TECHNICAL DIGEST 2023

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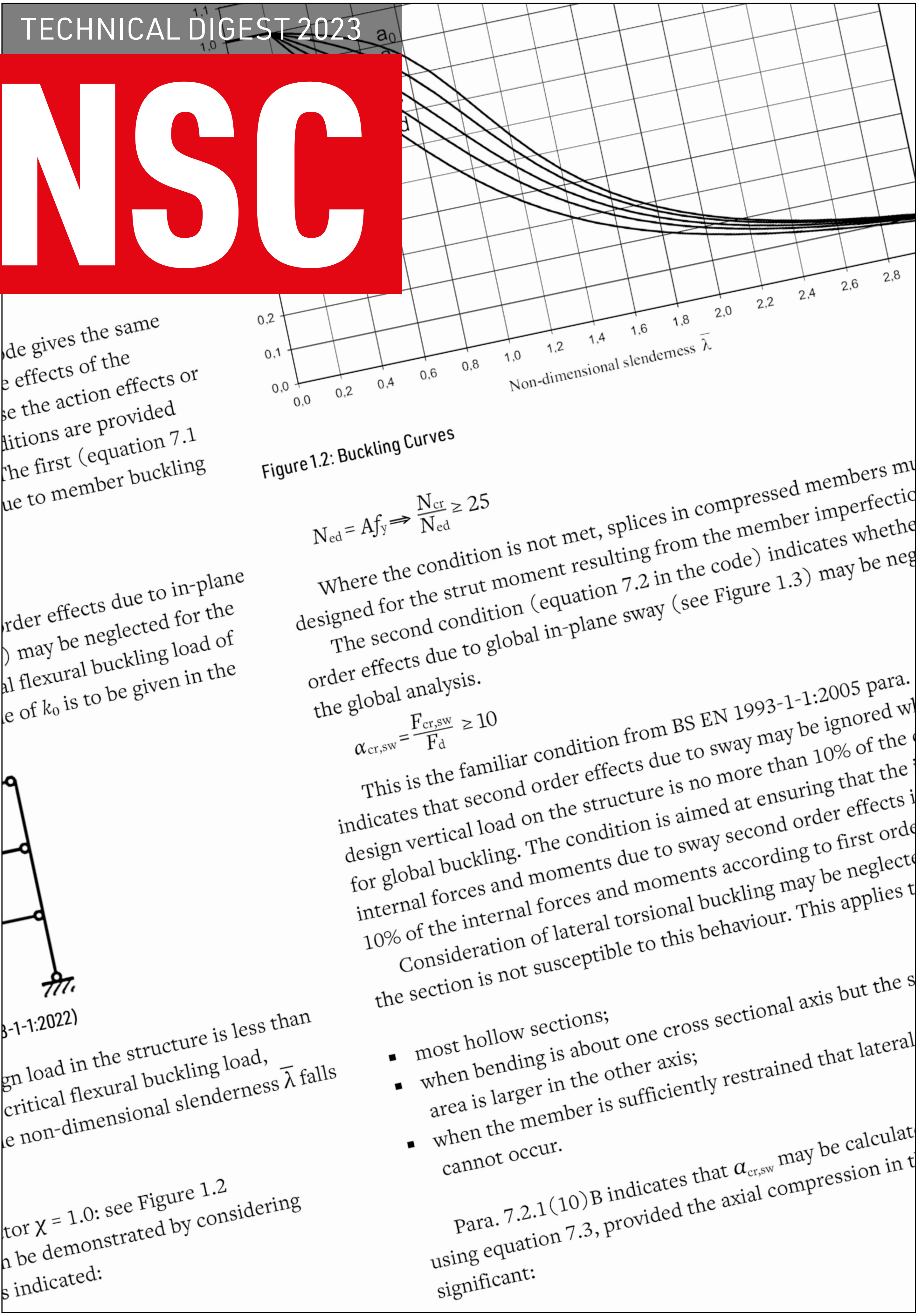

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tor χ = 1.0: see Figure 1.2
n be demonstrated by considering
s indicated:

Figure 1.2: Buckling Curves

$$N_{ed} = Af_y \Rightarrow \frac{N_{cr}}{N_{ed}} \geq 25$$

Where the condition is not met, splices in compressed members mu
designed for the strut moment resulting from the member imperfectio
The second condition (equation 7.2 in the code) indicates whethe
order effects due to global in-plane sway (see Figure 1.3) may be neg
the global analysis.

$$\alpha_{cr,sw} = \frac{F_{cr,sw}}{F_d} \geq 10$$

This is the familiar condition from BS EN 1993-1-1:2005 para.
indicates that second order effects due to sway may be ignored wh
design vertical load on the structure is no more than 10% of the
for global buckling. The condition is aimed at ensuring that the
internal forces and moments due to sway second order effects i
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Consideration of lateral torsional buckling may be neglecte
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- most hollow sections;
- when bending is about one cross sectional axis but the s
  area is larger in the other axis;
- when the member is sufficiently restrained that lateral
  cannot occur.

Para. 7.2.1(10)B indicates that $\alpha_{cr,sw}$ may be calculat
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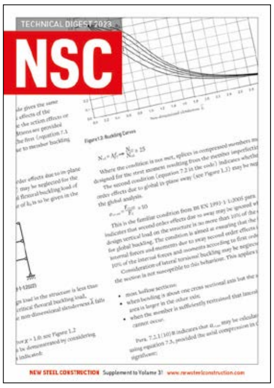

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Essential reading for carbon conscious designers

Nick Barrett - Editor

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

Launched after requests from readers that the technical content of *NSC* be brought together in an easily accessible format, the Technical Digest has an established place on the essential reading section of the digital 'bookshelves' of architects and engineers. The Digest brings together all the Advisory Desk Notes and Technical Articles published in *NSC* in the previous year in a format that is available as a free downloadable pdf at the *steelconstruction.info* website, or for online viewing.

The Digest is part of the steel construction sector's long-established commitment to keep designers in steel up-to-date with the latest technical guidance to ensure that they can take advantage of the numerous benefits of steel as a sustainable construction material, which is more important than ever as the construction industry enthusiastically adopts the need for change to support the drive to net zero carbon.

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

NSC is a popular source of advice and news, and is where the highly popular Advisory Desk Notes and longer Technical Articles from the steel sector's own experts - that are included in the Technical Digest - are first published. They are immediately made available on *newsteelconstruction.com*.

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

The more detailed Technical Articles offer deeper insights into what designers need to know to deliver the most efficient and sustainable steel construction projects. Technical Articles can be in response to legislative changes or changes to codes and standards. Technical updates will occasionally be provided following a number of relatively minor changes that it is felt could usefully be brought together in one place.

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

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Use of fibre-reinforced concrete in composite slabs (part 1)

Recent years have seen increasing interest in replacing the reinforcement mesh with steel fibres in composite slabs on steel decking. In this first of two articles, Constantinos Kyprianou, Principal Engineer at the Steel Construction Institute, provides an overview of the characteristics of the different types of fibres that can be mixed in concrete. Most of the information provided here has been gathered from publications by the Concrete Society [1].

Introduction

The use of fibres within the concrete mix, and in particular steel fibres, is not a new technology. The concept has been in existence for many years, while the first patent was granted in 1874[1]. Despite this, commercial use only took off in the 1970s, mainly in Europe, USA and Japan. Although fibre manufacturers have been producing guidelines for the design of pile and ground-supported floors over a number of years, an agreed design approach for steel-fibre-reinforced-concrete (SFRC) was published for the first time in 2003 by RILEM[2]. This design method[2] provides a methodology for determining the material properties of SFRC, a design approach for bending and shear at ultimate limit states and crack control rules for the serviceability limit states. This method was adopted and published in 2007 as guidance in TR34[3] and TR63[4] by the Concrete Society. Design of steel-fibre-reinforced concrete is not currently covered by design standards including the Eurocodes, where the only reinforcement considered is conventional steel bars or mesh.

According to the Concrete Society[1], in the UK, several millions of square metres of steel-fibre reinforced slabs for ground-supported and pile-supported applications have been constructed over the past decade. Precast elements and suspended composite slabs on steel decking are some of the other potential applications for fibre-reinforced concrete. For composite slabs with steel fibres the same methodology as in [2,3,4] for determining the material properties of SFRC is followed, while many of the concepts in the design approach are also adopted. However, some distinctions in design philosophy exist. For example, in the design of composite slabs at normal ultimate limit state the presence of fibres is typically (and conservatively) ignored. For the accidental fire limit state, the presence of fibres is, however, recognised and considered in enhancing the bending capacity of the slab at elevated temperatures. As such, fibres can play a critical role in the behaviour of a composite slab since not only do they control cracking but also provide an alternative to mesh reinforcement in the fire situation. Before jumping into more detailed technicalities of composite slab design with fibres, which will be reported in the second article, the different types of fibre and their associated characteristics should be defined.

Types of fibre

Three main types of fibre may be used in a concrete mix: steel fibres, macro-synthetic and micro-synthetic polymer fibres. In Figure 1 typical forms for each of these three main types of fibre are illustrated with photos of similar scale to allow comparison in shape and size. In accordance with BS EN 14889[5], steel fibres and macro-synthetic fibres provide post-cracking or residual moment capacity to concrete and as such can be used in the design of elements under flexure, whereas micro-synthetic fibres do not provide any post-cracking ductility and as such cannot be considered in structural design.

The required fibre dosage, specified in kg/m³, will vary depending on the type of application, concrete mix design and the performance requirements of each particular project. The main characteristics of the three types of fibre are described in the following sections.

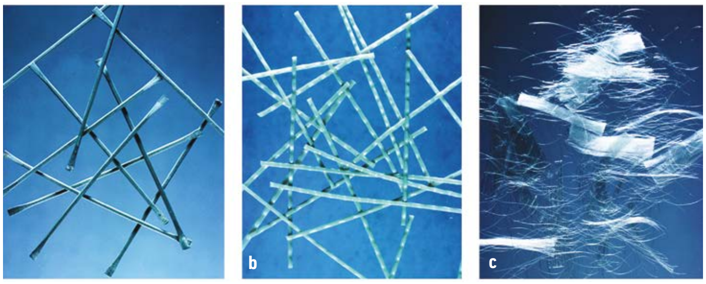

Figure 1: Various types of fibres: (a) steel fibres, (b) macro-synthetic fibres and (c) micro-synthetic fibres. Images from [1]

Steel fibres

For composite slab applications, steel fibre dosage will typically be in the range of 20 kg to 40 kg/m³.

As per BS EN 14889[5], steel fibres can be classified into five groups, according to the method of manufacture, as follows:

• Group I:	Cold-drawn wire
• Group II:	Cut sheet
• Group III:	Melt extracted
• Group IV:	Shaved cold drawn wire
• Group V:	Milled from blocks

Other common methods of characterisation, which describe shape, geometry and strength are:

• Cross-section:	Round, flat, crescent, etc.
• Deformations:	Straight, wavy, hook-end, double hook-end, etc.
• Length:	19 – 60 mm
• Aspect Ratio (length/diameter):	30 – 100
• Young's modulus:	205 kN/mm²
• Tensile strength:	345 – 1700 N/mm²

Currently, fibre manufacturers produce steel fibres with diameters ranging between 0.5 mm and 1.0 mm and the most common lengths are between 35 mm and 60 mm; typical examples are shown in Figure 1 (a). Nowadays, the usual tensile strength range is between 1100 N/mm² and 1500 N/mm², but some examples of high-performance fibres report strengths of up to 2300 N/mm². Some of their physical characteristics directly affect key aspects of performance, such as the residual flexural tensile strength of steel-fibre-reinforced-concrete, while others are less important. A requirement set by BS EN 14889-Part 1[5] is for the supplier to declare the respective dosage in kg/m³ which achieves a residual flexural strength of at least 1.5 MPa at CMOD (crack mouth opening displacement) 0.5 mm and a residual flexural strength of at least 1 MPa at CMOD 3.0 mm, when tested in accordance with the standard notched beam test. Further information on the use of steel fibres can be found in TR34[3] and TR63[4].

Macro-synthetic fibres

Macro-synthetic fibres, illustrated in Figure 1 (b), are usually made from a blend of polymers and were originally developed to provide an alternative to steel fibres in some applications. They generally have a reasonable tensile strength (about 600 N/mm²) and a relatively high modulus of elasticity (about 10 kN/mm²), but when compared with the most modern high strength steel fibres they are lacking.

The properties of the fibres are covered by BS EN 14889-Part 2[5] and they have the same requirements for demonstrating residual strength as steel fibres. In addition, the manufacturer should demonstrate that the fibres are unaffected by the alkalis in the cement paste, and are resistant to moisture and to the substances present in air-entraining and chemical admixtures. They must also be resistant to chlorides when used in marine structures or those subjected to de-icing salts. Also, the effects of long-term creep of macro-synthetic fibres are thought to be significant and need to be considered. Further information on the use of macro-synthetic fibres may be found in TR65[6].

Micro-synthetic fibres

Micro-synthetic fibres are various types of short, thin and chopped polypropylene fibres, as shown in Figure 1 (c). Typically, they may be added to concrete at a rate of about 0.9 kg/m³ and they can be used along with steel-fibres. Their primary role is to modify the properties of fresh concrete. They increase the homogeneity of the mix, stabilising the movement of solid particles and blocking bleed water channels. This reduces the bleed capacity of the concrete and slows down the bleed rate, helping to reduce plastic settlement. Polypropylene fibres have a limited effect on the material properties of the hardened concrete and as such are not considered in design. They have been shown to reduce the spalling of concrete in a fire. One note of caution is that their use can reduce the slump of concrete as they act as a thickening agent.

Material properties of fibre-reinforced concrete

The effect of the fibres on the strength of the concrete is determined in accordance with BS EN 14845[7] using a standard notched beam test described in BS EN 14651[8]. Specimens 150 mm-wide × 150 mm-deep are tested under 3-point loading over a span of 500 mm. The specimens are notched with a 25-mm deep cut across their width at mid-length, and then tested with the notch in the tension face, as shown in Figure 2. A test set consists of at least 12 nominally identical samples, i.e. same dosage and compressive strength of concrete.

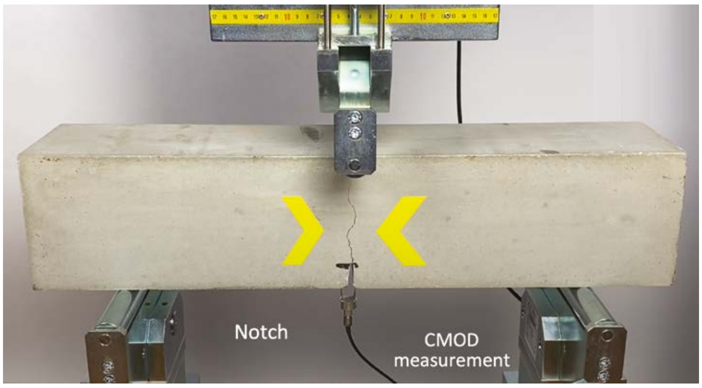

Figure 2: Notched beam tests in accordance with BS EN 14651. Images from [9]

The crack mouth opening displacement (CMOD) (i.e. the increase in width of the notch) and the load F are recorded at CMODs of 0.5, 1.5, 2.5 and 3.5 mm. A typical graph of applied load F_R against CMOD is shown in Figure 3.

