =

$$\frac{0.49 \times 355 \times 1460 \times 12}{\sqrt{3} \times 1.0} \times 10^{-3} = 1745 \text{ kN}$$

The contribution from the flange, assuming $M_{Ed} = 0$ at the supports, is 33.5 kN, so less than 2% of the contribution from the web – and small enough to be neglected.

The plastic shear resistance should be verified in accordance with clause 6.2.6(2) of BS EN 1993-1-2, and in this example is found to be 1980 kN – as expected, shear buckling is critical.

Stiffened section - contribution from the web – clause 5.3

If intermediate transverse stiffeners are provided, the shear resistance increases. In this case, it is assumed that the transverse stiffeners are spaced such that $a/h_w = 2$ (Figure 3).

Then $k_\tau = 6.34$; $\sigma_E = 12.84$; $\tau_{cr} = 81.4$; $\bar{\lambda}_w = 1.59$; $\chi_w = 0.522$ and the web resistance increases to 1874 kN.

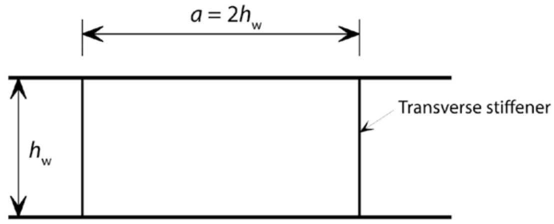


Figure 3: Aspect ratio of web panel

In this stiffened case, the additional contribution from the flanges increases to 96 kN, so is more significant.

Moment resistance or shear resistance – which is critical?

If, as calculated above, the LTB resistance, M_b is 2899 kNm, and assuming that M_{Ed} is 90% of M_b, then the UDL is 326 kN/m

The end shear is therefore 1304 kN, so in this example, even the unstiffened shear resistance of 1745 kN is sufficient.

Flange induced buckling (clause 8)

To prevent flange induced buckling, the following criteria must be satisfied:

$$\frac{h_w}{t_w} \le k\frac{E}{f_{yf}}\sqrt{\frac{A_w}{A_{fc}}}$$

Since the elastic moment resistance has been calculated, the factor $k = 0.55$

Then $\dfrac{1460}{12} \le 0.55\,\dfrac{210000}{440}\sqrt{\dfrac{1460 \times 12}{7999}}$ or $121.7 \le 388.5$

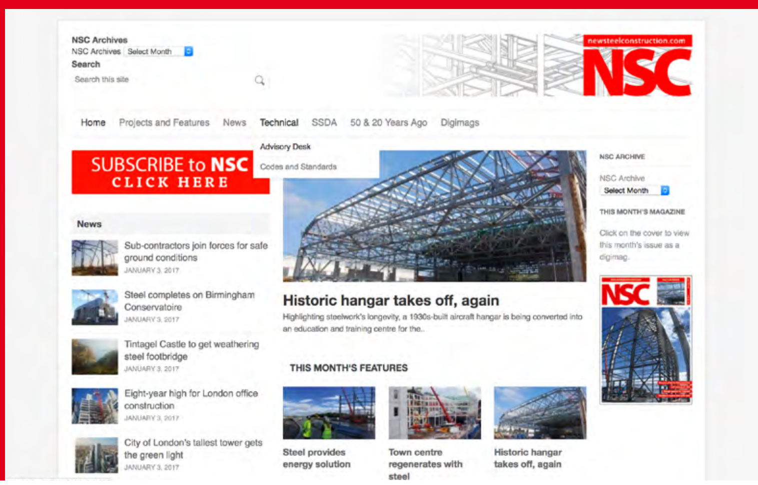
The criteria is satisfied, so there is no flange induced buckling of the web.

Further guidance

The Designer's Guide to EN 1993-2 by Hendy and Murphy has extensive coverage of BS EN 1993-1-5. *Structural Design of Steelwork to EN 1993 and EN 1994* by Martin and Purkiss contains examples of fabricated section design including the design of intermediate transverse stiffeners and end posts. Readers of the second resource should note that the larger elastic section modulus (not the smaller) is calculated in example 5.4, with a consequent problem in the calculated resistance.

Conclusions from Part 2

This article has attempted to introduce the design rules that apply to any fabricated beam section with a Class 4 web – the fact that the web and flanges are of different grades is not significant. The hard work was completed with the calculation of the cross section moment resistance, covered in Part 1. On-line tools cannot help here, as the calculation of M_{cr} – where on-line tools are invaluable – is based on the gross section properties, not the effective section properties needed for the cross sectional resistance.



Brittle fracture: selection of sub-grade for 'quasi-static' structures

Selection of steel sub-grade is an important responsibility for all steel designers, to manage the risk of brittle fracture. David Brown of the SCI discusses a new publication (P419[1]) which presents steel thickness limits which may be used in buildings where fatigue is not a design consideration.

The Eurocode basis

Designers familiar with BS EN 1993-1-10 or PD 6695-1-10 will know that the selection of a steel sub-grade depends on the stress level, the type of detail, the service temperature and the material thickness. BS EN 1993-1-10 presents (in Table 2.1) limiting thicknesses for steel sub-grades, depending on the so-called reference temperature. The reference temperature is the service temperature, but then subject to various adjustments.

In the UK, significant modifications are made to the 'core' Eurocode, via the National Annex. The effect of the UK NA is accounted for in the thickness limits presented in PD 6695-1-10, which contains look-up tables for steel in buildings (internal and external) and bridges. Worked examples showing how to select a steel sub-grade using the Eurocode, PD and UK NA were presented in NSC, October 2016[2].

A JRC publication[3] provides comprehensive background on how the thickness limits in BS EN 1993-1-10 were derived. The background document is not easy to digest, but after the various formulae have been committed to a spreadsheet, it is possible to replicate the values found in Table 2.1 of the Standard. For anyone rising to that challenge, there is some (variable) degree of rounding in the printed table.

The effect of fatigue

The Eurocode states in the Note to clause 2.1(2):

"For elements not subject to tension, welding or fatigue, the rules can be conservative. In such cases evaluation using fracture mechanics may be appropriate, see 2.4 (of the Standard). Fracture toughness need not be specified for elements only in compression."

The JRC background document is clear in paragraph 1.4.3(2) that Section 2 of the Eurocode was developed for structures subject to fatigue (such) as bridges, crane runways or masts subject to vortex induced vibrations. The background document goes on to say:

"its use for buildings where fatigue plays a minor role would be extremely safe-sided"

The effect of fatigue is to cause an initial crack to grow to a much larger design crack. The assessment of sub-grade is then carried out on the basis of the design crack.

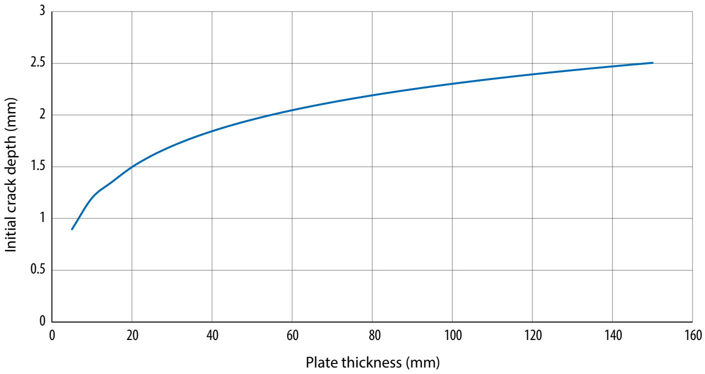


Figure 1: Initial crack size

The initial crack size is related to the thickness of the element, as shown in Figure 1.

The size of the initial crack assumed in the Eurocode is one that might be missed during inspection after fabrication. The JRC background document demonstrates that the minimum crack width detectable by inspection methods after fabrication is smaller than the assumed crack width, implying that the assumed crack sizes should be detected. It is assumed that the steelwork is fabricated, welded and inspected in accordance with the requirements of BS EN 1090-2.

The effect of fatigue is to grow the initial crack to a much larger defect, as shown in Figure 2. The curve is a representation of the expression given in the JRC background document.

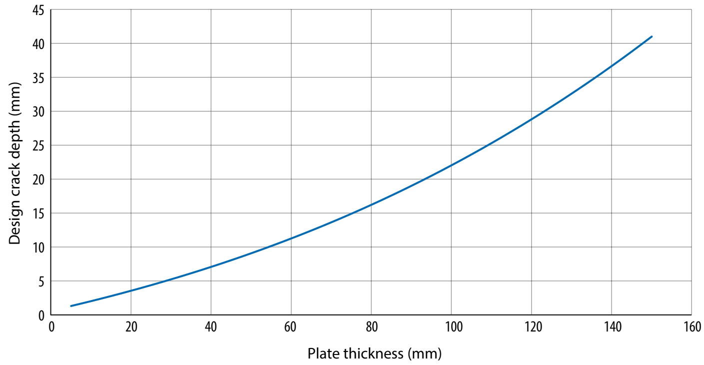


Figure 2: Design crack size

The design crack depth a_d is expressed as:

$a_d = 2 \times 10^{-6}\, t^3 + 6 \times 10^{-4}\, t^2 + 0.1341t + 0.6349$

where:

t is the plate thickness.

It is interesting (and possibly sobering) to note that the Standard is based on 0.5 million cycles, equivalent to a 25 year life. Usually, 2 million cycles are assumed for a 100 year life. The Eurocode approach therefore anticipates inspection of a fatigue-sensitive structure at 25 year intervals, and repair if necessary to reinstate the original conditions. Such inspection would be normal in bridges.

The new publication

The new guide reduces the growth due to fatigue. The word "reduces" has been used, since to assume no growth at all would be to eliminate the effect of fatigue altogether. After consultation, it was decided that some fatigue should be allowed for even though for the structures within the intended scope, fatigue would not be a design consideration. Based on indicative guidance from a DIN Standard, 20,000 cycles was chosen to allow for some fatigue in structures where fatigue is not a design consideration – most buildings. The term "quasi-static" would cover such structures – in reality that there may be some limited cycling of load, but that would not normally be considered – the design approach is to consider all loads as static.

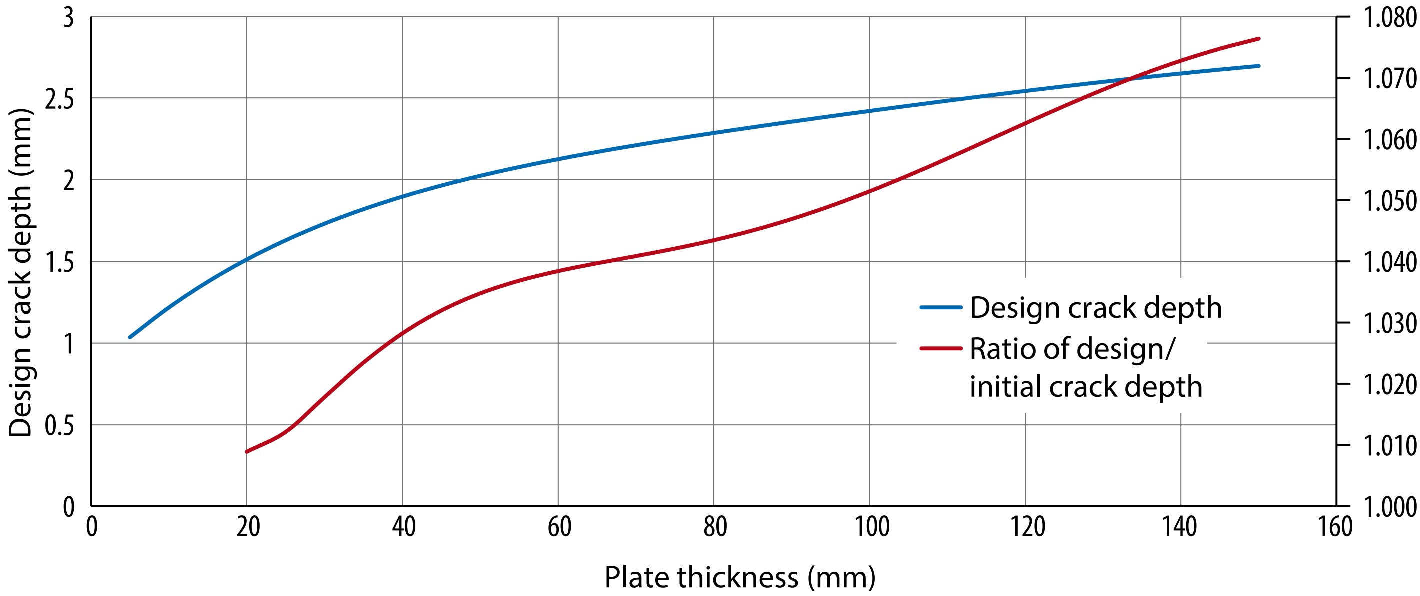


Figure 3: Initial and design crack depths (20,000 cycles)

The key to the new approach is the formula to express the crack growth under 20,000 cycles. Experts at the University of Aachen (who were also deeply involved with the development of the Eurocode) provided this all-important expression.

For structures where fatigue is not a design consideration, the new expression for crack growth is given by:

$$a_d = 3.6258 \times 10^{-11}\, t^5 - 2.2316 \times 10^{-8}\, t^4 + 5.3365 \times 10^{-6}\, t^3 + 6.3837 \times 10^{-4}\, t^2 + 0.045124\, t + 0.82483$$

The resulting design crack depth is only a little larger than the initial crack depth, as can be seen in Figure 3. The ratio (design crack depth)/(initial crack depth) is plotted on the right hand axis.

Based on the revised design crack size, limiting thicknesses are determined and presented in the new publication, including an equivalent Table 2.1 from the Eurocode, and equivalent look-up tables from PD 6695-1-10 for use in the UK.

Additional modifications

The title of this section is deliberately misleading. Apart from the revised crack growth, there are no other modifications to the process described in the background document. All the assumptions made in developing Table 2.1 of the Eurocode are followed, without exception.

The provisions of the UK NA have also been followed, without exception. These include:

- The adjustment for detail type, described in NA.2.1.1.2;
- The adjustment for the Charpy test temperature, described in NA.2.1.1.4;
- The adjustment for the applied stress, described in NA.2.1.1.5, which means that the limiting thickness values are based on an applied stress of 75% of the yield strength;
- The adjustment for the steel strength grade, as described in NA.2.1.1.6.

These provisions of the UK NA are listed simply to emphasise that they have been properly observed in developing the tables presented in the new publication. The publication does not allow for impact or cold forming; in these cases the limiting thicknesses can be calculated from the tabulated data provided.

Revised thickness limits

The effect of much reduced crack growth is very significant. The limiting thicknesses are much larger than those in PD 6695-1-10, which allowed for crack growth under 0.5 million cycles. Table 1 presents a comparison for external S355 J0 material, covering combinations 4 to 10 (the welded detail types).

Scope of the new publication

Firstly, if the structure under consideration is subject to fatigue, the tables in the new publication should not be used; The Eurocode, NA and PD 6695-1-10 must be followed in the UK. For structures where fatigue is not a design consideration, the new publication presents less onerous thickness limits. For structures outside the UK and not subject to fatigue, the new publication provides an equivalent to Table 2.1 of the Eurocode which may be used as a basis for steel sub-grade selection.

References

1 *Brittle fracture: selection of steel subgrade to BS EN 1993-1-10* Brown, D. and Cosgrove, T. SCI, 2017

2 *The selection of steel-subgrade* Henderson, J. R., NSC, October 2016

3 *Commentary and worked examples to EN 1993-1-10 "Material toughness and through thickness properties" and other toughness oriented rules in EN 1993* Sedlacek, G. et al, JRC, 2008

Detail type	Tensile stress level $\sigma_{Ed}/f_y(t)$						
Welded - moderate	≤ 0	0.15	0.3	≥ 0.5			
Welded - severe			≤ 0	0.15	0.3	≥ 0.5	
Welded - very severe				≤ 0	0.15	0.3	≥ 0.5
	Maximum thickness (mm)						
	Comb.4	Comb.5	Comb.6	Comb.7	Comb.8	Comb.9	Comb.10
PD6695-1-10	67.5	55	45	37.5	30	22.5	17.5
P419	200	200	200	188.5	145.5	95.5	65.5

Table 1: Limiting thicknesses for external S355 J0 steel

Cast-in plates

Richard Henderson of the SCI discusses the design of plates cast into concrete to connect to steel beams and the forthcoming SCI design guide.

No 1 Spinningfields Manchester

Introduction

It is a truth universally acknowledged, that a simple beam in proximity to a concrete wall, must be in want of a cast-in plate… Wandering around cities examining construction activity, one cannot but be struck by the number of buildings in which the lateral stability system is provided by concrete cores and the floor beams and columns are structural steel. It is perhaps surprising that until now there has been no design guide for cast-in plates available in the UK.

What do cast-in plates do?

The floors that surround concrete cores are supported on steel beams that in turn are supported by the core walls. Steel plates are cast into the wall during construction and subsequently, as the steelwork is erected, connections are made to the cast-in plates. The connections carry the design loads from the beams: the vertical reaction at the end of the beam and possibly a horizontal force (e.g. from wind loads on the façade and a separate tie force for robustness). The cast-in plates must therefore transmit these forces into the concrete walls through shear studs, reinforcing bars, anchor plates and the like, fixed to the back of the plate and embedded in the concrete.

Simple Connections

Many steel buildings in the UK have been designed with braced frames providing lateral stability and simply supported beams carried by the steel columns. Standard connections have been developed which are able to resist the vertical reaction at the end of the beam but are sufficiently flexible to allow the beam to take up the end slope corresponding to a simple support. The connections are also capable of resisting the horizontal tie force required to provide adequate robustness, in a separate load case. The details are published in the SCI 'Green book', publication P358 . Various types of connections are included: partial and full-depth end plates and fin plates. Tests on connections to beams up to 610 serial size have been carried out to demonstrate the behaviour of the connections.

Coexistent shear and tension

Connections may be required to carry shear and axial force in the same load case. Where beams are supported on inclined columns, significant horizontal forces may be developed. If risers for building services are grouped round a concrete core, the floor slab may stop short, leaving no opportunity to transfer horizontal forces through the floor slab. In such cases, the connection falls outside the details in the 'Green Book' and the connection must be designed from first principles. The ability of such a connection to rotate to relieve a fixed-end moment while carrying a significant horizontal force must be carefully considered.

Connections to cast-in plates

Where a building has been designed with simply supported beams carrying shear and no horizontal forces and with tying forces in a separate load case, connections can be taken from the 'Green Book'. A common form of connection is a fin plate welded to a cast-in plate. An end-plate connection could be formed by welding a Tee to the cast-in plate, but such an arrangement where a beam has to be erected between two vertical surfaces

Photo courtesy of William Hare Ltd

may make the steelwork less easy to erect than with a fin plate. If at all possible, it is preferable to transfer the horizontal tie force through the concrete floor slab into the concrete core, to avoid the need to design the core to resist a local tension delivered through the cast-in plate and provide any necessary shear reinforcement.

Issues to consider

Split of responsibility

The design of concrete cores usually belongs to the building structural engineer. In most contractual arrangements, the required performance of the connections and the forces they are to resist are also determined by the structural engineer. The design of connections between steel elements are usually the responsibility of the steelwork contractor because they can be detailed to suit the production process. The work of these two parties comes together at the face of the cast-in plate. For the smooth progression of the construction process, it is necessary for each party to know the assumptions, behaviours and limitations of the structural components that come together at this point.