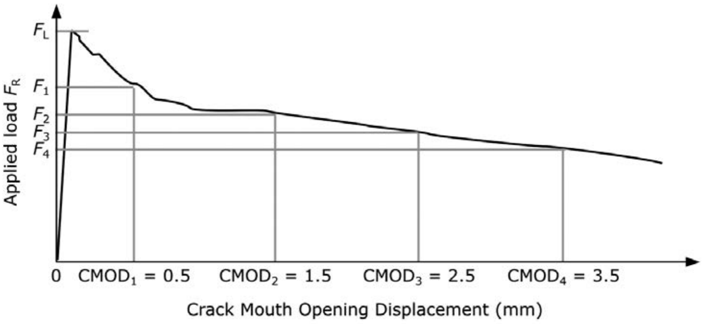

Figure 3: Typical load - CMOD curve for a beam notched test. Graph adapted from [3]

Maximum applied load (F_L) is achieved at the point just before the concrete beam cracks. Typically, the capacity of the section reduces as strain and crack width increases, but certain combinations of fibre type and dosage can exhibit strain hardening behaviour. Strain hardening is identified in a notched beam test when F_1 is equal to or greater than F_L and F_4 is greater than F_1.

Residual flexural strength f_R in N/mm² is derived using the following equation[2,3]:

$$f_R = \frac{3F_R l}{2bh_{sp}^2}$$

where:

F_R is the applied load at the respective CMOD stage,
i.e. for $CMOD_1$ this is F_1,
l is the span of 500 mm,
b is the width of 150 mm and
h_{sp} is the depth of the section at the notch, i.e. 150 − 25 = 125 mm.

The residual strengths f_{R1}, f_{R2}, f_{R3} and f_{R4} for each CMOD are reported as the mean values from all 12 tests of a set with the same dosage. Tests should cover the whole range of fibre dosages to be used. Interpolation between results for different dosages can be made, but not extrapolation. Also, the range between test results when interpolation is made should not be greater than 10 kg/m³[3].

Commentary

This article has reviewed the characteristics of the main type of fibres and the established test method to determine the material properties of fibre-reinforced concrete. In the second part of this two-part article, design and construction considerations will be explored. ■

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Use of fibre-reinforced concrete in composite slabs (part 2)

Recent years have seen increasing interest in replacing the reinforcement mesh with steel fibres in composite slabs on steel decking. The drivers are largely economic, while the reduction of construction material suggests decreases in the carbon footprint may be possible. Along with reduced waste on site and less labour demand, the use of fibres makes a case for being more environmentally friendly when compared to using steel mesh reinforcement. Nevertheless, certain technical and practical issues should be considered when wishing to use fibres in concrete. In this second of two articles, Constantinos Kyprianou of the Steel Construction Institute reports on the design and construction considerations on the use of steel fibres mixed in concrete instead of conventional mesh reinforcement for a composite slab on steel decking.

Design of composite slabs with steel fibres

By testing

A common approach is design assisted by testing. The basic premise is that the performance of one metre wide composite slabs is representative since composite slabs on steel decking are always one-way spanning slabs. Such an approach has been adopted for almost 20 years ago by the Steel Construction Institute (SCI) for specified combinations of steel fibres and decking, and has been used principally to complement design with BS 5950.

By adapting conventional reinforced concrete design and using a mechanics-based approach

A simple approach adopted by some fibre suppliers is to firstly carry out a conventional design and then replace the area of mesh reinforcement with an equivalent (in terms of tensile capacity) amount of fibre while keeping the slab thickness unchanged. The required dosage of fibre is determined using standard bending theory such that the fibre-reinforced cross-section has the same post-cracking moment resistance as the mesh-reinforced cross-section. Similarly, equivalent tensile strength is used to determine the dosage needed to satisfy the minimum reinforcement requirements of the standards for crack control and longitudinal shear resistance for composite beams.

Flexural capacity can also be calculated using a plastic design approach, as described in TR34[1]. This approach is adopted by SCI to determine the moment resistance in fire when designing with the Eurocodes. This new method allows for more flexibility in design, and acts as an evolution of the approach previously developed by SCI for use with BS 5950. It should be noted that although in principle the methodology is simple, its implementation necessitates significant computation because of the iterations involved to calculate the required stress blocks. Therefore, it is almost always necessary to implement this method within software.

For the axial tensile strength of a flexural member the strength at CMOD 0.5 mm and 3.5 mm are considered, these are σ_{r1} and σ_{r4} respectively, and in accordance with TR34[1] are taken as:

$\sigma_{r1} = 0.45 f_{R1}$

$\sigma_{r4} = 0.37 f_{R4}$

where, f_{R1} and f_{R4} are defined in the first article.

These strengths are used to determine the cross-section resistance.

Fire design

Composite slabs at the normal stage are typically designed as single spanning, even when the deck is continuous over supports and designed as such at the construction stage. For typical UK practice, with no bars in the troughs, this means that the only reinforcement considered in design is that provided by the decking, which enhances the sagging moment resistance. Any hogging resistance provided by reinforcement, or fibres, is ignored. It is only during the accidental fire situation that the presence of mesh, or fibres, is recognised. SCI has produced a method using a plastic design approach of stress blocks to allow the tensile resistance of steel fibre-reinforced concrete (SFRC), deck and potentially bars in troughs to be considered in the determination of hogging and sagging resistances at elevated temperature. The methodology for considering the effects of SFRC is similar to the one described in TR34. Figure 1 shows the stress blocks to be considered for a typical flexural element with SFRC only, of which σ_{r1} and σ_{r4} are the tensile forces of SFRC.

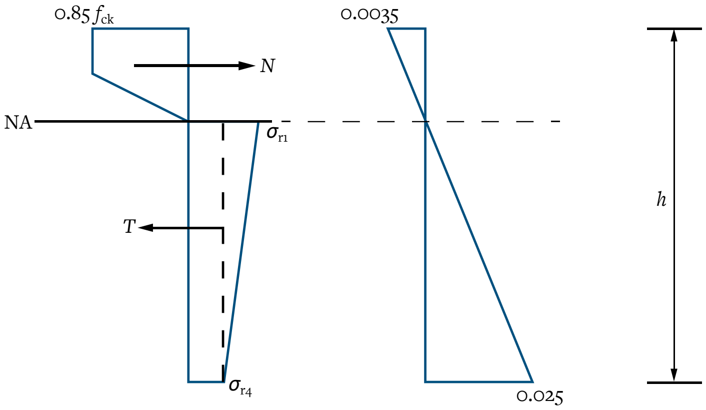

Figure 1: Typical stress block and strains of a flexural member with fibre-reinforced concrete. Adapted from [1]

For a fire scenario, a model for calculating concrete, deck and reinforcement temperatures is described in NCCI PN005c[2]. Based on these elevated temperatures, reduction factors are applied to the compressive strength of concrete and tensile strengths of SRFC, deck and bars in troughs (if present). Reduction factors are taken from the relevant Eurocodes, while for SFRC suitable factors have been determined by SCI based on the codified properties for concrete and reinforcement.

It should be noted that, as with conventionally reinforced slabs in fire, when the slab is physically single spanning (i.e. no end continuity), then a bar in trough is always required. When the slab is physically continuous over a support, this continuity is recognised in design and a semi-empirical rotation capacity check over the supports is performed to ensure that the continuity can be relied upon. A lack of rotation capacity could result in premature fracture of tensile components.

Considerations at the normal stage, ultimate limit state

Bending capacity

Typical composite slab design, at ambient temperature, ignores any tensile contribution from SFRC. The calculated shear bond parameters, τ_u and m & k, which are determined from full scale tests are assumed to be unaffected by the presence of fibres. The deck is considered to act compositely in (typically) partial interaction with the concrete, as external reinforcement.

Vertical shear

Currently in EN 1994[3], the vertical shear resistance of a composite slab only considers the role of the concrete, even though this appears conservative in ignoring any enhancement from the deck. The presence of steel fibres in concrete enhances its shear capacity; this is described in detail in the RILEM report[4] and TR34[1]. In current design this effect is conservatively ignored.

Punching shear

A punching shear resistance check is needed in the presence of a concentrated load. Punching shear resistance is partly provided by two cross-sections of the slab that run parallel to the slab span – the tensile reinforcement for these sections is provided by the decking (not mesh), as for vertical shear. Additional resistance is provided by two cross-sections that run perpendicular to the span of the slab – the tensile reinforcement for these is traditionally provided by mesh but can be provided by fibres.

Longitudinal shear – beam check

Satisfying the longitudinal shear resistance, which is a composite beam check even though the reinforcement is associated with the slab, requires a minimum area of transverse reinforcement to be determined in accordance with EN 1992-1-1 Clause 9.2.2 (5)[5]. A strut and tie model is used to determine the minimum area of reinforcement. This may be converted to an equivalent tensile force and compared to a resistance using SFRC.

Considerations at the normal stage, serviceability state

In accordance with EN 1994-1-1 clause 9.8.1 (2)[3], for continuous slabs that are nevertheless designed as simply supported the cross-sectional area of the anti-crack reinforcement above the ribs of the deck should be not less than 0.2% of the cross-sectional area of the concrete above the ribs for un-propped construction, and 0.4% of this cross-sectional area for propped construction.

For fibre design the required area of mesh reinforcement is converted to an equivalent tensile force, which is then used to determine the appropriate fibre dosage.

Construction and design considerations

Advantages

The main benefits of replacing mesh reinforcement with steel fibres are:

- Improvement in impact resistance and fatigue endurance.
- Improved durability of slab as a result of reduced cracking.
- Test evidence of SFRC enhancing the fire resistance of composite slabs [6,7].
- Reduction in construction time since it removes a trade.
- Reduction of site waste with unused mesh reinforcement.
- No issues associated with displacement of conventional mesh within the depth of the slab and clashing with the studs.

Savings in the cost of supplying and fixing conventional mesh reinforcement can offset the extra cost of adding fibres to the concrete. There may also be health and safety benefits resulting from the reduced handling of reinforcement.

Issues

Some of the issues that could arise with the use of steel fibres are:

- No standing platform for labour (which mesh offers) when pouring concrete, so care should be taken by workers not to damage the steel deck during pouring. Designers are advised to consider an extra construction-imposed load allowance to cover the direct contact with the deck.
- If the steel fibres are not already in a ready mixed concrete when delivered to site, care should be taken to ensure their effective mixing and even distribution during pouring. It can be difficult to control and check the even distribution of fibres. Although offsite mixing is associated with high quality control, contractors and suppliers should ensure that every effort is made for checking the quality of the pour and the even distribution of fibres in the concrete mix during construction of the slab.
- Admixtures such as superplasticisers, water reducers and hardeners may be needed in the concrete to aid with the even distribution of fibres within the concrete mix, and its fluidity, since the presence of fibres can act as a thickening agent.
- The finished surface will be rough because of protruding fibres, and will need additional screed if a smooth surface is needed.
- At the decommissioning phase of slabs, it can be extremely difficult and expensive to separate steel fibres from the concrete mix and re-use them. Similar, if not less onerous, issues exist as those currently faced by the construction industry for concrete-based structures at the end of their life.

Final comments

Although the use of steel fibres with composite slabs is nothing new, uptake is expected to rise as economic and sustainability drivers push this approach forward. With an experienced contractor and ready-mix concrete provider, the benefits can far outweigh the issues. In design, if by testing is not adopted, a mechanics-based methodology implemented within software provides a practical alternative. ■

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The ever changing moods of composite slabs

There has been significant interest recently in the vertical shear resistance of composite slabs. This is partly due to different values offered by manufacturers of products that are physically very similar. It has also been of interest because of the work that has been undertaken to evolve EN 1994 into its Generation 2 version (SCI has produced a so-called Eurocode Nugget that presents the revised rules for vertical shear resistance of slabs). In this article Graham Couchman takes a broader look at the work that has been done, and raises some practical issues for designers to consider. Composite slabs are also discussed in a more general context, in particular how their behaviour apparently changes depending on loading and other conditions.

Introduction

Various researchers, including those responsible for drafting the Generation 2 EN 1994 rules, are in agreement that the current Eurocode approach of only taking into account the vertical shear resistance of the concrete is conservative for composite slabs. It is worth noting that (one of) the reason(s) why EN 1994[1] only considers concrete resistance is because it refers 'back' to EN 1992[2] in order to comply with the Eurocode philosophy that content cannot be presented in more than one Eurocode, or Eurocode Part. Some researchers suggest the conservatism of the current EN 1994 approach is by a factor of four in some cases! A number of existing, non-European, national codes already add contribution from the decking. In France , designers adopt non-conflicting complementary information (NCCI)[3] that uses a clear first principles and apparently sensible approach, proven through use in practice over nearly a decade, to combine the:

- Concrete resistance (as given in the current EN 1994)
  - This takes into account the flanges of the decking as tensile reinforcement
- Shear buckling resistance of the decking webs (taken from EN 1993-1-3[4])

For this approach to be valid it is important that the decking is sufficiently anchored to be able to provide the necessary level of force. When the decking is continuous it will be fully anchored at the face of the support, and when discontinuous it is traditionally assumed that thru-deck welded studs provide a 'deemed to satisfy' level of anchorage (alternatively sufficient contact area is needed between the deck and concrete). Alongside a significantly more complex alternative model for certain situations, the 2022 draft of prEN 1994-1-1[5] provides a simple alternative that is, unsurprisingly, very similar to the French approach:

$$V_{v,Rd} = V_{c,Rd} + k_v V_{b,e,Rd}$$

The difference is that a factor k_v has been introduced, potentially to down rate the contribution from the decking. Three notes in the prEN suggest what value should be used by a designer, and although not stated in the draft document they provide alternative ways of achieving the same end result. The easiest of the three options to understand, and the one advocated for use by SCI, is Note 2:

> The value of k_v is to be taken as 0.5 when $V_{b,e,Rd}$ (the effective resistance) is considered as the design value of vertical shear resistance of the profiled steel sheeting $V_{b,e,Rd}$, unless the National Annex gives a different value.

When we appreciate that the three notes are alternatives, then it can be understood that this reduction factor is intended to be a simple and presumably conservative way of allowing for the combined effect of shear and moment, which Note 3 explicitly states should be considered together. The origin of this 50% reduction, and why it may be deemed relevant, are explained below.