The design responsibilities of the different parties must be clearly defined and understood. It is logical for the split of design responsibilities to lie on the face of the cast-in plate. The design of the plate and the embedded elements fall to the structural engineer, who is in control of all the details of the concrete core wall, including geometry, strength of concrete and reinforcement details. The steelwork contractor selects and details the element welded to the cast-in plate and must also be satisfied that the element chosen (e.g. a fin plate) working together with the cast-in plate will perform as required.

The work of the structural engineer is in advance of the steelwork contractor and therefore it is necessary for the engineer to be aware of the impact of decisions on the steelwork contractor's subsequent activities. For example, assuming beams are simply supported and specifying shear and axial forces in the same load case means that connections cannot be selected from the standard details in the 'Green Book' and must be designed from first principles.

Accuracy of construction and agreed deviations

Concrete cores or shear walls are likely to be specified and built using the National Structural Concrete Specification and structural steelwork fabricated and erected using the National Structural Steelwork Specification . These documents have different requirements for the accuracy of erected elements which must be reconciled where the different elements come together, notable at the cast-in plates. A set of deviations must be agreed by the appropriate parties early in the project to avoid problems later.

Adjustments to allow for the connections between the steel and concrete elements to be made are often arranged as follows. After the concrete core is constructed, the positions of the cast-in plates are surveyed and a fin plate cut to suit the as-built position and site-welded to the plate. A similar process would be used if a Tee stub were to be preferred. The steelwork must be detailed to allow for a longer or shorter fin plate, depending on the results of the survey. The resistance of the cast-in plate must be such as to carry the loads resulting from a connection anywhere within the agreed positional deviations. Maximum values of ± 35 mm in plan position are typical.

Design Guide

The forthcoming design guide is going through its final checks before publication. It has been funded by BCSA and Steel for Life and a working party with members drawn from various parts of the construction industry has made contributions and made comments on drafts of the guide.

The guide proposes a design model for the design of cast-in plates which is based on design codes. As might be expected, these are Eurocodes 2, 3 and 4, dealing with concrete, steel and composite construction respectively. A consequence of this approach is that the design model uses shear studs to resist shear forces and steel reinforcement to resist tension. The tension is transferred into the concrete via bond with the reinforcement. Eurocode 4 considers shear studs in combined shear and tension but states that tensions greater than one tenth of the stud shear resistance are outside the scope of the code. There is also at present no code-based interaction formula for assessing combined shear and tension on shear studs.

The guide discusses the issues outlined above in more depth and addresses other issues such as weld details for reinforcement, the potential for thermal expansion of the cast-in plate during welding and handling of the cast-in plate during construction. The guide also presents a design example of a cast-in plate to receive a simple connection from a 610 serial size UB.

It is intended that the guide will identify many of the issues which are relevant to this form of construction for structural engineers who are embarking on such a project for the first time. The guide will also provide a starting point for discussions between the different parties involved.

References

1 Joints in steel construction Simple joints to Eurocode 3, (P358), SCI, 2014
2 National Structural Concrete Specification, 4th Edition, Concrete Centre, 2010
3 National Structural Steelwork Specification, 5th Edition CE Marking version, (52/10) BCSA, October 2010

Design Guide

SCI P416 The design of cast-in plates was published on 8th December 2017 and is available via www.steelconstruction.info

Southbank place

Design of buildings to resist external accidental explosions

Bassam Burgan of the SCI discusses the design of low to medium rise buildings against external explosions and SCI's forthcoming design guide.

Figure 1: Northgate Building, Buncefield (courtesy of the Health and Safety Laboratory)

Blast caused by industrial accidents

The chemical and petrochemical industries process substances that are essential to our lives (e.g. mineral fertilizers, fuels and pharmaceuticals). Under certain conditions, such substances may be flammable, explosive or toxic. These industries continuously strive to improve the safety of their manufacturing processes and today, they are amongst the safest industrial sectors. However, when accidents happen, they can impact not only the industrial facility itself, but also its neighbourhood, sometimes extending several miles from the accident site. Major high profile incidents such as Flixborough, UK (1974), Seveso, Italy (1976), Bhopal, India (1984), Shell Norco, USA (1988), Phillips Pasadena, USA (1989), BP Texas City, USA (2005), Buncefield, UK (2005), Caribbean Petroleum Corporation, Puerto Rico (2009) and Indian Oil Company, India (2009) demonstrated the loss of life and property and the environmental, economic and reputational damage that can be caused by such accidents.

An explosion caused by an industrial accident results in a blast wave. High pressure blast waves that travel through air at a velocity greater than the speed of sound are referred to as shock waves (Figure 2(a)). By contrast, lower amplitude blast waves travelling at speeds below the speed of sound are referred to as pressure waves(Figure 2(b)).

Blast wave interaction with building structures

Both shock and pressure waves result in a "global" action on buildings at adjacent sites. The response of a building to blast is influenced by the magnitude, rise time and duration of the blast wave and there is strong coupling between the action and the building due to reflection and diffraction effects. Once the envelope of a building fails, pressure distribution can be amplified by multiple reflections inside the building and building elements, such as floors, are exposed to upward pressure, an action for which they are not normally designed.

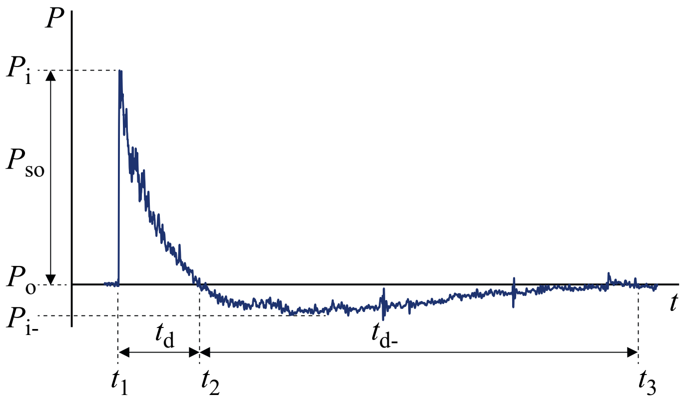


Figure 2(a) Shock wave

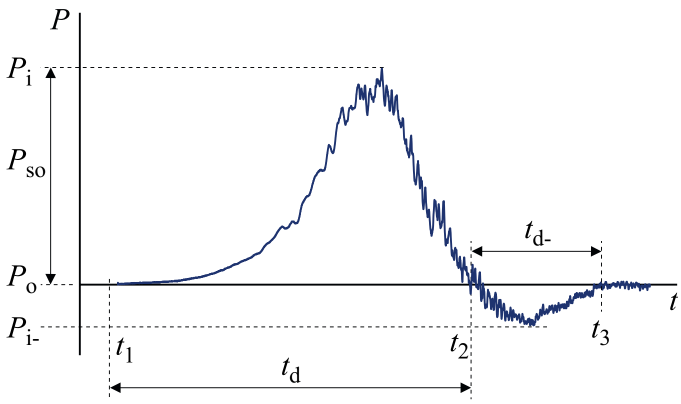


Figure 2(b) Pressure wave

When a blast wave strikes the surface of a building, the air molecules at the front of the blast wave are arrested abruptly by the building's surface. These molecules are compressed by the trailing blast wave, which causes the reflected pressure to be greater in magnitude than the incident pressure (Figure 3). The increase in the magnitude of reflected pressure depends on the angle at which the incident blast wave strikes the building surface.

Blast wave interaction with a building structure was studied in a recent SCI led European project in which experimental studies quantified the nature and distribution of the blast actions on a typical "out-of-town" office building (reference building)- see Figure 4. The tests were performed on two 1:60 scale models of the building, one solid and one with openings to investigate blast wave interaction with the building floors and walls within the building. The scale models were tested at different angles of incidence of the blast to determine the variation in the reflected pressure as a function of the angle of incidence. The results were also used to assess the accuracy of analytical equations used for calculating blast action and for providing guidance on numerical (computational fluid dynamics) modelling of blast actions.

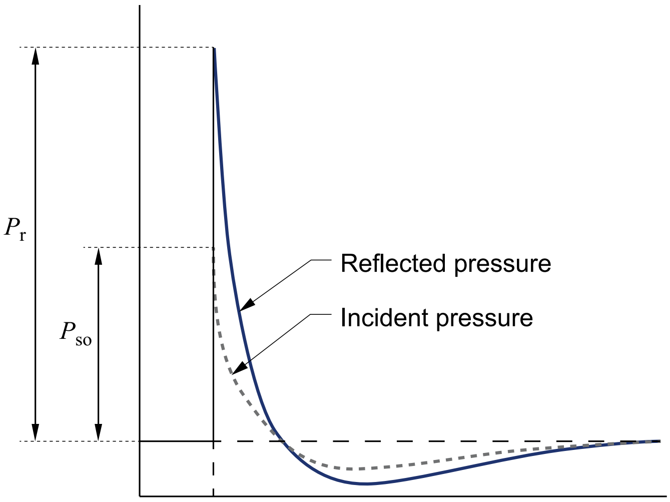


Figure 3: Pressures due to incident and reflected shock wave

Response of building components and whole building to blast

Large scale blast tests were also performed on building elements and sub-assemblies including masonry and composite cladding, simple beam-column connections and composite floors. The purpose of these tests was to study the transfer of blast actions from the envelope to the frame of the building and the performance under blast action of building elements such as floors and connections. The results were used to validate detailed numerical models and carry out numerical parametric studies. Based on this work, sub-models (a component connection model and a 2D flat shell element for the composite slab) were calibrated and used in whole building finite element models. The whole building model enabled the behaviour of the reference building under a series of explosion scenarios to be studied, identified failure modes and was used to propose retrofitting strategies.

New SCI design guide

The project led to the development of a new SCI design guide which provides recommendations and advice for the structural design of low to medium rise steel-framed buildings (typically two to five storeys high) subjected to blast action due to external explosions. Step-by-step methods are given for the calculation of the resultant blast action on a building as a result of the interaction of the blast wave with the building elevations.