The behaviour of composite slabs

Before looking in more depth at vertical shear resistance, it is worth considering how composite slabs are normally assumed to behave. The majority of such slabs are governed by the construction stage, namely the ability of the decking to support the wet weight of concrete and other construction loads. Deflection of the decking can be critical, to stop ponding of the concrete increasing self-weight. For this reason, decks are often designed to be continuous over at least one support, because of the resulting structural benefits (Figure 1).

At the 'normal stage' the concrete has hardened, and the decking acts as external tensile reinforcement when the slab is subject to sagging. This is achieved through the embossments and overall form of the deck, which assure structural interaction between concrete and steel by resisting interface slip (like the studs on a composite beam). The cross sectional area of the decking is large, and the lever arm large, so significant sagging resistance can be generated. The hogging resistance, where fabric typically provides the tensile component, is relatively much lower and in the interests of simplicity of design is neglected. The slabs are assumed to be simply supported, even when the decking and concrete are continuous at one or both ends.

The final 'stage' we consider is when the slab is subject to fire from below. The decking is totally exposed and loses much of its strength. To compensate for this strength loss, bars may be used in the troughs. Being insulated by a certain amount of concrete they remain cooler and so retain more strength. However, in the UK we normally avoid the use of bars, and despite its loss of

Figure 1: Decking continuous over several bays (courtesy SMD)

strength do make an allowance for the decking. We also take into account the hogging resistance over continuous supports, which is relatively much more important than at ambient temperature because in hogging the tensile component, the fabric, remains 'cold'. The UK method is justified by a multitude of tests over many decades, as well as having a plausible mechanical model as described previously.

So the decking apparently goes from being continuous, to simple span, and back to continuous when we are considering the behaviour of the composite slab subject to fire. The concrete goes from having no continuity over internal supports to having full continuity. But nothing physical changes!

A problem?

Even though a slab may be assumed to be simply supported, if it is continuous over internal supports it will attract hogging moment. As noted above this condition applies to the majority of composite slabs constructed in the UK. That moment will certainly be lower than one determined assuming elastic behaviour and uniform stiffness, because the slab in hogging will be less stiff and therefore shed moment into the span. Drawing an analogy with beams and semi-continuous design, the slab in hogging will be 'partial strength', i.e. have a moment resistance that is significantly lower than that of the slab in sagging. It might therefore be sensible, and certainly not unconservative, to assume that the section in hogging reaches its ultimate resistance, the concrete is cracked, the deck plastified, we get rotation, and moment is shed. The slab in hogging is then rotating as a plastic hinge.

The moment resistance in hogging is a combination of:

- Compression in the lower parts of concrete acting as a couple with tensile reinforcement in the upper part of the slab
- The moment resistance of the decking

So, in theory at least, one might imagine that the decking is working at full capacity in bending, even if it is assumed in design to be acting like a pin.

If we then turn to EN 1993-1-3[4] to see how a deck behaves in combined bending and shear (and axial force) we find this:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{Rd}}}+\frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{y,Rd}}}+\left(1-\frac{M_{\mathrm{f,Rd}}}{M_{\mathrm{pl,Rd}}}\right)\left(\frac{2V_{\mathrm{Ed}}}{V_{\mathrm{w,Rd}}}-1\right)^2 \leq 1.0$$

Where:

N_{Ed}	is the imposed axial force
N_{Rd}	is the design resistance of the cross section for uniform tension or compression
$M_{\mathrm{y,Ed}}$	is the imposed moment
$M_{\mathrm{y,Rd}}$	is the design moment resistance of the cross-section
V_{Ed}	is the imposed shear force
$V_{\mathrm{w,Rd}}$	is the design shear resistance of the web
$M_{\mathrm{f,Rd}}$	is the moment resistance of a cross section comprising the effective area of the flanges alone
$M_{\mathrm{pl,Rd}}$	is the plastic moment resistance of the cross-section

Already apparent from this formula, the same clause nevertheless emphasises that 'no reduction due to shear force need not be done' (sic) provided that (the applied shear is no more than 50% of the shear resistance). It is understood this clause is the origin of the 0.5 suggested in Note 2 of the prEN clause stated previously.

However, we are not interested in how much moment resistance remains in the presence of shear, we are interested in how much shear resistance remains in the presence of (unwanted, unneeded but nevertheless present) moment. It is informative to try some numbers in Equation 6.27:

- Assume that the flanges contribute 80% of the bending resistance of the complete deck

Figure 2: A continuous floor plate awaiting concrete pouring. Rows of studs indicate the beam lines, and even with continuous concrete the slabs will be designed to be simply supported at these points (at ambient temperature)

- Assume that the applied moment is 90% of the moment resistance
- Then to satisfy:

$$0.9+0.2\left(\frac{2V_{\mathrm{Ed}}}{V_{\mathrm{W,Rd}}}-1\right)^2 \leq 1.0$$

- Requires that $V_{\mathrm{Ed}} \leq 0.85\ V_{\mathrm{W,Rd}}$

So even when the moment is at 90% capacity, we still retain 85% of the shear resistance. This is reassuring, and suggests that the value of 0.5 for the factor k_v proposed by prEN 1994-1-1 may be conservative. What that value does do, however, is provide reassurance that no matter what the (unidentified) moment may be, the section will be able to support the combination of actions. Moreover, using 0.5 will still lead to a significant increase in shear resistance compared to current practice, and avoid it governing in all but extremely unusual cases. Searching for a better result, that might be more difficult to justify, therefore seems rather pointless.

Conclusions

UK practice for composite slab design is another of those methods where pragmatism and engineering judgement are adopted. We make assumptions that simplify the design process, and produce 'the right end result', even though they cannot be physically correct. But in order to exploit safely the benefits of the proposed new Eurocode rules for the shear resistance of composite slabs, it is necessary to revisit the traditional pragmatism. This article has shown that the traditional approach can be combined with the new rules, even though the latter may need to be conservatively applied through the use of a factor that reduces the shear resistance of the decking (as recommended by the new code itself). In many situations the extra resistance provided by the decking will not be needed, but it may be helpful when considering relatively narrow strips of slab supporting a concentrated load.

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Rafter stiffeners – needed or not?

Zealous application of the rules in EN 1993-1-8 has resulted in stiffeners being provided in rafters at the “sharp end” of portal frame haunches – where the haunch flange meets the rafter. David Brown and Bogdan Balan of the SCI report on the investigation to demonstrate that in the particular arrangement of portal frame haunches, the rules in BS 5950 are a more appropriate assessment of the need (or not) to add reinforcement.

The problem at the “sharp end”

The “sharp end” of the haunch is the point where the haunch flange meets the underside of the rafter flange, as shown in Figure 1. In UK practice, the haunch flange is connected to the underside of the rafter with a weld across the rafter flange, often with a leg length equal to the thickness of the haunch flange.

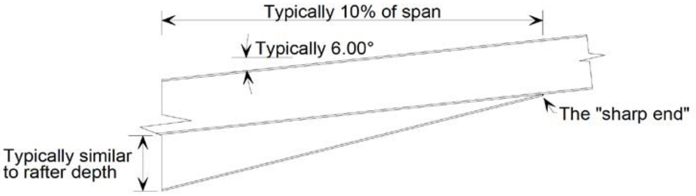


Figure 1: Typical haunch details

The concern at the “sharp end” is whether stiffeners are required in the rafter. In some cases, it is absolutely clear that stiffeners must be provided. If there is a plastic hinge at that location, and the transverse force exceeds 10% of the shear resistance of the cross section, web stiffeners must be provided in the rafter within $h/2$ of the hinge location. This is covered by clause 5.6(2) of BS EN 1993-1-1. BS 5950 has equivalent requirements in clause 5.2.3.7. The recommendations in this article do not change these requirements for stiffeners at plastic hinge locations. When the detail is as shown in Figure 1, the calculation of the transverse force applied to the rafter may be uncertain, but this will be covered later in this article.

Connections to unstiffened flanges

The Eurocode requirement leading to the provision of stiffeners in the rafter is clause 4.10 of BS EN 1993-1-8, which covers welded connections to unstiffened flanges. The situation is shown in Figure 2, with an indicative non-uniform stress distribution. The non-uniform stress distribution is due to the area adjacent to the web being relatively stiff, compared to the more flexible flange tips. Design codes deal with this by determining an effective width over which the stress may be assumed to be uniform.

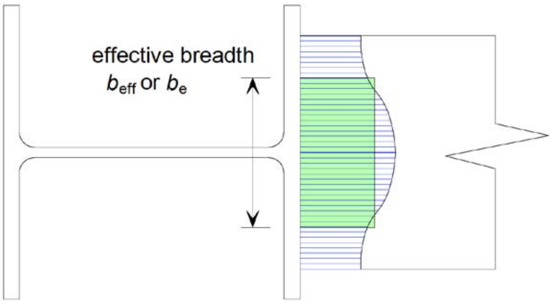


Figure 2: Welded connection to unstiffened flange

The effective breadth is calculated by assuming the web and root radius (or weld) is stiff and allowing for some distribution through the flange, as shown in Figure 3. Distributions range from 1 in 3.5 (BS EN 1993-1-8) to 1 in 2.5 (clause 6.7.5 of BS 5950).

The effective breadth is always narrower than the width of the flange. Clause 4.10(3) of BS EN 1993-1-8 requires that:

$b_{eff} \geq (f_{y,p}/f_{u,p})b_p$ where the subscript “p” refers to the plate. If this requirement is not satisfied, the clause notes “Otherwise the joint should be stiffened”.

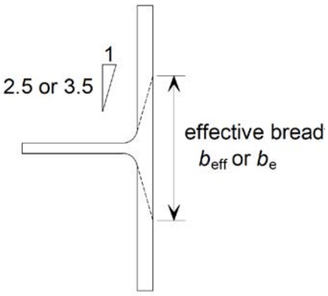


Figure 3: Calculation of effective breadth

If the plate (in this case the haunch flange) was in S355, then

$b_{eff} \geq (355/470)b_p$ or $b_{eff} \geq 0.75b_p$

Since the haunch flange is usually as wide as the rafter this requirement is generally not satisfied and the result is that stiffeners are required. Some design software report this as an advisory note rather than a “fail”.

It should be noted that the Eurocode requirements are not influenced by the load which is being transferred – even if the load were trivial, stiffeners may be required because the requirement is only based on the geometry of the joint.

Bring on the FE analysis

With support provided by BCSA, SCI proceeded to prepare a series of FE models, analysing typical haunch geometries and various levels of load. Mesh sizes were carefully considered, elements in the model carefully selected and the results validated by calculating the resulting shears forces and bending moments computed from the stress of individual elements in the cross-sections considered at various positions along the model. Full details of the FE work are available from the SCI.

The models included an unwelded length of the haunch web (where physical access makes welding impossible) and truncated haunches, both as shown in Figure 4. Models were analysed with and without stiffeners in the flanges, and with different haunch flange thicknesses. The geometry of the haunch and rafter was arranged to reflect typical roof slopes and typical haunch dimensions found in portal frames. This qualification is important, as the design recommendations are for orthodox portal geometry, where the angle between the haunch flange and the rafter flange is generally around 7 - 12°. The design recommendations which follow should not be applied out of context.

Separately, transverse loads of increasing magnitude were applied to a plain rafter section, so that the resulting stress patterns could be examined and compared to the codified requirements to provide web stiffeners under concentrated loads.

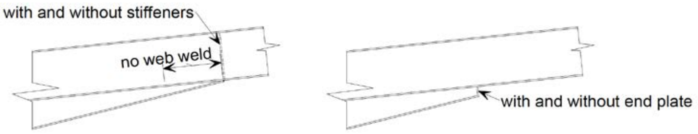


Figure 4: Haunch arrangements investigated

A typical equivalent stress plot is shown in Figure 5. Several more stress plots could be shown, but the results are not significantly different. After carefully reviewing several models it was concluded that there were no indications that the rafter required stiffening.

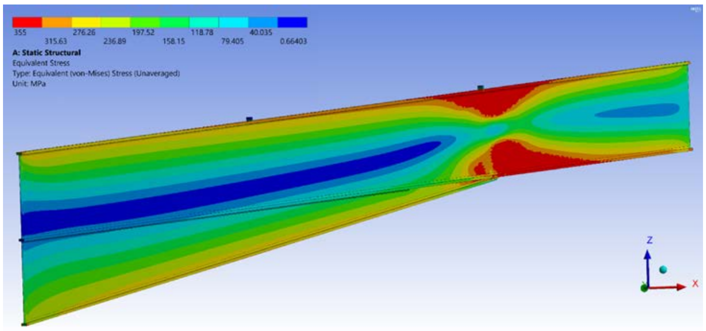


Figure 5: Typical equivalent stress plot

Calculation of the transverse force

Assuming the haunch flange is welded to the underside of the rafter, the transverse force can be calculated as the component of the force in the haunch flange perpendicular to the axis of the rafter. The force in the haunch flange can be determined by calculating the properties of the compound section immediately adjacent to the "sharp end" of the haunch and calculating the stress in the haunch flange. It was found that this approach (perhaps unsurprisingly) gave a good correspondence with the force derived from the FE models.

If the design bending moment is relatively high at the "sharp end" some local plasticity and strain hardening is to be expected – so designers following the recommended approach may find that the calculated force in the haunch flange equates to the cross sectional resistance of the haunch flange. This has implications for the weld between the haunch tip and the rafter, as discussed later.

Recommended design rules

Although the FE analysis did not identify reasons to reinforce the rafter, to have no design rule at all might be considered reckless. The recommended approach is therefore to follow the guidance given in BS 5950, which is supported by several decades of successful practice. Although BS 5950 has similar design rules for welded connections to unstiffened flanges as the Eurocode, the critical difference is that the BS 5950 guidance relates the need for stiffening to the applied force, rather than being based on joint geometry alone.

The BS 5950 procedure is to calculate the maximum force which can be carried over the effective width. The requirement for stiffening is given by (in BS 5950 nomenclature):

If $b_e < 0.5(F_x/P_x)\, b_p$ then stiffeners are required, where:

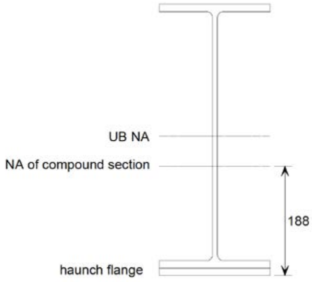
F_x is the applied force (the component of the force in the haunch flange)

P_x is the resistance (the maximum force which can be carried over the effective breadth)

Calculation example

In this example, both the rafter and haunch are assumed to be a 457 × 191 × 67 UB in S355.

In the haunched region, immediately adjacent to the "sharp end", the cross section and calculated properties are shown in Figure 6.



$I_{yy} = 397 \times 10^6$ mm^4

$A = 11000$ mm^2

Figure 6: Cross section adjacent to the "sharp end"

Assuming an applied moment at this location of 480 kNm, the average stress in the haunch flange is 219 N/mm². The force in the haunch flange is 533 kN. If the included angle between the haunch flange and the underside of the rafter is taken as 8°, the force applied transverse to the rafter is 74 kN.