Guidance is given on the calculation of material properties to be used in the design of members. Yield stress design values are increased by a static increase factor to account of the fact that the actual yield strength of the common grades of steel (up to S355) is frequently greater than its guaranteed minimum value by more than 25%. This reduces conservatism in a design situation which involves an accidental combination of actions and ensures that the forces and moments transmitted from members to connections are not underestimated. To avoid failure of the connections, the static increase factor is not applied to the connection components. This is similar to the approach adopted in BS EN1998-1 for capacity design.

The mechanical properties of structural steels are affected by the rate of load application. The guide therefore recommends values to account for the increase in yield and ultimate strength due to the dynamic nature of the blast action.

In blast response analysis, the calculation of forces, moments and deformations requires the use of dynamic analysis of the building structure. Simplified analysis software (www.blastresponse.com) was developed comprising three modules: (i) an advanced single degree of freedom (SDOF) model capable of accounting for generalised boundary conditions and loading of a structural element, (ii) SDOF composite floor model and (iii) a multi-degree of freedom (MDOF) model capable of analysing 3D building structures with general grid layouts. The software was validated using advanced finite element analysis.

The response of the building frame members is verified by reference to deformation limits (both deflection and rotation) which correspond to different damage levels. These depend on member type and slenderness and on the nature of loading acting on the member. Connections are also verified by reference to rotation limits. To ensure overall frame stability, inter-storey drift and frame member rotation limits are imposed.

Member capacity checks are performed in accordance with BS EN1993-1-1. This is modified in some cases to allow for the large deformations that may be tolerated in the case of blast response. Furthermore, the verification of columns in braced bays is modified as per BS EN1998-1 to ensure that the columns can resist the additional forces transmitted to them by the bracing members.

Design examples

The design guide is illustrated by a series of design examples based on the reference building. They include two explosion scenarios of different severity and the design verification of (a) a lintel supporting building cladding, (b) a composite floor and (c) the overall assessment of the building frame and the redesign of the bracing and columns to resist the more severe of the two blast scenarios.

The project was funded by the European Union's Research Fund for Coal and Steel (RFCS) under grant agreement no. RFSR-CT-2013-00020 and the UK Centre for the Protection of the National Infrastructure (CPNI).

Reference Building

Advisory Desk 2017

AD 401a: Appropriate anchorage of parallel decking

Revised

Where profiled steel decking is parallel to the supporting beam, BS EN 1994-1-1 (incorporating corrigenda April 2009: 2004) allows the shear resistance of a headed stud to be based on the resistance in a solid slab multiplied by a reduction factor that is given in expression (6.22), without the need for additional reinforcement, provided that the decking is continuous across the beam or is 'appropriately anchored' and the studs are located within a certain region (Figures 6.12 and 9.2).

One purpose of providing appropriate anchorage is to prevent loss of any containment to the concrete rib provided by the decking, thus avoiding a reduction in stud resistance. A second purpose is to prevent so-called splitting of the concrete, which would be a non-ductile mode of failure.

Where the sheeting is not continuous across the beam and is not appropriately anchored, clause 6.6.4.1(3) requires 6.6.5.4 to be satisfied, which involves dimensional restrictions and rebar bent into the trough, as illustrated in Figure 6.14. It is impractical, on the scale of typical composite slab profiles, to provide bent bars such as would be provided in a formed haunch. It is therefore all but obligatory to provide appropriate anchorage and 6.6.4.1(3) notes that the means to achieve appropriate anchorage may be given in the National Annex.

UK NA.4 refers to Non-Contradictory Complementary Information (NCCI), which is available in a recently updated NCCI document (PN003c-GB), now available on *www.steel-ncci.co.uk* and defines three alternatives for ensuring decking is appropriately anchored when through deck welded studs are not present. In order of increasing 'complexity' these are presented as Options 1 to 3 here.

Option 1

Finite Element Modelling (FEM) has been used to show that when the geometry of the haunch and detailing of the shear studs satisfy the requirements defined below, then only nominal fixity is needed in order to contain the concrete around the studs and prevent longitudinal splitting of the slab. The provision of nominal fixity (1 kN/m) is valid when:

- The decking geometry, flange width and stud placement is such that the angle between the base of the stud and shoulder of the decking is no more than 50°.
- There are single studs fixed along the beam centreline, providing edge cover of not less than 50 mm. Multiple studs at a given cross section must be avoided because of their potential to transfer a higher force into the concrete.
- The longitudinal stud spacing is not less than 150 mm. When studs are more closely spaced there is an increased likelihood of interaction between adjacent studs resulting in slab splitting, but the FEM

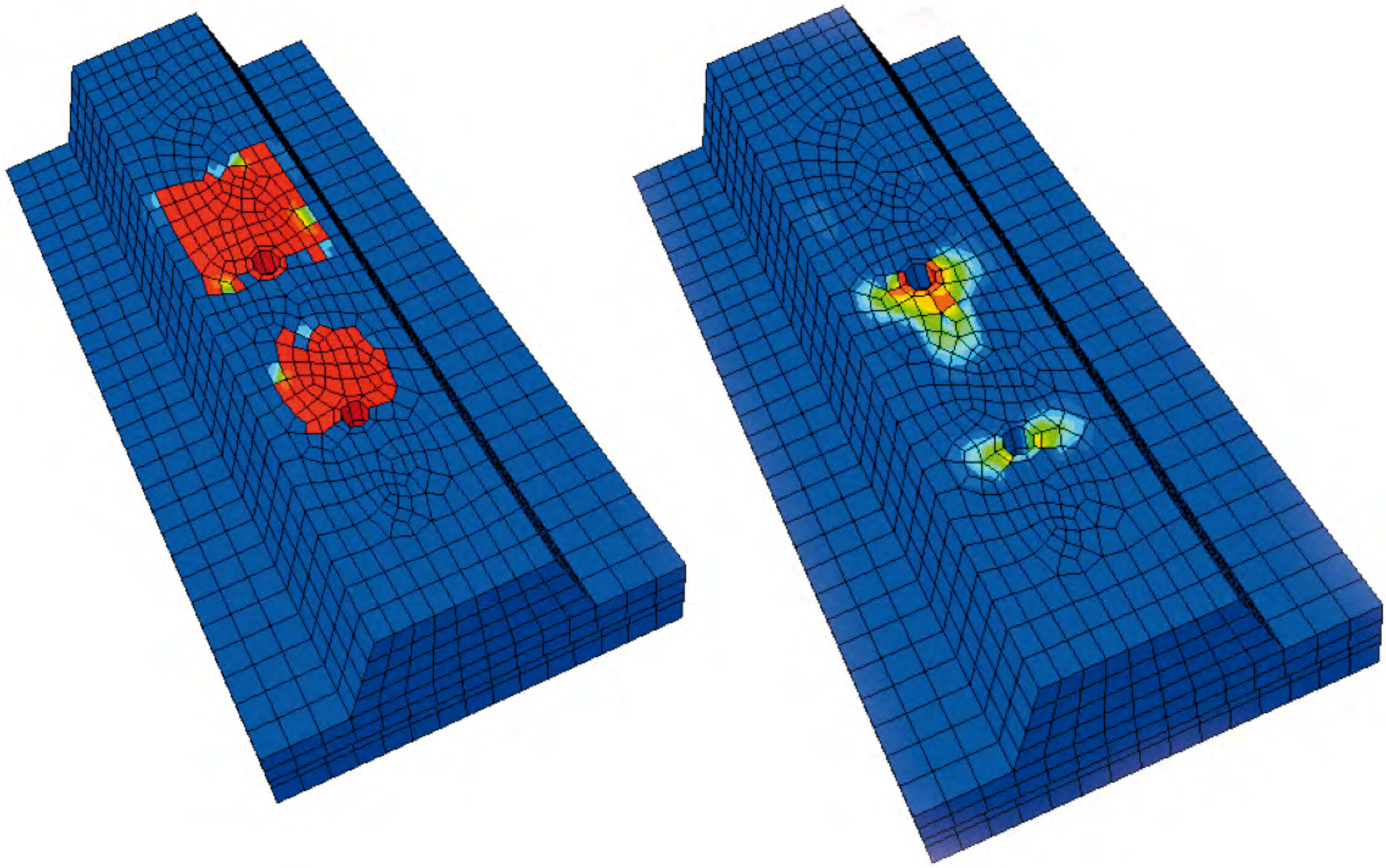


Figure 1: : Concrete damage in a) compression and b) tension at a slip of 6 mm

demonstrated that even at slips in excess of 10 mm - which is almost twice the slip anticipated by BS EN 1994-1-1; there is no interaction for studs at 150 mm centres (Figure 1).

- The beam is simply supported.

Note that the detailing rules above are similar to those presented in BS EN 1994-1-1 as necessary to assure adequate concrete confinement around the studs in a haunch.

Option 2

When the limits given above are not satisfied, it seems reasonable to assume that it will suffice to provide resistance equal to the force which would be needed to 'unfold' the profile if it were subject to transverse tension, as this sets a limit to the containment provided by the profiled decking. It can readily be calculated that a 60 mm deep profile, 0.9 mm thick, grade S450, with plastic hinges top and bottom, will unfold at less than 4 kN/m. Fixings at 250 mm centres, which is also a spacing close enough to ensure reasonable proximity to the zone of influence of any one stud, should suffice to provide this level of fixity. With thicker decking, the bearing resistance of the screw or nail will improve more than commensurately with the demands made on it. With a profile depth less than 60 mm, a more relaxed view can be taken, as the studs should normally be at least 95 mm in height (100 mm, if welded direct to the beam), reducing the need for containment. It seems reasonable to provide fixings at 250 mm, as for the deeper profile.

Option 3

The third option open to designers is to provide additional reinforcement in the haunch, in accordance with BS EN 1994-1-1, clause 6.6.5.4.

REVISION a: The minimum stud spacing has been reduced to 150 mm based on additional FEM undertaken in early 2017. Figure 1 has been updated to reflect the new findings, with indicative concrete damage now shown at a slip of 6 mm.

Contact: **Graham Couchman**
Tel: **01344636525**
Email: **advisory@steel-sci.com**

AD 403:
Steel strengths for fabricated haunches

This AD is a simple reminder that the steel strength selected for haunches must match that assumed in the design calculations. As S355 is now the common steel strength for rolled sections, it is highly likely that the calculations for the haunch have also assumed S355 steel – it is important that rolled sections or plate used for the haunch matches the higher grade, unless design calculations have verified a lower strength steel.

In the UK, S275 rolled sections are no longer readily available, with S355 being the common steel strength. For the design of portal frames, the increase in strength is not always beneficial – the opportunity to select smaller sections means that deflections will increase and second order effects (which are calculated based on deflections) will be more significant.

Most haunches are cut from rolled sections, so will normally be the higher grade steel. The potential for a mistake is increased with haunches fabricated from plate. Plate (particularly in the form of flats) is available in S275 steel, so connection designers need to be careful to specify the appropriate steel strength clearly.

Contact: **Richard Henderson**
Tel: **01344636525**
Email: **advisory@steel-sci.com**

AD 404:
Columns in simple construction

SCI has received reports that some designers are disregarding the rules for the design of columns in braced frames (simple construction). In some cases the columns have been designed for an axial load only - even when the loading from the beams is not symmetrical. In another case with a fin plate connection the assumed eccentricity from the face of the column was the actual dimension to the bolt line, rather than the nominal 100 mm.

The rules governing the design of columns in simple construction are given in clause 4.7.7 of BS 5950-1 and – for design to the Eurocodes, NCCI document SN048, available at *www.steel-ncci.co.uk/*.

Whatever style of nominally pinned connection is to be used, the nominal moment is calculated based on an eccentricity from the face of the column of 100 mm, even if the physical dimension to the assumed location of the pin is different. A net moment will result if the beam reactions are different on either axis; the moment is distributed to the column lengths above and below.

The rules for this type of column design, including the apparently arbitrary nominal eccentricity from the column face of 100 mm have reassuring provenance – they were described in BS 449 and had been successfully used for decades. Designers should not depart from these rules without careful consideration.

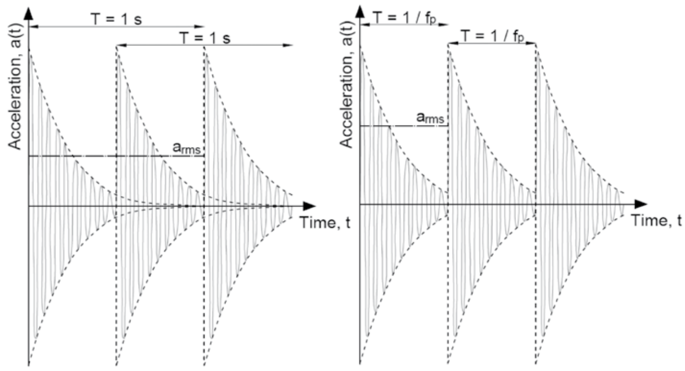
Contact: **Abdul Malik**
Tel: **01344636525**
Email: **advisory@steel-sci.com**

AD 405:
Vibration assessment of transient response factors

This advisory desk note clarifies advice given in SCI P354: Design of floors for vibration: a new approach, regarding the calculation of the transient response factor of a floor system. The transient response factor *R* is given by equation (38) in the publication as the weighted root mean square (rms) acceleration, $a_{w,rms}$, divided by 0.005 ms^{-2}. A generic formula to calculate the weighted rms acceleration is given as equation (12) in section 2.4.1.

$$a_{w,rms} = \sqrt{\frac{1}{T}\int_0^T a_w(t)^2 dt}$$

For the calculation of $a_{w,rms}$, values are needed for the time period under consideration, *T*, and the acceleration function, $a_w(t)$. For transient vibration analysis, the weighted acceleration function, $a_w(t)$, can be found in section 6.3.3, as equation (34). A superposition formula is provided to calculate the acceleration of each impulse by summing the acceleration responses of each mode of vibration of the floor.



In section 2.4.1 and 6.3.3 different values for the time period *T* to be considered are given. In section 2.4.1, it is suggested that a time period of $T = 1$ s should be used, while in section 6.3.3 it is recommended to take $T = 1/f_p$ when calculating the rms acceleration using equation (12). For an average walking pace of $f_p = 2$ Hz that would lead to a time period of $T = 0.5$ s.

Both of these recommendations refer to a single step. The time period $T = 1$ s does not represent two steps, but instead allows for the time that it takes for the acceleration caused by a single step to fade out, which may overlap with other steps. The time period $T = 1/f_p$ represents the time between two steps.

The difference between the two assumptions can be better understood with the figure above.

SCI recommends $T = 1/f_p$ to be used for the calculation of the transient response factors. This ignores the response at the tail end of the step, but this is generally small compared to the initial acceleration caused by the step. As seen in the figure above, using $T = 1/f_p$ leads to a marginally higher rms acceleration, and is therefore on the safe side.

Contact: **Antonia Pilpilidou**
Tel: **01344636525**
Email: **advisory@steel-sci.com**

AD 406: Transient response factors in vibration analysis of staircases

SCI recommends that for most orthodox designs the **transient** response of a staircase should not be considered in design, as first implied by AD330[1]. The purpose of this advisory desk note is to clarify the reasoning behind this advice.

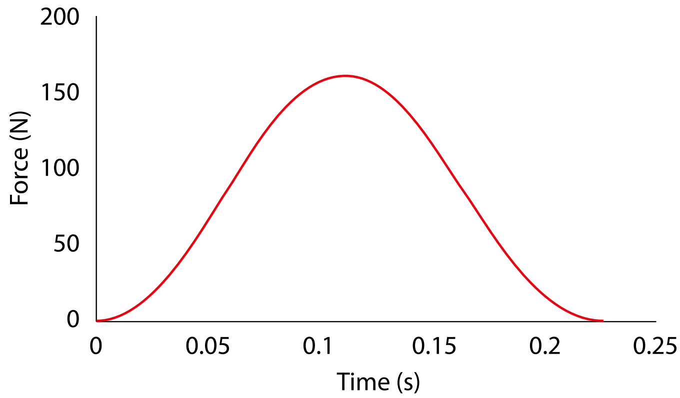
The SCI's key publication on design for vibration is P354[2]. This publication describes an acceleration-based checking methodology (response factor), suitable for both floors and staircases, that supersedes traditional checking of the natural frequency of the structure. The publication describes two checks that must be performed; steady-state and transient analysis. While both checks must be carried out, steady-state response tends to be critical for lower natural frequency structures, while the transient response tends to be critical for structures with higher natural frequency.

Several differences exist between design of staircases and floors for vibration. Staircases tend to have low mass and a low damping ratio. Staircases are also subject to a different force profile, since users tend to travel faster and step harder when ascending or descending a staircase than they would on a flat surface. The force functions provided by Bishop et al.[3] are recommend for use in steady-state analysis.

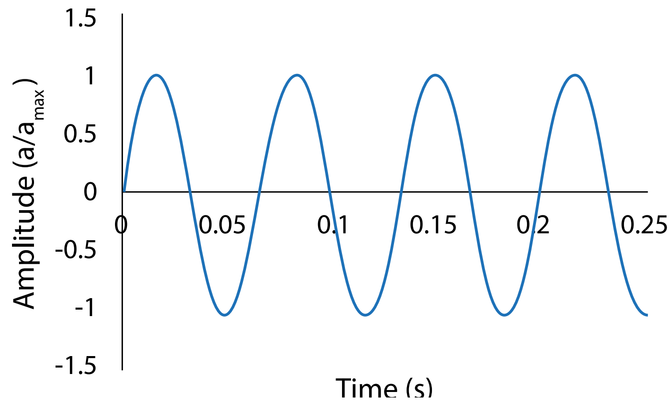
Conversely, the acceptable response factor for a staircase is higher (less onerous) than for a floor, as the audio and visual stimuli that accompany vibration of a floor, such as monitors and shelves shaking, are not present on a staircase. SCI currently recommends limiting response factors of 32 for light use (such as stairs in offices) or 24 for heavy use (such as stairs in public buildings and stadia)[1,2].

Even with the less onerous acceptance criteria, it is very difficult to design a staircase with a low frequency that would pass the steady state criteria. In SCI's experience, staircases with natural frequencies, *f*, less than 15 Hz will struggle to pass. Designers may increase either the mass or stiffness to decrease the response factor, which is usually achieved by increasing member sizes.

The calculation of the transient response assumes instantaneous impulsive loading. For most structures, the response time of the structure is much larger than the contact time of a footstep so this assumption is valid. However, for structures with frequencies over about 15 Hz, this



Force of a footstep (Input)



Response of structure (Output)

assumption begins to break down.

In reality, a footstep delivers most of its energy to the structure over a contact time of about 0.2 seconds[3]. A typical staircase might have a natural frequency of 15 Hz or greater, which gives a natural period of about 0.066 seconds. The response time of the structure is therefore less than the contact time of a footstep.

The figure shows the force function from Bishop et al. for a fast ascent (4.5 Hz) compared to the normalised response of a 15 Hz mode (left and right respectively). The x-axis, showing time, is consistent in both plots. This figure highlights the higher natural frequency of the structure compared to the frequency of the forcing function.

The assumption of instantaneous impulsive loading is therefore invalid in this case. The increased contact time between the person and the structure will result in destructive interference in the oscillation, which the analysis method does not take into account.

For the reasons presented, SCI considers that the transient response prediction is not applicable to typical staircases, and therefore should not be used in design.

References

1. AD 330: Vibration of steel staircases, Steel Construction Institute
2. SCI P354 Design of Floors for Vibration: A New Approach, Revised Edition 2009
3. N.W.M. Bishop, M. Willford, R. Pumphrey, Human induced loading of flexible staircases, Safety Science, Volume 18, Issue 4, February 1995, Pages 261-276, ISSN 0925-7535, http://dx.doi.org/10.1016/0925-7535(94)00035-2.