The limiting force P_x is given as:

$$P_x = [4\sqrt{2}T_c^2 + (t_c + 1.6r_c)t_p]p_{yc} \text{ but } P_x \le (5Tc + t_c + 1.6r_c)t_p p_{yp}$$

where:

p_{yc} and p_{yp} are the design strengths of the column and plate respectively (in this case, of the rafter and haunch)

r_c is the root radius of the rafter

T_c is the flange thickness of the rafter

t_c is the web thickness of the rafter

t_p is the thickness of the connected flange (of the haunch)

The dimension t_p is measured parallel to the rafter. It is assumed that this dimension is equal to the weld leg length, which is equal to the thickness of the rafter haunch.

Therefore:

$$P_x = [4\sqrt{2} \times 12.7^2 + (8.5 + 1.6 \times 10.2) \times 12.7] \times 355 \times 10^{-3} = 436 \text{ kN}$$

$$\text{but } P_x \le (5 \times 12.7 + 8.5 + 1.6 \times 10.2) \times 12.7 \times 355 \times 10^{-3} = 398 \text{ kN}$$

Then:

$$b_e = \frac{P_x}{t_p p_{yp}} = \frac{398 \times 10^3}{12.7 \times 355} = 88 \text{ mm}$$

According to BS 5950, stiffeners are required if b_e is less than:

$$0.5\left(\frac{F_x}{P_x}\right)b_p = 0.5\left(\frac{74}{398}\right) \times 189.9 = 18 \text{ mm}$$

Thus, according to BS 5950, since 88 > 18, no stiffeners are required – by a considerable margin.

If the BS EN 1993-1-8 rules are applied, $b_{eff} = 118$ mm. The limiting value of $0.75b_p$ is 0.75 × 189.9 = 142 mm, meaning that stiffeners would be required.

Truncated details

The truncated haunch detail shown in Figure 4 is not popular in the UK, but used in other parts of the world. The advantage of the truncated details is that the web is continuously welded and there is no large weld between the haunch flange and the rafter. If this detail is adopted, it is recommended that some form of welded end plate is provided at the end of the haunch. Without restraint the analysis confirmed that as expected, the tip of the haunch showed signs of distortional buckling.

Welds

Within the FE analysis, welds were modelled with a fine mesh and the same material properties as the parent metal. Full size (leg length equal to the haunch flange thickness) and smaller weld sizes were investigated. In all cases, at higher applied moments, equivalent stresses in the weld were approaching yield with some local plasticity. Physical strains were calculated at different locations in the weld. Whilst all strains were smaller than those measured when welds fracture, the strains with smaller welds were significantly larger than that of the full size weld. The recommendation from this comparison is that if the end of the haunch is to be welded, it should have a full size weld. A smaller weld may experience unacceptable deformations, based on the modelling exercise undertaken.

Conclusions

The work described in this article has demonstrated that in the very specific situation of orthodox portal frame construction (notably a small included angle between the haunch flange and the rafter), the requirement for stiffeners at the "sharp end" of the haunch can be based on the rules in BS 5950. This recommendation should not be applied in other situations. Stiffeners may still be required if there is a plastic hinge at the "sharp end" of the haunch. ■

Let the stress cover the strain

For some structures it is necessary to consider elastic behaviour, but for many a designer can adopt more simple plastic design. The latter is particularly attractive for composite construction, not least because you don't need to get into the sequence of load application, first to steel then to composite components. Graham Couchman of SCI takes a detailed look at the adoption of plastic design for composite beams (i.e. as used in buildings) in light of two recent trends that may add a certain complexity, namely the use of higher-grade steels and the use of deeper slabs.

Either of the two trends mentioned above can mean that in order to achieve sufficient curvature to strain the steel so that it approaches its plastic moment resistance, the strains in the upper fibres of concrete may go beyond the point at which crushing occurs. One way to avoid needing to consider strains explicitly is to limit the plastic moment resistance. According to Eurocode 4[1] this reduction is achieved through a so-called beta factor, which has been mildly revised, and applied to shallow floor beams for the first time, during the development of the Generation 2 Eurocode 4.

Strain and stress in a composite beam

Designers do not generally worry too much about strain. When designing a composite beam they determine the level of applied moment, and normally choose a section that has adequate resistance using simple stress blocks to determine the forces in the steel and concrete. It is assumed that the materials can strain enough to justify those stresses. When checking deflection the stiffness of the cross-section is determined based on the assumption that strains are sufficiently low that the section remains elastic, and so Young's Modulus can be used when calculating deflection values. It is generally appreciated that excessive strain could invalidate the stiffness assumptions, and codes sometimes introduce checks to ensure that yielding does not occur at serviceability. However it is also necessary to ensure that strains are not sufficiently high to invalidate the assumptions about stresses that are behind the resistance calculations.

On the assumption that plane sections remain plane as an element bends, as the curvature of a cross-section increases the absolute values of the strains throughout the depth of that cross-section, be they positive or negative, will increase. For those used to considering bending, but not curvature, the former is the curvature multiplied by the flexural rigidity *EI*. Curvature of a cross-section of a composite beam in which the steel and concrete elements were joined by infinitely stiff (and sufficiently strong) material would result in the strains shown in Figure 1a. A beam of this type would be described as having full interaction. This should not to be confused with full shear connection, which means the sum of the resistances of the connectors is not less than the maximum compressive force the concrete can resist, or the maximum tensile force the steel can resist – whichever is the smaller. Full shear connection is not related to the stiffness of the connectors. Of course nothing in the world of construction is really infinitely stiff – welded headed shear studs will have an initial stiffness of somewhere between 50 kN/mm and 100 kN/mm depending on the slab. A complete lack of stiffness, in other words a non-composite beam, results in slip at the steel to concrete interface, which manifests itself as a step in the strain diagram at that level (Figure 1b). A composite beam with shear connection having a finite level of stiffness would see strains that are somewhere between the two extremes shown in Figure 1. However, things get very complicated when slip is taken into account, so the theory and research reported hereafter ignore it, which is conservative as far as critical strains are concerned.

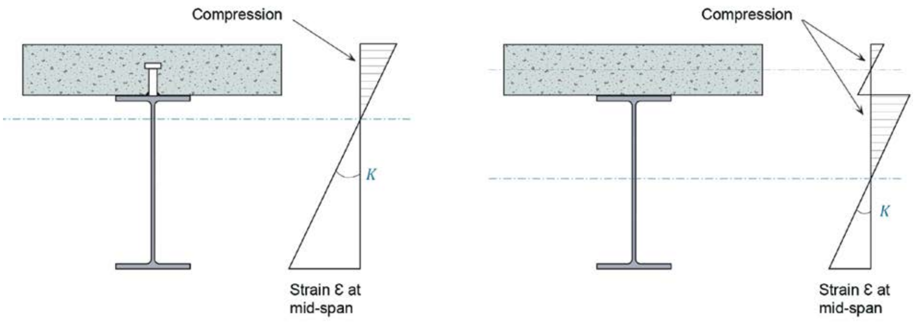


Figure 1: Strains in a composite beam with a) full interaction b) no interaction

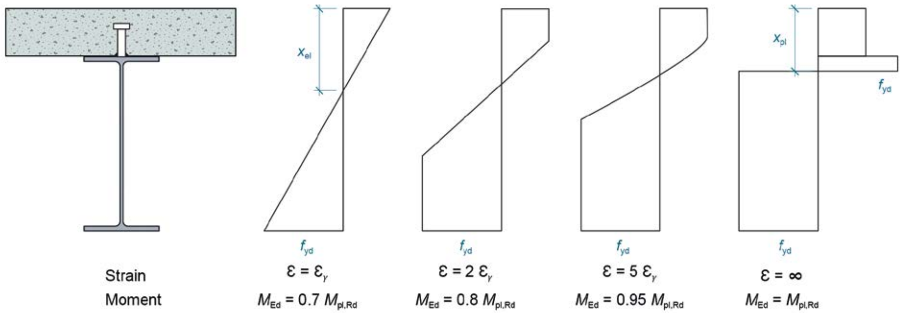


Figure 2: Evolution of stress in a typical composite cross section as curvature increases

If we now consider the stress distribution in a composite cross-section, the linear distributions of strain in the steel and concrete elements represented in Figure 1a will only be reflected in the stress distributions up to the point at which the bottom fibres of the steel yield. As curvature, and strains, continue to increase the stresses will evolve as shown in Figure 2 for a typical cross-section (with full interaction).

Identifying when curvature becomes a concern

Figure 2 shows that as curvature increases, at a certain point the bottom fibres of steel will start to yield. As curvature increases further this yielding will spread up the steel section, up to the point where a sufficient part of the section is at yield for the part that isn't, to be assumed to be (yielding over the total depth would require infinite curvature, and as noted above we do not deal with infinite things in reality). It is generally accepted that achieving 0.95M_p in a numerical model, which simplifies the modelling, is sufficient to represent the ultimate capacity of a beam. The work carried out by ourselves in the development of the rules for P405[2] adopted this approach. Further increasing curvature would be no problem for the steel, in fact we may start to see some beneficial strain hardening of the bottom

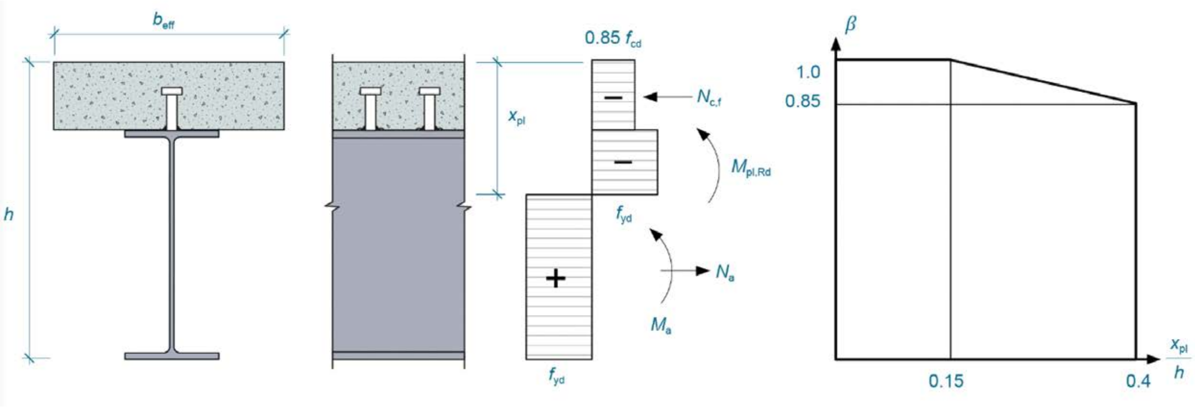


Figure 3:
Reduction factor beta, applied to $M_{pl,Rd}$ according to EN 1994-1-1

flange, but it may be a problem for the concrete, which will have a crushing strain of say 0.35%. The further the top fibres of concrete are from the neutral axis (so with a deep slab), or the further the curvature has to go in order to fully yield the steel (so exacerbated with higher strength steel), the more likely this is to be a problem. When concrete crushing occurs it will limit the cross-sectional resistance. At some point, which is a function of the steel grade and position of the neutral axis, one can no longer assume plastic stress blocks for both the steel and concrete, for this reason. However, it is possible to use a reduced plastic resistance such that the calculation remains simple. This reduction is achieved using a so-called beta factor.

Reducing the effective plastic moment resistance

Eurocode 4 (EN 1994-1-1 clause 6.2.1.2 (2))[1] provides a graph showing the beta factor that is to be used for composite beams using S420 or S460 steel. This graph is reproduced here as Figure 3.

This phenomenon has been investigated further in recent years as part of the development of the Generation 2 Eurocode 4. Tens of thousands of numerical simulations were used to refine the rules, and extend them to consider shallow floor beams as well as downstand beams[3]. To avoid excess complexity the simulations all considered beams with full interaction, and with reference to Figure 1 it will be appreciated that this makes the results, if anything, conservative.

Using stainless steel

SCI is currently preparing guidance that will supplement Eurocode 4 by providing rules to cover composite beams with stainless steel sections. The differing stress-strain behaviour of such steels may affect the need for, and values of, reductions in the plastic moment resistance. The yield strength of a 1.4462 duplex stainless steel beam is the same as that of an S460 carbon steel beam. However, in the duplex stainless steel beam the yield strength is reached at a larger strain. Because of this, for a composite beam in which the beam is made of duplex stainless steel, for a given curvature the portion of the steel section that has not reached yield is always going to be larger than in a composite beam with S460 carbon steel. On the other hand, because duplex stainless steel does not exhibit a yield plateau, any fibre that is strained beyond yield will develop a stress larger than the yield strength due to the earlier onset of strain hardening.

Numerical simulation has once again been used to predict values for beta that will result in equivalence with strain limited design (i.e. design using resistances that are limited by reaching a certain level of strain, rather than the maximum stress that can be achieved). Figure 4 shows some results, comparing values for beams with S460 and 1.4462 stainless steel.

Conclusions

Plastic design offers both simplicity for the designer, and economy of material use. However, in some situations the strain capacity of a material may limit the ability to achieve the levels of strain and therefore stress throughout a cross-section that are needed in order to justify the use of stress-blocks. For composite beams, using a reduction factor applied to the plastic resistance is one way of retaining simplicity and at the same time ensuring that premature crushing of the concrete would not invalidate calculated resistances. This article also suggests, however, that for many beams the impact of this reduction will be relatively insignificant. In other words, for these cases, the difference between the resistance calculated using strain limited design, and that for plastic design (i.e. based purely on stresses), is small. ■

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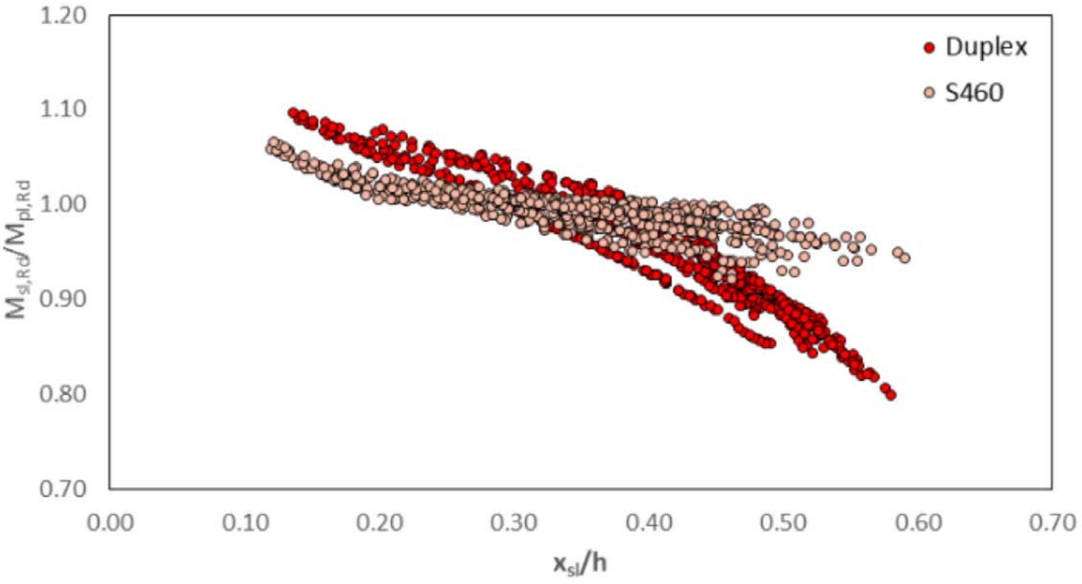

Figure 4:
Reduction factor beta for a beam with either carbon steel or stainless steel

Steel design since 1932

Successive editions of BS 449 illustrate significant changes in steel design and practice over the last 90 years. For a recent project, the design rules since 1932 were reviewed. David Brown of the SCI identifies some of the more interesting features.

Figure 1: Recovered steel members (courtesy Cleveland Steel & Tubes Ltd)

Introduction

No-one can have escaped the fact that minimising embodied carbon in construction is a really important part of the sustainability agenda. In structural steelwork, one opportunity to save both carbon and money is to reuse steelwork. Not to recycle, but to carefully recover beams and columns (Figure 1) during demolition, refabricate and erect into new structures. It is said that around 70% of current enquiries for structural steelwork reflect this desire to reuse in some form. In 2019, SCI published P427[1] which has become the "go-to" guide on steel reuse – if there is any doubt, see *The Structural Engineer*[2] of March 2023.

P427 has a limited scope, covering steel used in construction after 1970. Material characteristics (such as yield and ultimate strengths) are available from around that time, which were then used in the determination of material factors recommended in P427. As interest in reuse grew, extending the advice to cover the reuse of 'older' steel became important. This advice has now been published in P440[3]. As part of the work leading to this supplementary guidance, the design standards and material standards of the time were reviewed, revealing some historically interesting details. The start date was selected as 1932, since this was when BS 449 was first published.

Buckling – in the beginning

Early in the development of P440, the overall objective was to ensure that an 'old' piece of steel designed to the Eurocodes would not be credited with any more resistance than it would have at first use. It might be said that the steel never knew which code it was designed to, and that its structural mechanics has not changed over time – so if we know 'more' now, why not use that knowledge? However, it is clear that steel production may have changed over the last 100 years, perhaps especially during the war years when steel was in short supply. The decision taken was that the buckling codes of the time were appropriate for the steel of the time, and that the advice in P440 should be conservative. That decision resulted in a detailed review of the buckling rules in BS 449 since 1932. Surprisingly, the earliest edition at the SCI was from 1935 – The IStructE library was able to assist.

Compression

The first issue of BS 449 in 1932 had both a formula and a chart to determine the "Working stresses on Pillars and on Compression Members". Designers will recognise the Perry-Robertson expression also seen in BS 5950 and its algebraic equivalent in the Eurocode. In that sense, not much change over the last 100 years. The 1932 edition also included a table of effective lengths, noting that the values were "in respect of typical cases only and embody the general principles which should be employed in assessing the appropriate value for any particular pillar". Thus the designer was left to reach their own decision, in some cases assessing the "efficiency of the imperfect restraint". The length of a member "adequately restrained at both ends in position and direction" was to be taken as 0.75 of the actual pillar length. By the 1935 edition, the familiar values of 0.7 and 0.85 appeared accompanied by guidance on how the end restraint could be assessed. In the Eurocode, the designer must decide what the buckling length is without guidance, which some might say does not show progress.

The 1932 and 1935 editions introduce the design model for columns in braced construction, with beams applying moments based on the eccentricity of the reaction. Those moments may be divided proportionally to the stiffness of the lengths above and below. By 1948 the assumed eccentricity of the reaction was tabulated and the simplification introduced that if the stiffnesses of the lengths above and below did not exceed a ratio of 1.5, the moment could be divided equally. It was not until the 1959 edition when the eccentricity was defined as 4 inches from the face of the section, which is the 100 mm still used today.

Lateral-torsional buckling

In the 1932 edition of BS 449, lateral torsional buckling was simplicity itself. There was no reference to lateral torsional buckling, but rules are given for uncased beams without lateral support. The allowable stress on the extreme fibre of an uncased beam was given in Tons/in² by:

$$11 = 0.15\,\frac{L}{b}$$

If the length L was less than $20b$ then the allowable stress was 8 Tons/in². This is what we would recognise as the plateau. Finally, the L/b ratio could not exceed 50.

Figure 2 shows the comparison for a 305 × 165 × 40 UB. The allowable stress has been converted into a non-dimensional reduction factor.

The plateau extends to 20 × 165 = 3300 mm

No values are given past a length of 50 × 165 = 8250 mm

Figure 2 also shows the elastic critical stress, which has been calculated

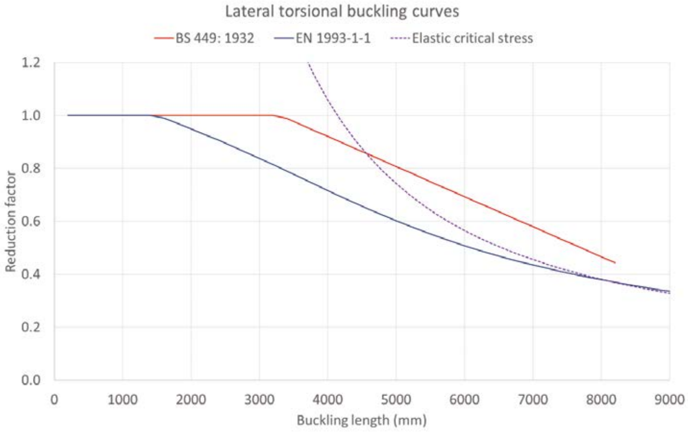

Figure 2: Lateral torsional buckling curves – BS 449:1932 and EN 1993-1-1

from M_{cr} and the reduction factor according to BS EN 1993-1-1 (the special method for rolled sections). The BS 449 resistance looks optimistic, extending out to a plateau of 0.8 in Eurocode terms, and past the elastic critical buckling curve.

The formula for lateral torsional buckling resistance had changed by 1948. The bending stress was given (in Tons/in²) by:

$$\frac{1000}{l/r} \times K_1 \text{ but not exceeding 10.}$$

where K_1 is a bending stress factor, depending on the ratio $\frac{r_{xx}}{r_{yy}}$. Tall, narrow sections have a lower value of K_1.

Figure 3 includes the 1948 buckling curve for the same 305 × 165 × 40 UB, indicating an even more optimistic buckling curve.

The ratio $\frac{r_{xx}}{r_{yy}} = \frac{129}{38.6} = 3.34$ and the K_1 factor becomes 1.415.

The length of the plateau is given by (1000 × 1.415 × 38.6)/10 = 5462 mm, or in Eurocode terms, a plateau length of about 1.23 (contrast with the Eurocode maximum value of 0.4).

The 1969 edition of BS 449 pulled the curve back to the 1932 line, and also limited the resistance to the elastic critical value. The final drama appeared when Amendment 8 to BS 449 was issued in 1989 (four years after BS 5950 was first issued), and the lateral torsional buckling curves were pulled back further and broadly align with the Eurocode curves. Note that BS 449 made no allowance for a non-uniform bending moment diagram. The approach taken in P440, as with compression, was to formulate a curve that is (just) conservative compared to all editions of BS 449.

Connection methods

The 1932 edition of BS 449 is silent on welding, describing rivets, "turned bolts of driving fit" and "black bolts" for work in both the fabrication works and on site. Three years later, the 1935 edition notes welding as an option in both the workshop and on site "when so specified by the Engineer or Purchaser". No design guidance for welds was included.

By the time the 1948 edition was published, welding was extensively addressed with rules for butt welds, fillet welds and (in Addendum No 1) welding round the ends of hollow sections.

Welding of 'early' steel is possible, although better practice would be to form joints as anticipated at the time – using bolts. Double angle cleats would be an appropriate connection for the ends of simply supported beams, for example. If welding is considered, the advice of the Responsible Welding Coordinator will be needed and welding trials should be considered.

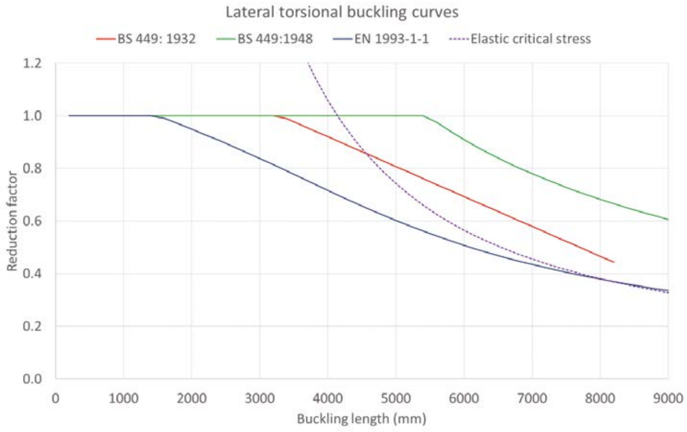

Figure 3: Lateral torsional buckling curves – BS 449:1932, 1948 and EN 1993-1-1

Brittle fracture

The various editions of both the design standard and the material standards chart the advancing knowledge about the risk of brittle fracture. Before the Second World War, there was little interest in impact toughness in building structures. This changed dramatically with the losses of "Liberty ships" during the war, when the problems of low temperatures, high stress and stress concentrations led to around 1500 instances of significant brittle fractures. The material standard for structural steel, BS 15, had no impact toughness requirements specified in the 1948 or 1961 editions, so it seems that after the war the construction industry did not treat the issue with urgency.

It was not until the 1959 edition of BS 449 that the standard included a note that whilst welded structures of steel to BS 15 are normally satisfactory, brittle fracture was a possible failure mode in certain circumstances. Amendment 6 of 1966 included impact test requirements for the first time. Steel designers will recognise the comprehensive requirements in BS 5950 of 1985, and the even more involved considerations within EN 1993-1-10, which represent a huge change from perhaps less informed days.

One of the key recommendations in P440 is that 'early' steel may have low toughness properties, and that even subgrade JR cannot be assumed.

Reducing scope – but increasing page count

The early editions of BS 449 covered the design of the entire building, including design guidance for other materials such as masonry, concrete and mortar. The standards also included imposed loading (50lb/ft² for office floors, which is 2.4 kN/m², so quite consistent – or unchanged over 90 years despite changing use of office space?) and wind loading.

In 1932, wind loading is covered in two paragraphs. A minimum of 15lb/ft² (0.7 kN/m²) was stipulated with a further provision to be made on the sea coast and similarly exposed situations (but no advice on what that provision should be). If the building height was less than twice its width, wind pressure could be neglected altogether, provided the building was "adequately stiffened by floors and walls". By 1948, the clauses covering wind loads ran to about six pages, including internal pressures, local pressures, multi-span roofs and different categories of terrain. By 1959, BS 449 – which was always in A5 format - focussed solely on steelwork design but the page count had still grown from 33 pages in 1932 to 87 pages in 1948 and to 115 pages in 1959. One wonders what the designers of the 1930s would make of our current design standards. ■

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Protected portal frames on fire boundary

Where single storey buildings have external walls close to a site boundary, the walls are required to have fire resistance. Such external walls have commonly in the past been provided with moment-resisting bases and fire protected columns. Richard Henderson of the SCI discusses an alternative approach to providing moment-resisting bases that has been adopted more recently.

Introduction

Building Regulations[1] require that external walls of single storey buildings that are close to site boundaries have fire resistance to protect the next-door land or property. Any structure that provides support to such walls must also have fire resistance. Portal frame structures with walls close to a site boundary have commonly been constructed with moment-resisting bases and fire protected columns to provide the necessary protection. SCI's guidance document P313[2] assists designers in satisfying the Building Regulations and is referred to in them. The publication provides guidance on the design of the moment resisting bases by providing methods of calculating the forces on protected columns that develop due to the effect of a fire on unprotected rafters.

Recently, as a result of changes in the relative costs of various forms of construction and fire protection, there has been a move away from providing moment resisting bases to columns on fire boundaries. The equivalent result is achieved by constructing standard bases to the columns and providing the entire portal frame span next to the fire boundary (referred to here as the boundary span) with fire protection. The structure is therefore able to provide support to the external wall for the required period and satisfy the Building Regulations.

Portal frames with several spans

Where this strategy is adopted in portal frames that have several spans, the boundary span is fire protected and the adjacent span is unprotected and subject to heating in the event of a fire. As set out in P313, the rafters of the unprotected portal frame weaken and form plastic hinges with less than 10% of the normal plastic moment resistance. This behaviour leads to the rafters inverting and results in horizontal and vertical loads being applied to the supporting structure, in this case, the protected boundary span. The arrangement of the structure is such that the method of calculating the loads from the unprotected rafters described in P313 can be applied and the protected boundary span checked to ensure the structure stands for the required period. According to P313 section 3.14, the loads applied by the unprotected rafters can be based on a symmetrical frame, even though one of the columns may be unprotected

Hit and miss fames

Hit and miss portal frames are so-called because internal columns on valley lines are omitted in alternate frames to provide more flexible internal space. Buildings may also be constructed with internal columns omitted in two adjacent frames in a hit, miss, miss, hit arrangement. Loads from the miss frames are supported on valley beams and transferred to the hit frame valley columns.

The absence of a valley column means that miss frames are less stiff in-plane than hit frames and bracing in the plane of the roof is often provided on the valley lines to share loads between the frames. In the fire load case, it is likely that bracing will be required to transfer loads from the miss frame(s) to the hit frames, unless the valley beam is designed for bending in two directions.

Behaviour in a fire

P313 describes the behaviour of portal frame structures in real fires and makes suggestions for the fire protection of various elements when designing boundary columns with moment-resisting bases. This guidance can be applied to structures where fire protected boundary spans are substituted.

Valley columns were observed to have remained standing following seven out of eight severe fires. As a result, P313 recommends that fire protection to valley columns can be omitted unless the ratio $L/Y < 1.6$ where L is the span of the portal frame and Y is the vertical height of the frame to the end of the haunch on the rafter centreline. If the frame formed by the protected outer column and protected rafters is not strong enough to resist the applied horizontal force, fire protection can be applied to the valley column to form a complete portal frame to provide the resistance. In most cases the valley column will need protection.

P313 also indicates that valley beams are assumed to be lightly loaded in a fire and do not require fire protection. With modern cladding and the increased placement of photovoltaic panels on roofs it is no longer considered reasonable to assume valley beams are lightly loaded and do not require fire protection. Table 2.1 in P313 indicates cladding arrangements where it is assumed that the full weight of the outer covering remains in place at the time of rafter collapse. It is reasonable to assume that external photovoltaic panels will also remain and the panel weight should be allowed for when checking the resistance of the frame.