Contact: **Phil Francis**
Tel: **01344636555**
Email: **advisory@steel-sci.com**

AD 407:
Section classification

SCI have been advised that some checking authorities have questioned the approach to calculating α and ψ, which are found in Table 5.2 of BS EN 1993-1-1:2005 and used when classifying the web of a section under combined bending and compression. The Eurocode is silent on how these two factors should be calculated, which leads to some differences across Europe.

In the UK, SCI provided the following formulae for both α and ψ, in Table 5.1 (page 37) of P362[1]

$$\alpha = \frac{1}{2}\left(1 + \frac{N_{ed}}{f_y c t_w}\right) \text{ and } \psi = \frac{2N_{ed}}{Af_y} - 1$$

These formulae may be found in a number of authoritative sources, including Gardner and Nethercot[2] (page 32). Conceptually, these formulae imagine that the axial force N_{Ed} remains constant, and the moment is increases until f_y is attained (across the section in Class 1 and 2; at the extreme fibres in Class 3). The UK and France follow this approach, as do a number of European guides[3,4].

Some other European authorities follow a different approach, increasing both N_{Ed} and the applied moment in proportion. A different value of α and ψ will result, and potentially, a different Class of section. In some circumstances, if this second approach is followed, the Class becomes more onerous.

These different approaches are discussed in more detail in ECCS publication[3] (Section 2.4 pages 110/111, Section 3.7.2 pages 243/246 and Example 3.17 pages 279/281).

A second issue is that using the above formulae may lead to values of α greater than 1.0. This simply indicates that (in the case of α), all the web is in compression. α should be limited to the range between -1 and 1. ψ will be between 0 and -1.

When the calculated value of α exceeds 1, and thus is limited to 1, the limiting *c/t* ratio for a Class 1 section is given by $\dfrac{396\varepsilon}{13\alpha - 1} = \dfrac{396\varepsilon}{12} = 33\varepsilon$

which is simply the same as the value for "part subject to compression". Similar comparisons may be made with the other Class limits when the calculation of α and ψ indicate that the web is entirely in compression.

References

1. SCI P362 Steel Building Design: Concise Eurocodes (2009)
2. L. Gardner, D.A. Nethercot. Designers' Guide to EN 1993-1-1 (2005)
3. ECCS Eurocode Design Manuals. Design of Steel Structures 2nd Edition. Eurocode 3: Design of Steel Structures. Part 1.1: General rules and rules for buildings (2016)
4. ECCS Eurocode Design Manuals. Fire Design of Steel Structures. Eurocode 1: Actions on structures. Part 1-2: Actions on structures exposed to fire. Eurocode 3: Design of steel structures. Part 1-2: Structural fire design (1st edition, 2010) (Pages 114/115)

Contact: **Abdul Malik**
Tel: **01344636555**
Email: **advisory@steel-sci.com**

AD 408:
Effective length of cantilevers

SCI has recently been contacted regarding the effective length of cantilevers and the effective length factors applied for destabilizing loads which are tabulated in Figure 3.2 of SCI publication P360[1]. The effective length factors were queried when compared with the factors tabulated in Table 14 of BS 5950-1[2]

This AD note demonstrates that the information given in P360 and BS 5950-1 are identical but presented differently.

In P360, a simplified formula for the non-dimensional slenderness of a doubly symmetric I-section beam, taken from NCCI SN002[3] is given as:

$$\bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UVD\bar{\lambda}_z\sqrt{\beta_w}$$

The effective length factor for destabilising load is parameter *D*. The minor axis non-dimensional slenderness $\bar{\lambda}_z = \lambda_z/\lambda_1$ and $\lambda_z = kL/i_z$ where *k* is an effective length parameter applied to the length of the beam *L* which takes different values depending on the restraint conditions. The remaining terms are defined in P360 Section 2.3. The combined effects of support conditions and destabilizing load are therefore allowed for in the product *kD*.

P360 Figure 3.2 repeats guidance given in NCCIs SN009[4] on the effects of common restraint conditions and destabilizing loads for cantilever beams. The restraint conditions identified are identical to those presented in Table 14 of BS 5950-1. This table (without diagrams) is repeated below. The values of the coefficients in the column for normal loading are the same as the corresponding *k* values in P360.

Restraint Conditions		Loading Conditions	
At support	**At tip**	**Normal**	**Destabilizing**
a) Continuous, with lateral restraint to top flange	1) Free	3.0*L*	7.5*L*
	2) Lateral restraint to top flange	2.7*L*	7.5*L*
	3) Torsional restraint	2.4*L*	4.5*L*
	4) Lateral and torsional restraint	2.1*L*	3.6*L*
b) Continuous, with partial torsional restraint	1) Free	2.0*L*	5.0*L*
	2) Lateral restraint to top flange	1.8*L*	5.0*L*
	3) Torsional restraint	1.6*L*	3.0*L*
	4) Lateral and torsional restraint	1.4*L*	2.4*L*
c) Continuous, with lateral and torsional restraint	1) Free	1.0*L*	2.5*L*
	2) Lateral restraint to top flange	0.9*L*	2.5*L*
	3) Torsional restraint	0.8*L*	1.5*L*
	4) Lateral and torsional restraint	0.7*L*	1.2*L*
d) Restrained laterally, torsionally and against rotation on plan	1) Free	0.8*L*	1.4*L*
	2) Lateral restraint to top flange	0.7*L*	1.4*L*
	3) Torsional restraint	0.6*L*	0.6*L*
	4) Lateral and torsional restraint	0.5*L*	0.5*L*

Table 14 Effective length L_E for cantilevers without intermediate restraint

If the values in the last column of the table above (equivalent to *kDL*) are divided by the corresponding values in the third column (equivalent to *kL*), then the destabilising parameter *D* can be derived. The result of this exercise is presented below. An additional column giving the values of *D* from P360 is included in the table for comparison.

Restraint Conditions		Loading Conditions		
At support	**At tip**	**Normal**	**Destabilizing**	***D***
a) Continuous, with lateral restraint to top flange	1) Free	*3.0L*	2.50	2.5
	2) Lateral restraint to top flange	2.7L	2.78	2.8
	3) Torsional restraint	2.4L	1.88	1.9
	4) Lateral and torsional restraint	2.1L	1.71	1.7
b) Continuous, with partial torsional restraint	1) Free	2.0L	2.50	2.5
	2) Lateral restraint to top flange	1.8L	2.78	2.8
	3) Torsional restraint	1.6L	1.88	1.9
	4) Lateral and torsional restraint	1.4L	1.71	1.7
c) Continuous, with lateral and torsional restraint	1) Free	1.0L	2.50	2.5
	2) Lateral restraint to top flange	0.9L	2.78	2.8
	3) Torsional restraint	0.8L	1.88	1.9
	4) Lateral and torsional restraint	0.7L	1.71	1.7
d) Restrained laterally, torsionally and against rotation on plan	1) Free	0.8L	1.75	1.75
	2) Lateral restraint to top flange	0.7L	2.00	2.0
	3) Torsional restraint	0.6L	1.00	1.0
	4) Lateral and torsional restraint	0.5L	1.00	1.0

Effective length factors for cantilevers without intermediate restraint

It can immediately be seen that the effective length factors for destabilising load included in P360 are the BS 5950-1 values rounded to two significant figures except in one case where three significant figures are adopted and the values are identical.

In fact the effective lengths of cantilevers assumed in design to EC3 were adopted from those in BS 5950-1.

References

1 SCI P360 Stability of steel beams and columns (2011)
2 BS 5950-1:2000 Structural use of steel in building – Part 1
3 NCCI SN002 Determination of non-dimensional slenderness of I and H section (2005)
4 NCCI SN009 Effective lengths and destabilizing load parameters for beams and cantilevers – common cases (2005)

Contact: **Richard Henderson**
Tel: **01344636555**
Email: **advisory@steel-sci.com**

AD 409: Recent Blue and Orange Book developments

Recent restructuring within the UK steel industry has led to new developments in the provision of structural steel design information produced by SCI; traditionally known as the 'Blue Book'. This AD note explains these changes.

The first version of the 'Blue Book': *Steelwork Design Guide to BS 5950-1: 2000, Volume 1, Section properties, Member capacities* (SCI publication P202) was published in 1985, based on BS 5950-1. Seven editions were published; the most recent in 2007.

In 2009, the Eurocode version of the Blue Book was published (SCI publication P363). Minor revisions and corrections were made in reprints published in 2011, 2013 and 2015.

Both of these publications (P202 and P363) are still available; hardcopy from the SCI shop *http://shop.steel-sci.com* or electronically to SCI members only, from *www.steelbiz.org*.

The first electronic version of the Blue Books was released by Corus in 2006. Since then, several downloadable versions of the Blue Books have been developed by SCI.

All downloadable versions of the electronic 'Blue Books' are no longer supported by SCI and users are encouraged to use the new, up-to-date, 'Blue Book' and 'Orange Book' websites as described below.

A web-based version of the 'Blue Book', known as the 'Tata Steel interactive Blue Book', was released in 2008. This website was withdrawn in March 2017.

Recent electronic 'Blue and Orange Book' developments

Recent 'Blue Book' user surveys revealed that many users were unaware of the additional functionality offered in the downloadable products. Where they were aware, they generally did not value these features. Furthermore, with increased company IT security, it was becoming problematic for users to download, install and update the software on their company PCs or laptops. The web-based versions also had the added benefit of enabling section ranges to be easily updated without the need for any software updates.