Out of plane stability

The possible modes of collapse in the direction perpendicular to the span of the portal frames must be considered. In the case of boundary columns supported on moment resisting bases described in P313, the out of plane (longitudinal) stability of each perimeter column is assumed to be provided by its base fixity. The lateral restraints provided by the longitudinal members chosen for the permanent works are considered to be adequate without fire protection according to P313 section 2.8.

Stability of the structure in normal conditions in the longitudinal direction can be achieved in different ways: by providing discrete stability systems in the perimeter walls and in the valley lines, or providing wall bracing in the perimeter walls and roof bracing spanning from exterior wall to exterior wall. Stability systems on the valley lines are likely to be goal-post frames to avoid restricting the floor space, particularly in hit and miss framed buildings.

If no moment resisting bases are to be provided, out of plane stability must be provided by alternative means. The permanent vertical bracing and longitudinal eaves beam should be fire protected to ensure stability for the fire resistance period.

Where longitudinal stability to valley lines is provided by a plan roof truss spanning from one outside wall to the other, the members in the plan truss should be fire protected in the boundary span. In the most extreme case, the whole valley line nearest to the boundary wall could collapse completely in the longitudinal direction, although the description in P313 suggests this is unlikely. Such a collapse will involve a rotation of the boundary span in plan. The provision of a fire protected plan truss in the boundary span together with protected valley beams and valley columns will inhibit complete longitudinal collapse of the frames on the valley line. If some longitudinal movement of the valley line occurs, the proportions of the frame limit the reduction in height of the boundary wall, maintaining the integrity of the boundary.

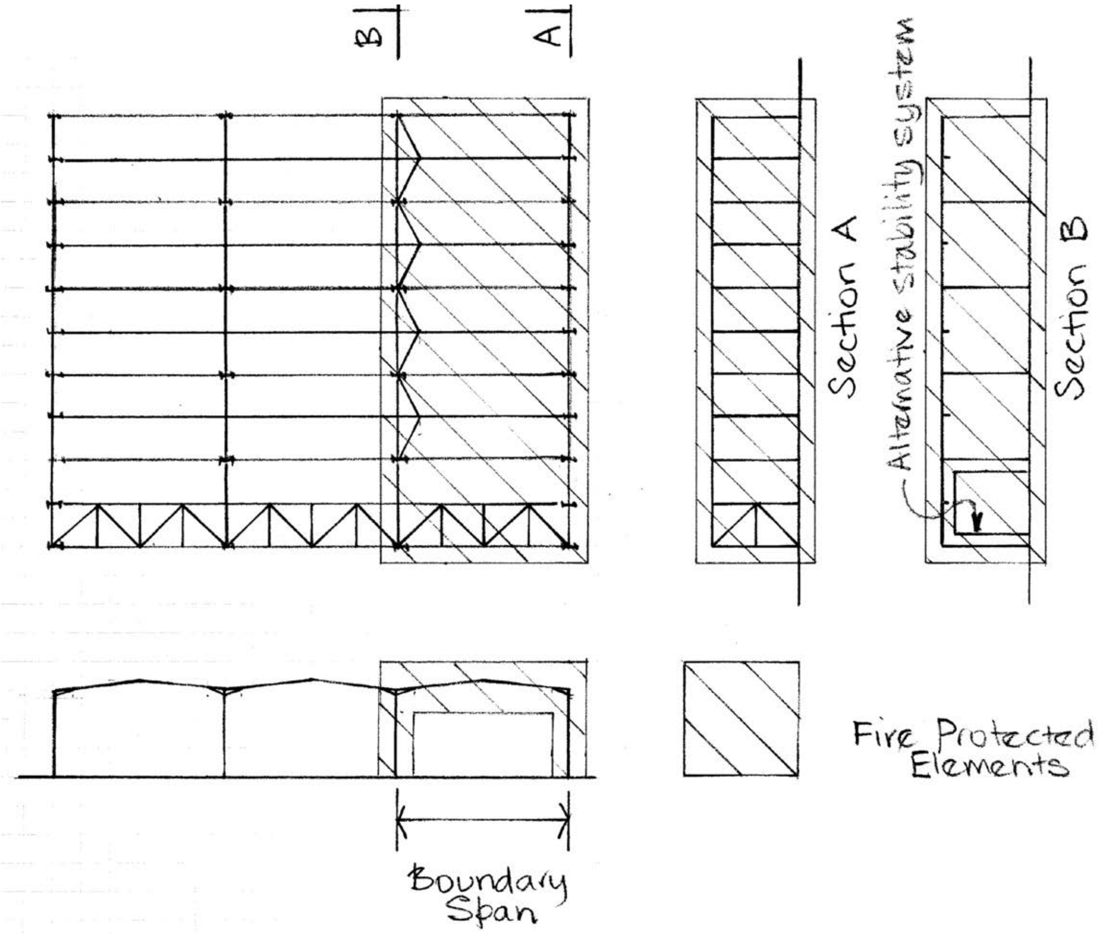

Figure 1: Fire protected elements as listed

Where longitudinal stability systems (bracing or goal-post frames) are provided on the valley lines, fire protection to these systems (including longitudinal members between frames) should be provided on the valley line of the boundary span.

Recommended treatment of a boundary portal span

The elements that require fire protection in a particular arrangement of building will vary depending on the structural arrangement (conventional, hit-miss etc.), geometry (span, column height etc.) and the loading present. An appropriate analysis should be carried out case by case to identify those parts of the boundary spans that are required to resist the loads they will be subjected to.

Where a protected boundary portal span is to be adopted, fire protecting many of the following elements will be necessary, noting that the plan bracing and valley line longitudinal stability systems are alternatives.

- boundary span perimeter columns;
- boundary span rafters;
- boundary longitudinal stability system;
- boundary longitudinal eaves member;
- boundary span plan bracing;
- boundary span valley columns;
- boundary span valley beam;
- boundary span miss frame bracing;
- boundary span valley line longitudinal stability system.

These elements are shown diagrammatically in Figure 1.

Conclusion

Protection to a fire boundary equivalent to that described in P313 can be achieved by fire protecting elements in the portal frame next to the fire boundary (the boundary span) as described. The protected boundary span should be checked to demonstrate adequate resistance to the forces from the collapsing rafters in the next span determined as described in P313. ■

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Benefits of Steel Decking

Although most designers are familiar with composite construction, it is worth a reminder of the benefits that can be achieved and how design choices can have an impact on these benefits. In this article Liam Dougherty of SCI revisits the numerous and diverse benefits, and points to some new developments that can add to these.

Figure 1: Bundles of decking positioned on the structure and the individual sheets then installed by hand (Photo courtesy of Severfield)

Figure 2: Typical detail of decking installation around a column (Photo courtesy of SMD)

Introduction

Approximately 65% of non-residential multi-storey buildings in the UK are steel-framed and the vast majority have composite beams and slabs. Composite construction has therefore contributed significantly to the dominance of steel frames in the building sector in the UK. Its success is due to the strength and stiffness enhancement that can be achieved with an efficient use of materials. Composite floors have been in common usage in the UK since the mid-1980s and have traditionally found their greatest application in steel-framed office buildings, but they are also appropriate for the following types of building:

- Commercial buildings
- Industrial buildings and warehouses
- Leisure buildings
- Stadia
- Hospitals
- Schools
- Cinemas
- Housing; both individual and residential buildings
- Refurbishment projects
- Car parks

Composite slabs can also be used in conjunction with other framing materials, including light steel framing and masonry. Composite slabs consist of profiled steel decking with an in-situ reinforced concrete topping. The steel decking (referred to as sheeting in the Eurocodes) has two main structural functions:

- During concreting, the decking supports the weight of the wet concrete and reinforcement, together with the temporary loads associated with the construction process.
- In service, the decking acts 'compositely' with the concrete to support the loads on the floor. Composite action is obtained by shear bond and mechanical interlock between the concrete and the decking. This is achieved by the embossments rolled into the decking – similar to the deformations formed in rebar used in a reinforced concrete slab – and by any re-entrant parts in the deck profile (which prevent separation of the deck and the concrete).

The stiffness and bending resistance of composite beams means that shallower floors can be achieved than in non-composite construction. The reduced self-weight of composite slabs has a knock-on effect by reducing the forces in those elements supporting them, including the foundations. The reduction in floor depth may lead to smaller storey heights, more room to accommodate services in a limited ceiling-to-floor zone, or more storeys for the same overall building height. As well as the benefits of lower material usage, recent developments in steel and concrete technology allow significant reductions in the embodied carbon of composite slabs. Such examples include the use of low carbon concrete and the reuse of composite slabs after their first cycle of use. While there are many benefits of composite slabs, there are also many benefits associated with the steel decking alone, particularly during the construction stage of a project when it acts as permanent formwork.

Saving in transport and storage

Steel decking is light and is delivered in pre-cut lengths. The profile shape of trapezoidal decking allows the sheets to be stacked on top of each other into tightly packed bundles. Typically, one lorry can transport more than 1000 m² of decking. Therefore, a smaller number of deliveries are required when compared to other forms of construction such as precast concrete. Minimal site storage is required which is beneficial for congested sites.

Speed of construction

Bundles of decking can be positioned on the structure by crane and the individual sheets can be installed by hand as shown in Figure 1. Using this process, crane time is reduced compared to precast concrete floors, which could facilitate the use of a mobile crane instead of a tower crane. More than 400 m² of decking can be installed by one team in a day, depending on the shape and size of the building footprint. Steel decking can be easily cut and fitted around awkward shapes as shown in Figure 2.

The use of the decking as a working platform speeds up the construction process. Decking is usually designed to work unpropped, negating the need for propping and allowing other trades to proceed with clear floor access. Minimal reinforcement is required, and large areas of floor can be poured quickly allowing work to progress up the building. Figure 3 shows the Swiss RE building (the

Figure 3: Swiss RE building showing work progressing up the building

Gherkin) under construction, where the steel decking is being installed on the upper levels, concrete is being placed on the floors below and the façade is being installed below this.

Manufacturers of steel decking can receive an order, roll and deliver to site in 7-10 days. Installers of decking stock common sheet lengths and gauges and can deliver to site in 1-2 days after receiving an order.

Continuous spanning steel decking is faster to install than single spanning decking because fewer fixings are required. Additionally, less crane time is required because the number of bundles required to cover the same area is reduced. The design of decking is often governed by construction stage deflections, so using continuous spans significantly reduces the deflections. For composite beams to use continuous spanning steel decking without losing the benefits of speed, it is essential to thru-deck weld to avoid complications trying to align the studs and holes.

By shortening the construction programme, the impact on neighbours and the public within the vicinity of the construction site from noise, dust and traffic congestion is minimised.

Figure 4: Decking providing a safe working platform and safety 'canopy' to protect workers below from falling objects (Photo courtesy of Composite Profiles UK)

Safe method of construction

The decking can provide a safe working platform when placing the reinforcement and pouring the concrete. The decking also acts as a safety 'canopy' to protect workers below from falling objects as shown in Figure 4.

Structural stability

The decking can act as an effective lateral restraint for the beams during construction, provided that the ribs run transversally and the decking fixings have been designed to resist the in-plane loads. Thru-deck welded shear studs provide sufficient fixity.

The decking may also be designed to act as a large floor diaphragm to redistribute wind loads in the construction stage, and the composite slab can act as a diaphragm in the completed structure. The floor construction is robust due to the continuity that can be achieved between the decking, reinforcement, concrete and the primary structure.

Easy installation of services

Cable trays, pipes, false ceilings, and ventilation equipment are often required to be hung from the underside of a floor. The best way to eliminate the hazardous activity of post-drilling concrete to attach services is to use hangers, and designers are encouraged to specify them. Many decking profiles have re-entrant slots into which proprietary wedges can be inserted to receive threaded rods. The rods serve as hangers for the services, and they have a safe load-carrying capacity of, typically, 100 kg to 200 kg each. Some examples of these attachments are shown in Figure 5 for trapezoidal and re-entrant decking profiles.

Designers wishing to make use of such attachments should seek information, including safe load capacities, from the decking and/or hanger supplier.

Figure 5: Examples of wedge attachment fixings for ceilings and services (Photos courtesy of SMD)

Demountability

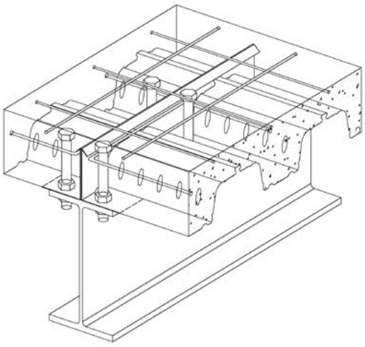

Figure 6: Demountable system, featuring shear studs bolted to the flange of a beam

Many types of building might have a relatively short life span in their first cycle of use, however the economic and sustainability related benefits of composite construction can be retained by being able to demount and rebuild the structure. There are various techniques proposed by which composite slabs are cut into segments, demounted (slab separated from the steel beams), and reused. An example of a demountable system, featuring shear studs bolted to the flange of a beam and embedded in a composite slab is shown in Figure 6. Further information on demountable composite construction systems can be found in SCI P428[1].

Choice of Decking

The choice of decking can affect the degree to which the benefits of composite construction can be exploited. Decking is produced by a number of manufacturers in the UK. Although there are similarities between their profiles, the exact shape and dimensions differ between manufacturers. There are two generic types of so-called shallow decking: re-entrant (dovetail) profiles and trapezoidal profiles.

Shallow decking profiles are between 50 to 80 mm deep and typically span 3 m to 4.5 m, for which temporary propping is usually not required. Some manufacturers also offer trapezoidal decking profiles in excess of 80 mm deep, the deeper of which can span 6 m unpropped as a simply supported member.

The evolution of trapezoidal decking has seen a reduction in the volume of concrete when compared with re-entrant decking. This means that the overall slab self-weight is lower and allows an increased span for the same applied loads. Whilst beneficial for composite slabs, in some cases this reduction in concrete volume has had an adverse effect on shear stud resistances and composite beam design. A reduced volume of concrete can also have an adverse effect on acoustic performance.

Conclusion

As well as serving its primary function of acting 'compositely' with the concrete, there are many benefits in using steel decking. Large quantities of decking can be ordered 'off-the-peg' with minimal storage requirements. The speed of construction is increased, reducing the overall project time. The decking acts as both a safe working platform and cover for construction workers. It provides stability to both the individual steel beams and the overall building structure. Services may be easily installed using proprietary wedges inserted into the re-entrant slots. The choice of steel decking profile will influence the spanning capabilities.

Extensive guidance on the design and construction of composite slabs, addressing the good practice aspects of these activities, is given in P300[2] which has been revised. ■

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Better can sometimes be worse – the dangers of over-strength

As engineers we are always concerned with strength – do we have enough resistance to cope with ultimate (ULS) levels of load? We then consider stiffness – will deflections and dynamic behaviour be acceptable for serviceability (SLS) requirements? This order of priority seems reasonable as something falling down is likely to be more important than it simply deforming too much. However, there is a third criterion that may not always be appreciated, namely that of ductility. Failure to assure appropriate ductility, including accidental failure through the provision of over-strength materials, could result in premature collapse, as Graham Couchman explains and illustrates below through some typical examples.

What defines the strength, stiffness, and ductility of a structural member?

If we assume that stability is not a problem, in other words there is no danger of local or global buckling limiting resistance, then the strength of a member is a function of its cross-section and material strength. Its stiffness is a function of the cross-section and the elastic modulus of the material. These material properties are represented on a stress-strain curve for steel by the initial slope of the curve (up to the elastic limit), and the stress associated with the 'plateau' (or, for a material that shows a less bi-linear response than structural steel, the stress at a certain level of 'proof' strain). Figure 1 reproduces the simplified bi-linear stress-strain curve that BS EN 1993-1-1[1] says may be used in design for structural steel, illustrating these values.

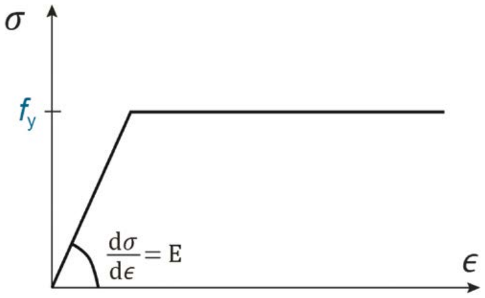


Figure 1: Idealised stress-strain curve for structural steel

Ductility is assured when a material can accommodate a significant amount of strain beyond the point at which it reaches its 'strength'. In other words a long plateau on a stress-strain curve, in compression or tension as appropriate, indicates a material with ductility. Typically, structural steel actually sees an increase in strength along this plateau, as strain hardening occurs. So the ultimate strength of a piece of steel, f_u, exceeds its yield strength f_y. The rules given in Eurocode 3 are only valid for steels that satisfy certain limits. In clause 3.2.2 these limits are given in terms of the ratio between yield and ultimate strengths, a minimum value for elongation at failure over a certain gauge length, and the ultimate strain that corresponds to the ultimate strength. Although the beneficial effect of strain hardening on section resistance is normally ignored, satisfying these relationships ensures that the assumptions concerning plastic behaviour implicit in some of the Eurocode design rules are not invalidated.

Why may ductility be critical?

The way something fails can be very important. Cars used to be designed with large bumpers and strong sub-structures so they could best resist an impact (and remain relatively unscathed). Today they are designed with crumple zones, that contain materials that can deform, i.e. they are ductile, and in so doing absorb the energy of the impact. Significant local damage is accepted. Examples are considered below to illustrate that in steel structures making a component strong is not always the best answer, and indeed building something that contains components that are stronger than assumed in design could be a problem because the structure would not then fail as intended.

Some examples

Partial strength end plate joints

Perhaps the most obvious example of a situation where the materials and components used need to have the correct strength is partial strength (a term that means the resistance of the joint is less than that of either of the connected members) moment resisting joints that adopt end plates, and are assumed to be able to rotate as 'plastic hinges'. Rotation takes place in the joint, not the connected members, because the joint is the weak link. Such joints contain a number of components, such as welds, bolts, and the end plate itself, as well as the two members the joint connects together. Each of these components has a different resistance, which can be determined using the component method as presented in BS EN 1993-1-8[2]. The lowest of the resistances of the different components defines the moment resistance of the joint itself (along with the lever arm relevant to the critical component). As well as different resistances, the components have different levels of ductility – bolts and welds cannot accommodate large amounts of strain (they are brittle), whereas an end plate deforms plastically out-of-plane, exhibiting yield lines and therefore having significant ductility (Figure 2). The component with the lowest resistance will also dictate the ductility of the joint. The so-called moment connections Green Book P398[3] includes some partial-strength standard joints alongside the more usual 'rigid connections', and talks about different failure modes for a joint, namely Modes 1, 2 and 3 (see Figure 3). Mode 1 is the most ductile, and Mode 3 the least. Many economically proportioned joints will exhibit Mode 2, meaning a sufficient level of ductility can be achieved. Joints for use in frames designed according to SCI publications concerning semi-continuous braced frames and wind-moment frames[4,5] will invariably fail in Mode 2. Conversely, for a joint that is designed to be 'rigid' Mode 3 failure may be desirable, because the deformation associated with Mode 2 means the joint may be less rigid than assumed.

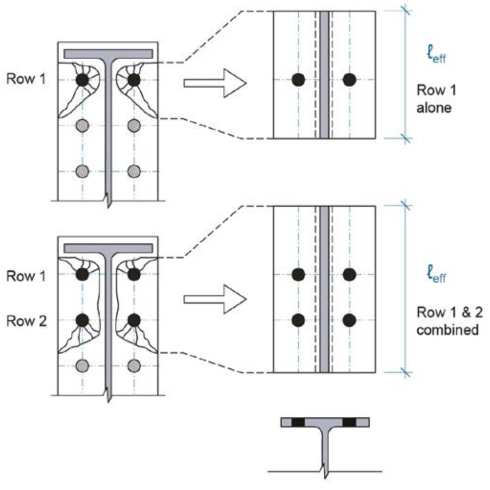


Figure 2: Example yield lines defining ductile bending of an end plate and enabling its resistance to be quantified

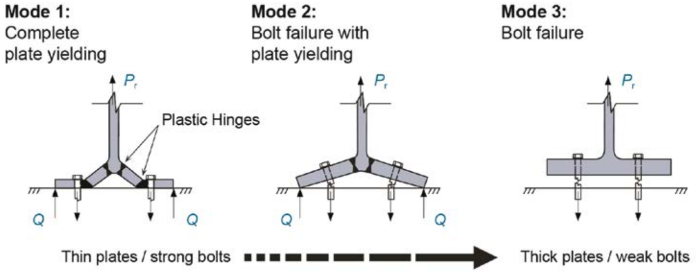


Figure 3: Definition of failure modes for an end plate joint

However, if one of the components in a joint is supplied 'over-strength' it could result in not only the resistance of the joint increasing, but also the failure mode changing. Using an over-strength end plate could result in a joint that was supposed to show ductility, i.e. rotation capacity, becoming brittle because its resistance was then dictated by, say, failure of the bolts. The age of the SCI publications referred to above means the standard details they propose adopt S275 end plates, with plate thicknesses and bolt sizes carefully 'matched' to ensure the bolts are stronger than the plates. However, in 2023 there is a reasonable chance that S355 plates might be supplied, not to mention the fact that steel is normally supplied to achieve a minimum yield strength, with no upper limit specified. Bigger bolts, and welds, could be needed to assure the behaviour assumed by the designer, so at the fabrication stage it is important to ensure the correct material has been used. It should be recognised that of course bolts and welds may also be supplied overstrength!

As an aside when considering steel grades, the maximum loads attracted by structural elements supporting others that are subject to blast loads, depend on the ultimate strength of the connected parts. Unlike yield, ultimate strengths are given as minima and maxima in product standards for steel sections and plates e.g. for S355, ultimate strength must be between 470 MPa and 630 MPa for material up to 100 mm thick. There may be instances where the upper bound strength is the relevant one to use for assessing the effect of blast or other accidental loads.

Shear stud resistances and transverse reinforcement in a composite beam

BS EN 1994-1-1[6] presents rules, through reference to BS EN 1992-1-1[7] for determining how much transverse reinforcement is needed in a composite beam. Although not explicitly stated, the reinforcement should be chosen as a function of the number and resistance of the shear studs. The purpose of this reinforcement is to ensure that the forces transferred locally from the steel beam into the concrete slab via the shear studs can migrate out into a larger width of slab. The relationship between transverse reinforcement and shear stud forces is much easier to understand in the way BS 5950-3.1 clause 5.6.2 presents the design rule, which is simply that the longitudinal force to be resisted per unit length v is the resistance of the shear connectors (NQ, where N is the number of connectors in a group and Q is the resistance of an individual connector) divided by the longitudinal spacing of the connectors/groups s:

$$v = \frac{NQ}{s}$$

A reason for 'sizing' the transverse reinforcement based on the number and resistance of the studs, rather than an applied force, is that failure of the transverse shear plane in a composite beam may not provide the level of ductility (slip capacity) associated with stud failure. Potential planes, as presented in BS EN 1994-1-1, are shown in Figure 4. However, the rules in the codes for plastic design of composite beams assume that the studs have sufficient ductility to redistribute forces between themselves, so it is necessary to avoid non-ductile failure. It is therefore important not to underestimate the resistance of the connectors and by so doing fail to provide sufficient transverse reinforcement. It is also worth noting that even though design with ductile connectors assumes the shear force is equally distributed between them, in reality the studs nearer the support experience higher levels of slip than those near the centre line (for uniform loading). So even when the applied loads do not require all studs to be 'at capacity', some of them will be. Assuming a lower force could, in theory, result in insufficient transverse resistance.

Seismic design

Design for seismic conditions is unusual in the UK, however it provides a very good example of the importance of ensuring that the intended 'weak link' in a structure is indeed the weak link. The use of I-section beams with notched flanges is common, where the notches ensure that the resistance is lowest at a specific point (where the designer has assumed the plastic hinges will form). This avoids the joints being over-loaded. Some steel frames designed for seismic events also adopt so-called fuses, weak points which are designed to be the focus of damage and can therefore be replaced without the need to replace beams and columns during renovation.

Stainless steel

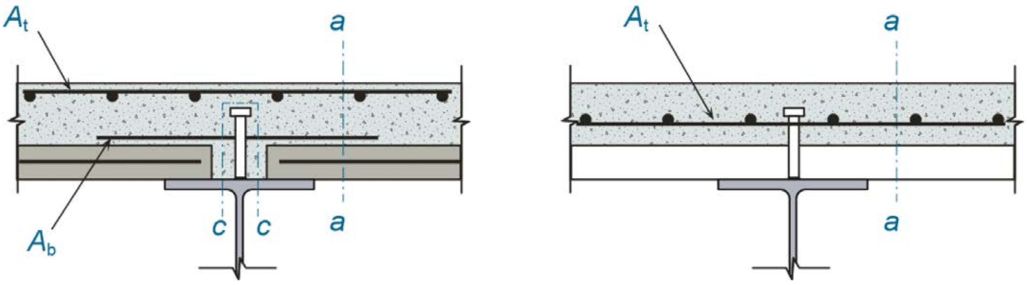
Stainless steel, particularly austenitic, exhibits significantly more strain hardening behaviour than carbon steel. As an example, this could result in the moment resistance of a beam being underestimated by around 20%, depending on the beam's cross-section. The strain hardening exhibited by stainless steel can also lead to a large increase in strength following cold working. The yield strength of a cold-formed hollow section made of austenitic stainless steel can be up to 50% greater than that of the preformed material. Depending on how such steels are to be used, designers should beware that this phenomenon does not result in changed, detrimentally, failure modes.

Conclusions

It feels only natural to assume that if something is stronger than assumed in design, it will be more able to support the applied loads than was assumed, and that this can only be a good thing. However, the examples given above show that the relative strengths of structural components that interact with each other is also important – not just their absolute strengths. If the way in which they interact changes as a result of one of them being stronger than expected, it can affect which component is critical. This can potentially change ductile behaviour of the combination of components into brittle behaviour, and although failure could be at a higher applied load than anticipated, changing the critical component could have very negative consequences. ■

1. BS EN 1993-1-1:2005. *Eurocode 3: Design of steel structures. General rules and rules for buildings* BSI, 2005
2. BS EN 1993-1-8. *Eurocode 3: Design of steel structures. Design of joints* BSI, 2005
3. P398 *Joints in steel construction: Moment-resisting joints to Eurocode 3* SCI and BCSA, 2015
4. P183 *Design of semi-continuous braced frames* SCI, 1997
5. P263 *Wind-moment design of low rise frames* SCI, 1999
6. BS EN 1994-1-1:2004. *Eurocode 4: Design of composite steel and concrete structures. General rules and rules for buildings* (incorporating corrigendum April 2009) BSI, 2004
7. BS EN 1992-1-1:2004. *Eurocode 2: Design of concrete structures. General rules and rules for buildings* (incorporating corrigendum January 2008 and November 2010) BSI, 2004

Figure 4: Potential transverse shear planes in a composite beam (according to Eurocode 4). The slab on the left is formed from a combination of precast and in-situ concrete, the one on the right uses metal decking.



Structural modelling for analysis: Section 7 in BS EN 1993-1-1:2022

The updated version of EN 1993-1-1, BS EN 1993-1-1:2022, has been finalized and is being considered by the team draughting the UK National Annex. Adoption of an updated version of BS EN 1993-1-1 is likely to be in 2028. According to a paper by Marcus Knobloch *et al* [1], Section 5 of EN 1993-1-1 led to many questions and misunderstandings attributed to the different understanding of engineers in different countries, often due to different traditional approaches. The corresponding section in BS EN 1993-1-1:2022 has been completely restructured and rewritten as a result. The section is also renumbered. Richard Henderson of the SCI considers some of the changes.

Introduction

Effect of joints

Section 7 of the code addresses structural analysis and begins by discussing joint modelling in para. 7.1.2. This paragraph indicates that the effects of joint behaviour only need to be taken into account in the analysis where they significantly affect the distribution of internal forces and moments in the structure. The assumption of simple (pinned) and continuous (rigid) joints does not need any specific treatment in the analysis.

Consideration of second order effects

Second order effects are considered in para. 7.2.1. The code gives the same requirements as BS EN 1993-1-1:2005 and states that the effects of the deformed geometry should be considered if they increase the action effects or modify the structural behaviour significantly. Two conditions are provided which determine if second order analysis is required. The first (equation 7.1 in the code) indicates whether second order effects due to member buckling may be neglected in the global analysis. If:

$$\alpha_{\mathrm{cr,ns}} = \frac{F_{\mathrm{cr,ns}}}{F_{\mathrm{d}}} \geq k_0$$

where the recommended value of k_0 is 25, second order effects due to in-plane or out-of-plane non-sway buckling (see Figure 1.1) may be neglected for the global analysis. $F_{\mathrm{cr,ns}}$ is the minimum elastic critical flexural buckling load of the structure and F_{d} is the design load. (The value of k_0 is to be given in the National Annex).