It was therefore decided to develop a new suite of 'Blue and Orange Book' websites. As at June 2017, the following products, developed by SCI, are available:

1. Users of the IHS Construction Information Service (CIS) are able to access a dedicated 'Blue Book' website from *www.ihsti.com/CIS*. Data are provided for the BS4 product ranges and commonly available European section ranges. British section resistance data, to the Eurocodes (UK National Annex), are provided for grades S275, S355 and S460 steel and the European ranges for grades S355 and S460.
2. The Steel for Life 'Interactive Blue Book' website, *www.steelforlifebluebook.co.uk*, is essentially the replacement for the previous Tata Steel interactive Blue Book. The Steel for Life Blue Book provides comprehensive design data; both Eurocode (UK National Annex) and BS 5950-1 for open section ranges to EN 10365:2017 and for structural hollow sections; hot finished to EN 10210-2 and cold formed to EN 10219-2.
3. The new Tata Steel 'Blue Book' website, *www.tatasteelbluebook.com*, replaces the recently withdrawn 'Tata Steel interactive Blue Book' website and provides data on Tata Steel's Celsius and Hybox structural hollow section ranges. Eurocode resistance data are available for S355 and S420 steel sections both in English (UK National Annex) and German (German National Annex). In addition, for the Celsius 355 range, resistance data to BS 5950-1 are provided.
4. The ArcelorMittal Orange Book website, *http://orangebook.arcelormittal.com*, provides member resistance data for S355/HISTAR355 and S460/HISTAR460 steel. Eurocode data (UK National Annex) are provided for the European open section ranges produced by ArcelorMittal and for the British section ranges according to EN 10365:2017.

With the exception of the IHS product, all websites are freely available and allow users to print information directly or export the data to their own computer.

Contact: **Michael Sansom**
Tel: **01344636555**
Email: **advisory@steel-sci.com**

AD 410: Pouring concrete to a constant thickness or to a constant plane

Composite flooring systems comprising concrete and profiled steel decking supported by a grillage of primary and secondary steel members are a popular form of floor construction. The in-situ concrete acts compositely with the steel decking which acts as permanent formwork for the concrete and as external reinforcement to the composite slab. This AD Note is an update to guidance given in AD 344 'Levelling techniques for composite floors' and reflects the most recent practice in pouring concrete to a constant level or thickness. However, the guidance in AD 344 is still valid.

For composite flooring systems the concrete can be poured to a constant thickness or to a constant plane. The type of floor construction is one of the issues that must be determined at the design stage and it is important that this is communicated to the concrete contractor. This AD Note describes the two methods that may be used to pour the concrete (constant plane or constant thickness), the expected surface finish (flatness and levelness) that may be achieved, the construction loads that should be taken in to account during design and the means of communicating the method of concreting to the concreting contractor.

1.0 Design considerations

An important design issue is to decide if the concrete is poured to a constant thickness or to a constant plane as the method of construction will affect the deflections of the steel decking and the steel frame and the amount of concrete placed. The two methods for concreting are:

- Pouring to a constant thickness and,
- Pouring to a constant plane

1.1 Constant thickness

Concreting a floor to a constant thickness can be achieved by using permanent proprietary formed tied construction joints, levelling pins (which are supported by either the steel decking and beams or the steel decking alone) or a depth gauge. The term 'Structural floor level' refers to the case where the screed rails etc. are supported by the steel decking and beams and the term 'Constant depth' refers to the case where the depth gauge or dip method is used. Both of these approaches are described below.

a. Structural floor level. In this approach the reference points defining structural floor level are supported by the steel decking and beams at the design slab depth from the decking profile. The reference points are usually placed as close as possible to the beam centre-lines to avoid excessive displacement during concreting. However, they will drop as the decking and beams deflect as concreting proceeds. The slab thickness will remain as defined by the reference point and deck levels but the finished profile will not be the same as the original position of the reference points. This method should give reasonable control over both the concrete thickness and flatness (but not levelness). This method will result in additional concrete (ponding) at mid-span decking regions as a result of deck deflection between the reference points.
b. A constant depth using a depth gauge. In this approach the reference point is a rod with the constant depth set off the steel decking so that the top profile will be parallel to the decking profile. Good control of thickness should be achieved but the finished surface profile will depend on the initial profile and subsequent deflection of the steel deck and supporting beams. This is typically the **recommended method** and should always be used where the beams are pre-cambered.

1.2 Constant plane

In this method the finished concrete level is determined using a staff and level, often a laser level. As levelling is to a constant reference plane, any deflection of the steel decking and supporting beams as the concreting proceeds can give rise to a considerable increase in the slab thickness and the volume of concrete placed. Additionally, previously levelled areas may drop as the supporting beams continue to deflect as adjacent areas are concreted. The fresh areas of concrete will continue to be levelled to the reference plane therefore small localised variations in level and flatness can occur across the slab pour. It is difficult with this method to achieve good control of level to datum, flatness and thickness. Using this method the slab thickness can be considerably thicker than designed due to the compound deflection of primary beam, secondary beams and steel decking. This depends on the centres and stiffness of the supporting beams.

1.3 Tighter tolerances on level

If tighter tolerances on floor level are required consideration should be given to providing a stiffer grillage of supporting primary and secondary floor beams. This will result in a combination of larger steel sections, short deck spans, more frequent beams and/or columns and possible a heavier gauge steel decking profile. Where strict control of floor level is required it is suggested that the deflection of the steel under construction loads is limited to 10mm. This approach is often considered uneconomic.

Alternatively propped construction may be used to reduce deflections during construction. However, use of propping should be considered at the design stage and not used as an afterthought on site. When a composite slab is propped during construction there is a higher demand on the shear connection between the decking and the concrete than in an unpropped slab, as a propped slab has to support the self-weight of the concrete through composite action. Consequently, a propped slab will have a higher degree of creep deflection under imposed loads than an unpropped slab, as well as the additional deflection of the decking under the self-weight of the concrete. A higher percentage of reinforcement must be specified for propped slabs to limit cracking over the supporting beams, and this clearly needs to be specified at the design stage.

Consideration should be given to deflections after the props are removed.

2.0 Construction loads

Clause 9.3.2(1) of BS EN 1994-1-1 gives recommendations for the actions to be considered during construction when the profiled sheeting is acting as permanent formwork. The following loads should be taken into account:

- Weight of concrete and steel deck,
- Construction loads including local heaping of concrete during construction, in accordance with clause 4.11.1 of BS EN 1991-1-6,
- Storage load, if any,
- 'ponding' effect (increased depth of concrete due to deflection of the sheeting)

Clause 3.2.2 of Technical Report 75 'Composite Concrete Slabs on Steel Decking' by the Concrete Society[4] gives further information on the loads to be considered during concreting.

With regard to 'ponding' clause 9.3.2(2) of BS EN 1994-1-1 gives the following recommends:

'If the central deflection ,δ, of the sheeting under its own weight plus that of the wet concrete, calculated for serviceability, is less than 1/10 of the slab depth, the ponding effect may be ignored in the design of the steel sheeting. If this limit is exceeded, this effect should be allowed for. It may be assumed in design that the nominal thickness of the concrete is increased over the whole span by 0.7δ.'

Pre-cambering of beams is sometimes used to decrease the deflections from construction loads. Where pre-cambering is used, Clause 5.4 of Technical Report 75[4] recommends that the composite floor slab is poured to a constant thickness. Unless the constant thickness method is used there is a risk that there will be insufficient cover to the mid-span of the beams should the camber not fully 'drop out'. Traditionally, engineers have specified a pre-camber of only $^2/_3$ to ¾ of the calculated simply supported deflection of the beam, or up to half the concrete cover to the decking (whichever is less). Doing this will greatly reduce the risk of a thin slab when the other methods of concreting are used.

3.0 Flatness and level tolerances

The main consideration with regards to the specification of tolerances is the building's use; buildings where the finished slab is to provide a wearing surface may require tight level and flatness tolerances, whereas buildings where subsequent finishes are applied such as office structures may not. The requirements in the specification need to be achievable: it is not possible to construct a composite slab to very tight level tolerance because of the deflections of the beams. However, tight tolerances are not necessary for most applications, and deviations can be taken up with screeds, levelling compounds or a raised floor. Where isolated areas in a building have more onerous flatness requirements, they can be achieved by using levelling compounds or screeds locally. Extensive grinding should not be used to modify flatness, as it can significantly reduce the slab thickness.

For the rare occasions where levelling compounds and screeds cannot be used, and tight level and flatness tolerances are required, the supporting beams will need to be designed to limit deflections to values which correlate with the required top surface tolerances. This could have significant implications for the cost of the beams.

The following general tolerances for levels are given in references 1, 2 and 3, relative to the level of the datum (normally structural floor level):

- ±15mm on top surface of concrete measured at a column
- ±10mm on top surface of supporting steel beams at a column position

The slab thickness tolerances at a column position will be about ±20mm using the above values. Further information on level and flatness tolerances can be found in reference 4.

4.0 Information required for the casting of the concrete

Where projects are working to the Construction (Design and Management) Regulations 2015 it is the responsibility of the Principal

Designer to make sure the right information is given to those that need it prior to concrete pouring work commencing. If the Construction (Design and Management) Regulations 2015 do not apply to the project then the Main Contractor will need to ensure those contracted to carry out the concrete pouring are provided with this information.

To avoid overloading the decking and supporting steelwork during the construction phase it is recommended that the method of concreting (constant level or constant thickness) is communicated to the concreting contractor on the scheme designer's construction drawings.
Whilst acceptable flatness (surface regularity over short distance) can be achieved, a level slab (level to a defined datum over large distances) can only be achieved where this has been considered early in the design of the steel frame with beam spacings, deck spans and deflections considered accordingly. Extensive guidance on this subject can be found in reference 4

5.0 References

1. British Standards Institute, BS EN 13670. Execution of concrete structures, BSI, London 2011.
2. British Standards Institute, BS 8204-2. Screeds, bases and in situ floorings. Part 2: Concrete wearing surfaces. Code of practice, BSI, London, 2003.
3. The Concrete Society. Concrete industrial ground floors. A guide to design and construction, Technical Report 34, 4th edition, 2013 (revised March 2016).
4. The Concrete Society. Composite concrete slabs on steel decking. Guidance on construction and associated design considerations, Technical Report 75, 2016

Contact: **David Moore**
Tel: **020 7747 8122**
Email: **david.moore@steelconstruction.org**

AD 411: Design of web to flange welds in plate girders

The SCI Advisory Desk is frequently asked how to design the welds between the flange and web of a plate girder. The following note discusses the standard formula for the shear flow between web and flanges of a doubly symmetric beam which is used for weld design and gives the background to the formula in Eurocode 3 Part 5[1]. An example is also presented.

The design of a plate girder element is the responsibility of the building structural engineer just as is the design of a rolled section beam. The difference is that plate girder design involves choosing explicitly the width and depth of the beam and also the thicknesses and arrangement of the constituent plates, including the connection between them. The web to flange welds are not connections between elements so in the contractual arrangement usually adopted on projects, their design is not in the steelwork contractor's scope of work.

The relevant stresses in the beam which are carried by the web to flange welds are the shear stresses which act on planes parallel to the longitudinal axis of the element and are the result of the change in bending moment over an incremental length of the beam. Shear stresses which are equal and perpendicular to the longitudinal stresses are developed in the plane of the cross section and are termed "complementary" shear stresses. The sum of these stresses over the area of the cross section equals the applied shear force. The stresses are determined using the standard formula for calculating the shear stress distribution over the cross section which is found in strength of materials text books:

$$\tau = \frac{V_{Ed} A \bar{z}}{I_z b} \qquad \text{equation 1}$$

where:
τ is the shear stress at a point in the cross section a distance z from the neutral axis of the section;
V_{Ed} is the design shear force on the section;
A is the area of the cross section further from the neutral axis than z;
$\bar{z}$ is the distance from the neutral axis to the centroid of area A;
I_z is the second moment of area of the whole cross section;
b is the width of the section at the point considered.

Applying the formula to a rectangular cross section with the long dimension vertical carrying a vertical shear force produces a parabolic distribution of shear stress over the section which is a maximum at the neutral axis and zero at the top and bottom. When applied to an I section it produces the familiar distribution showing that most of the shear force is carried by the web of the beam.

When considering weld design, equation 1 can be written in terms of shear flow s between the flange and web by substituting $s = \tau b$ as shown:

$$s = \frac{V_{Ed} A_f \bar{z}}{I_z} \qquad \text{equation 2}$$

where A_f is the area of the flange. The shear flow is the shear force per unit length which is to be carried by the weld.

Part 5 of Eurocode 3 gives conservative and simplified formulae for sizing web to flange welds in clause 9.3.5(1) as follows:

$$s = \frac{V_{Ed}}{h_w} \text{ if } V_{Ed} \le \chi_w f_{yw} h_w t / \sqrt{3}\gamma_{M1} \qquad \text{equation 3}$$

where h_w is the depth of the web. For larger values of V_{Ed} the weld should be designed for

$$s = \eta f_{yw} t / \sqrt{3}\gamma_{M1}$$

Equation 3 is used if the shear force on the web is less than the shear buckling resistance of the web which is given by the expression on the RHS of the inequality. Clause 5.1(2), gives a value of slenderness for an unstiffened web where shear buckling does not arise:

$$\frac{h_w}{t} < \frac{72}{\eta}\varepsilon \text{ where } \varepsilon = \sqrt{\frac{235}{f_y}}$$

Tests have shown that the shear resistance of a stocky web exceeds the resistance predicted by the Von Mises yield criterion due to strain hardening. This effect is allowed for by including the factor η, the value of which is subject to national choice. According to the UK National Annex, η should be taken as equal to 1.0, ie the effect of strain hardening is ignored.

The simple formula for shear flow in equation 3 can be shown to be a conservative approximation if the second moment of area of the plate girder is based on the second moment of the flanges with respect to the neutral axis (ie neglecting the web and the second moments of the flanges about their own centre-line). The $A_f\bar{z}$ term is the first moment of the flange about the neutral axis of the beam. Substituting these values in equation 2 gives:

$$s = \frac{V_{Ed} A_f (h_w + t_f)/2}{A_f (h_w + t_f)^2/2} = \frac{V_{Ed}}{(h_w + t_f)} \approx \frac{V_{Ed}}{h_w} \qquad \text{equation 4}$$

Neglecting the thickness of the flange in calculating the shear flow is clearly conservative.

Example

A 10m span plate girder 600 mm deep by 300 mm wide with 30 mm thick flanges and a 10 mm thick web (steel grade S355) carries a central point load of 800 kN. The top flange of the beam is fully restrained. Size the web to flange welds.

$I_z = 1/12(600^3 \times 300 - 540^3 \times 290) = 1.60 \times 10^9$ mm^4.
$W_p = 30 \times 300 \times 285 \times 2 + 270 \times 10 \times (270/2) \times 2$
$= 5.86 \times 10^6$ mm^3.
$M_R = 345 \times 5.86 \times 106/109 = 2.02$ MNm.
$M_{Ed} = 800 \times 10/4 = 2.0$ MNm
ie the beam is sized for bending.

Eurocode 3 Part 5:
Web slenderness: $h_w/t = 540/10 = 54$. The limiting slenderness is $72\varepsilon = 58.6$ so the web is not slender and shear buckling does not arise ie $\chi_w = 1.0$.

The limiting value of design shear force:
$V_{Ed} = 1.0 \times 355 \times 40 \times 10 / \sqrt{3} \times 1.0 = 1106 > 400$kN

The simple formula can be used:
$s = 400 / 540 = 0.74$ kN/mm. For two welds, this is 0.37 kN/mm per weld. The 6mm leg fillet weld length required (longitudinal resistance, 1.01 kN/mm) over 200 mm = (200 × 0.37)/1.01 = 73 mm. Adding twice the leg length for stops and starts gives 85 mm: use 90 mm. Provide an intermittent 6mm fillet weld on both sides of the web, 90 mm hit and 110 mm miss. The average shear resistance per mm is (90–12)/200 × 1.01 = 0.39 > 0.37 kN/mm – OK.

Apply the standard formula:
$s = 400 \times 9000 \times 285 / 1.6 \times 109 = 0.64$ kN/mm. For two welds this is 0.32 kN/mm per weld. The 6mm fillet weld leg length required over 200 mm = 200 × 0.32/1.01 = 63 mm. Adding twice the leg length for stops and starts gives 75 mm: use 80 mm. Provide an intermittent 6 mm fillet weld on both sides of the web 80 mm hit and 120 mm miss. The average shear resistance per mm is (80 – 2)/200 × 1.01 = 0.34 > 0.32 kN/mm – OK.

The simple formula in Eurocode 3 is more conservative.

The size of the smallest continuous fillet weld which is just sufficient to transfer the web to flange shear flow may be impractically small (a 3.0 mm leg fillet weld has a longitudinal shear resistance of 0.51 kN/mm). A larger intermittent fillet weld can be used, as in this example, but is not suitable for elements where corrosion is an issue because the web to flange joint is unsealed where there is no weld. In practice, a steelwork contractor may choose to provide a continuous fillet weld to avoid having to set out all the stops and starts. The works may also have a standard weld procedure for the relevant plate thicknesses with a pre-determined size of fillet weld which is larger than the calculated value.

Contact: **Richard Henderson**
Tel: **01344 636555**
Email: **advisory@steelconstruction.org**

1. BS EN 1993-1-5:2006 Eurocode 3 – Design of steel structures – Part 1-5 design of plated elements

AD 412: Issues related to coatings and availability of structural fastenings

Recently, SCI has received some queries related to the coatings and availability of structural fasteners. This advisory desk note addresses some of these issues.

It is probably useful to clarify the terminology applicable to zinc-coated bolts, nuts and washers. There are currently three common standardized types of zinc coating in use in the construction industry – galvanized, sherardized and electro-plated.

Galvanized, used correctly, means 'hot-dip galvanized' – a process of dipping in molten zinc. In the case of bolts and nuts, after dipping they are normally spun in a centrifuge while the coating is still fluid to clear the threads of excess zinc. This is described as 'spun galvanized'.

Sherardized means zinc-coated by a special process involving heat and zinc dust, normally carried out in a rotating drum.

Electro-plated, as the name implies, means coated by a process of electrolysis, which involves immersion in an acid.

Regrettably, 'galvanized' is sometimes used more loosely, either to mean any zinc-coated fastener, or any except electro-plated. This is confusing and should be avoided.

Q1. There appears to be a shortage of sherardized bolts in the market, do you know why this is? And what is the recommendation to use as an alternative? Electro-plated or hot-dip galvanized bolts?

A1. The shortage of sherardized bolts is generally a reflection of market demand with the hot-dip galvanized and electro-plated finishes dominating the structural bolting market with rough approximation of a 70 % hot-dip galvanized / 30% zinc-plated. Another factor is that the vast majority of non-preloaded bolting assemblies are imported and since sherardizing is not generally available in the manufacturing markets, sherardized structural bolts are more expensive

Q2. Bolt galling/lock up, does this become more of a problem with hot-dip galvanized bolts?

A2. Galling is not a problem with non-preloaded structural bolting assemblies whether hot-dip galvanized or zinc-electroplated. In the distant past there could be problems with hot-dip galvanized bolting assemblies because of excessively thick, uneven or rough coatings. However, these problems do not affect the current hot-dip galvanized structural bolting assemblies available in the UK market.

Q3. My understanding is a lubricant should be applied to prevent this, but should you only really need to use a lubricant in pre-loaded bolt assemblies? Or in all cases?

A3. For pre-loaded assemblies, it is a requirement of the European standards (EN 14399 series) that bolting assemblies are supplied with suitable lubrication to ensure satisfactory installation. However, it is essential that these assemblies are stored in suitable dry and well ventilated storage conditions to ensure there is no deterioration of the lubricant on site prior to installation. Provided the storage conditions are suitable no additional lubrication should be necessary.

In preparation of this AD note, SCI acknowledge assistance provided by Mark Tiddy of Cooper & Turner (bolt manufacturers).

Contact: **Abdul Malik**
Tel: **01344 636555**
Email: **advisory@steel-sci.com**

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