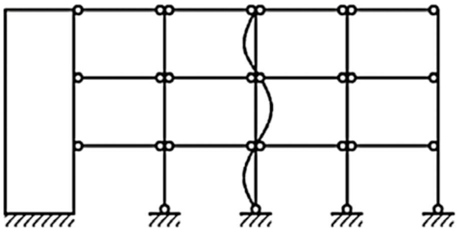


Figure 1.1: Non-Sway Buckling (Figure 7.1 BS EN 1993-1-1:2022)

This condition means that if the design load in the structure is less than 1/25 or 0.04 times the minimum elastic critical flexural buckling load, member buckling may be neglected. The non-dimensional slenderness $\bar{\lambda}$ falls on the buckling curve plateau if:

$\bar{\lambda} \leq 0.2$

so that the buckling reduction factor $\chi = 1.0$: see Figure 1.2

The condition in equation 7.1 can be demonstrated by considering buckling of an individual member as indicated:

$$(\lambda)^2 = \frac{Af_y}{N_{\mathrm{cr}}} \geq 0.04$$

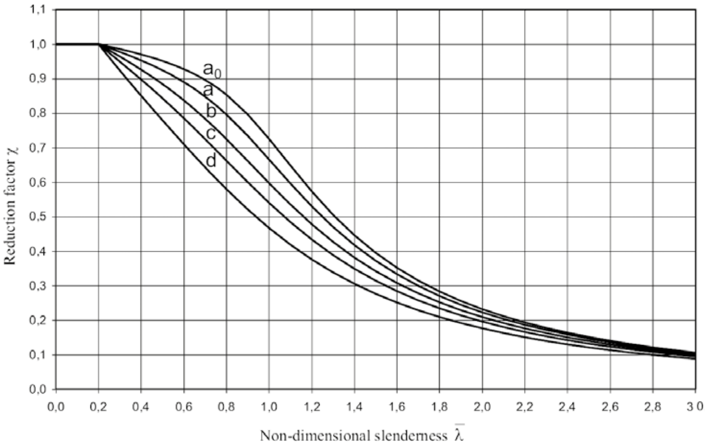


Figure 1.2: Buckling Curves

$$N_{\mathrm{ed}} = Af_y \Rightarrow \frac{N_{\mathrm{cr}}}{N_{\mathrm{ed}}} \geq 25$$

Where the condition is not met, splices in compressed members must be designed for the strut moment resulting from the member imperfection.

The second condition (equation 7.2 in the code) indicates whether second order effects due to global in-plane sway (see Figure 1.3) may be neglected in the global analysis.

$$\alpha_{\mathrm{cr,sw}} = \frac{F_{\mathrm{cr,sw}}}{F_{\mathrm{d}}} \geq 10$$

This is the familiar condition from BS EN 1993-1-1:2005 para. 5.2.1 which indicates that second order effects due to sway may be ignored where the design vertical load on the structure is no more than 10% of the critical load for global buckling. The condition is aimed at ensuring that the increase in the internal forces and moments due to sway second order effects is no more than 10% of the internal forces and moments according to first order theory.

Consideration of lateral torsional buckling may be neglected only when the section is not susceptible to this behaviour. This applies to:

- most hollow sections;
- when bending is about one cross sectional axis but the second moment of area is larger in the other axis;
- when the member is sufficiently restrained that lateral torsional buckling cannot occur.

Para. 7.2.1(10)B indicates that $\alpha_{\mathrm{cr,sw}}$ may be calculated for a storey using equation 7.3, provided the axial compression in the beams is not significant:

$$\alpha_{cr,sw} = \frac{K_{st} H_{st}}{\sum N_{ed,i}} \; ; \; K_{st} = \frac{H_f}{\Delta_f}$$

K_{st} is the lateral rigidity of the storey of height H_{st} given by a horizontal force H_f applied at the top of the storey divided by the corresponding lateral displacement Δ_f. The denominator is the sum of the design axial forces of all the columns in the storey. The minimum value of $\alpha_{cr,sw}$ in any storey is adopted for the whole building. The value of K_{st} must be determined from an analysis model where equivalent fictitious loads are applied to every storey in the structure, in proportion to the design vertical loads applied at that storey. Alternatively a buckling analysis of the whole structure may be carried out for a vertical load case where $\alpha_{cr,sw}$ is the eigenvalue for the first global lateral buckling mode for the structure.

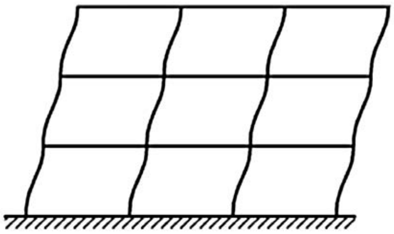


Figure 1.3: Figure 7.1 BS EN 1993-1-1:2022 Sway Buckling

Methods of analysis for ultimate limit state checks

Para. 7.2.2 identifies three approaches to dealing with second order effects:

- entirely in the global analysis;
- partially in the global analysis and partially by verification of the buckling resistance of individual members;
- by verification of the buckling resistance of “Equivalent Members” using appropriate buckling lengths in accordance with the global buckling modes of the structure.

Methods of analysis that may be used for ultimate limit state design checks are labelled M0, M1, M2, M3, M4, M5 and EM in order of increasing complexity. These methods are set out in Figure 7.3 of the code which gives a flow chart for determining the circumstances in which a given analysis method is suitable. For ease of understanding, the methods of analysis described should be considered with the analysis of rigid frames in mind, where the major axis bending of beams and columns provide the resistance to lateral loads on the frame. They are discussed in turn below.

Method M0

Details are given in para. 7.2.2(4). Method M0 applies if equations 7.1 and 7.2 are satisfied i.e.

- Compression members are not susceptible to flexural buckling;
- Second order effects due to sway can be ignored because the structure is laterally stiff;
- In addition, members are not prone to lateral-torsional buckling.

Imperfections do not need to be included in the global analysis and a cross-section check is sufficient. Excluding imperfections from the global analysis means that no equivalent horizontal forces (EHFs) need to be applied.

The elements in structures satisfying these criteria are stocky, making the structures extraordinarily stiff and strong. Such structures would only be adopted in very particular circumstances.

Method M1

Details are given in para.7.2.2(5). Method M1 is similar to M0 except that members are prone to lateral-torsional buckling because of their shape, orientation, degree of restraint or slenderness (see para. 7.2.1(6)). No global imperfection is considered because of the strength and stiffness of the structure. A cross section check based on first order internal forces and moments is sufficient. Verification of the lateral-torsional buckling resistance of beam members is required, based on first order internal forces and moments. Note that no reduction of member resistance due to flexural buckling is applicable because equation 7.1 is satisfied.

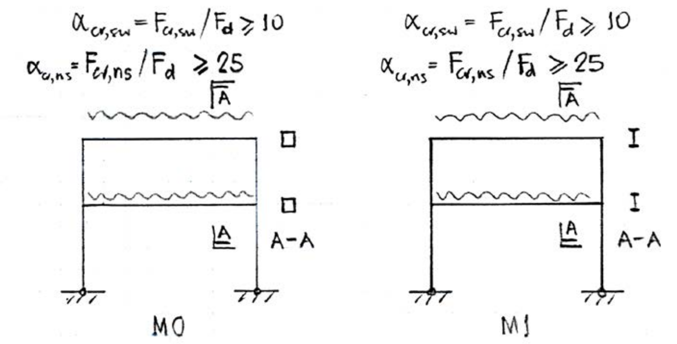


Figure 2.1: Methods M0 and M1

Method M2

Details are given in para. 7.2.2(6). In this case, equation 7.1 is not satisfied and the non-dimensional slenderness of compression members does not lie on the buckling curve plateau. The resistance of members to in-plane and out-of-plane flexural buckling must therefore be verified. Equation 7.2 is satisfied so global second order effects do not result in significant increases in internal forces and moments. However, global imperfections are considered so global EHFs are applied to allow for an out-of-plumb structure.

Cross section checks are based on first order internal forces and moments. In-plane and out-of-plane buckling checks are required based on first order internal forces and moments, considering appropriate buckling lengths for the non-sway mode (effective length factors of 1.0 or less) and corresponding bending moments.

Method M3

Details are given in para. 7.2.2(7)a). In this case, neither equation 7.1 nor equation 7.2 is satisfied. Global imperfections are included in the analysis. Member imperfections may be neglected in the global analysis where the axial load in compressed members that contribute to the sway stiffness of the structure is less than one quarter of the critical buckling load about the major axis. Internal forces and moments should be determined from a second order global analysis. (An approximate method is to use factor k_{amp} to amplify first-order values). Cross section checks are carried out using the partial factor γ_{M1} instead of γ_{M0}, contrary to section 8.2. In-plane and out-of-plane flexural buckling checks are carried out using internal forces and moments from the second order global analysis. The checks are carried out considering appropriate buckling lengths for the non-sway mode (effective length factors of 1.0 or less).

Columns and beams are designed conventionally and member imperfections are allowed for in the buckling checks – section 8.3 in the code.

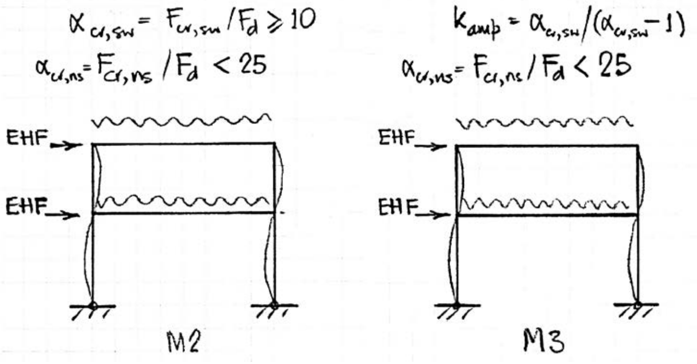


Figure 2.2: Methods M2 and M3

Method M4

Details are given in para. 7.2.2(7)b). As for method M3, neither equation 7.1 nor equation 7.2 is satisfied. Internal forces and moments are determined from a second order global analysis. The effect of member imperfections in compressed members is to reduce the stiffness of the frame and further increase the internal load effects. All in-plane second order effects (including the effects of residual stresses are allowed for in the global analysis and therefore the in-plane member buckling checks may be omitted. Members are subject to a cross-section check using the partial factor γ_{M1}. Out-of-plane buckling checks are carried out using the usual method.

Method M5

Details are given in para.7.2.2(8). In method M5, neither equation 7.1 nor 7.2 is satisfied. Global and member second order effects are included in the global analysis for both in-plane and out-of-plane effects, including torsional effects. As the global analysis allows for all second order effects in the behaviour of members, verification of the buckling resistance of members is not necessary and a cross section check using the partial factor γ_{M1} should be applied.

Method EM

Details are given in para. 7.2.2(9). In method EM, either equation 7.1 or equation 7.2 is not satisfied or both are not satisfied. Imperfections do not need to be included in the global analysis. The Equivalent Member method includes verification of the cross-sectional resistance based on first order internal forces and moments. The effective length of each individual member for buckling checks is determined using the stiffnesses of the members coincident at the joints of the member being considered. Second order effects are neglected in this method and the implications of doing so must be considered. For accuracy, they should be included and this renders use of this method inappropriate.

Conclusion

Many different structural analysis packages are available and they deal with second order effects in different ways. The structural engineer must be aware of the capability of the analysis package used for a particular project so that the analysis results can be applied appropriately and the necessary member design checks can be carried out. It is expected that methods M2 and M3 will be most commonly used for building structures. ■

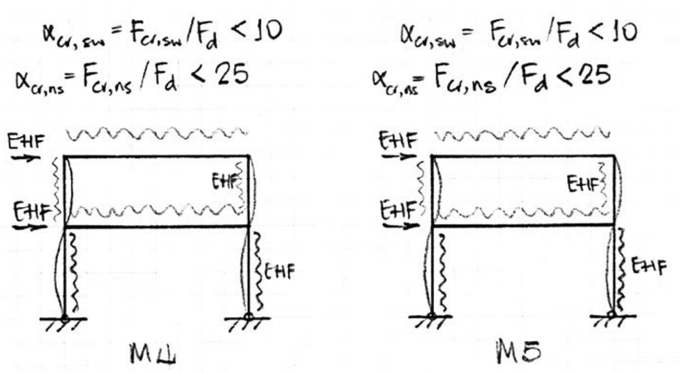


Figure 2.3: Methods M4 and M5

References

1. Marcus Knobloch et al, *Structural member stability verification in the new Part 1-1 of the second generation of Eurocode 3 - Part 1: Evolution of Eurocodes, background to partial factors, cross-section classification and structural analysis*, Steel Construction 13(2), May 2020

celebrating excellence in steel

Call for entries for the 2024 Structural Steel Design Awards

The British Constructional Steelwork Association and Steel for Life have pleasure in inviting entries for the 2024 Structural Steel Design Awards.

Now in their 56th year, the Awards celebrate the excellence of the United Kingdom and the Republic of Ireland in the field of steel construction, particularly demonstrating its potential in terms of sustainability, cost-effectiveness, aesthetics and innovation. The Awards are open to steel-based structures situated in the UK or overseas that have been built by UK or Irish steelwork contractors.

Why enter?

If your project is shortlisted, your company would have the kudos of being part of a prestigious Awards scheme - one with a long history, focussed solely on steel construction and the only one where expert judges visit every shortlisted project to truly appreciate its qualities. In addition, you'll receive:

- Free publicity for you, your project and your client, both online and in the construction press.
- Free attendance at a major Awards event in central London for your project team.
- Recognition of excellence for your project, be it large or small.

How to succeed?

Plan ahead and involve the whole project team from the outset in preparing a high-quality submission, don't leave it to the last minute. Read the entry criteria and particularly the 'Submission Material' section on the entry form and provide exactly what is required, nothing more, nothing less. In addition:

- High quality photos will portray your project at its best.
- A well written, flowing description of the context, concept design, outstanding features and key construction details will allow the judges to swiftly appreciate the essence of your project.
- Broad representation from all parties at the judges' visit will demonstrate collaboration and enthusiasm.

To find out more and download an entry form visit https://www.steelconstruction.info/Structural_steel_design_awards or call Zoe Williams (BCSA) on 020 7747 8139

Closing date for entries: Friday 23rd February 2024

BCSA

STEEL for life

Advisory Desk 2023

AD 497:

NHSS 20 - SHW Series 1800 Certification Requirements

BCSA has been made aware of a number of steelwork contractors, who intend working on schemes where the National Highways Specification Series 1800 for structural steelwork applies, who are confused over the certifications that are relevant to Series 1800 requirements. The National Highways Specification Series 1800 is available at *www.standardsforhighways.co.uk/mchw* .

In particular there is some misunderstanding over the certification requirements relating to the Register of Qualified Steelwork Contractors (RQSC) bridgeworks register and National Highways Sector Scheme 20 (NHSS 20). Some contractors believe that RQSC bridgeworks register certification (Series 1800, Cl 1800.4, 1) and NHSS 20 certification (Series 1800, Cl 1800.5.1, 1) are equivalent, and that only one is required to meet Series 1800 requirements.

This is not correct as the two relate to demonstrating different attributes of a contractor. Both certifications are necessary should a contractor choose to use these as a means of demonstrating compliance with the relevant Series 1800 requirements.

RQSC bridgeworks register certification demonstrates a contractor's general technical capability and competence to undertake the specific works required for a contract.

NHSS 20 certification demonstrates that a contractor operates an independently certified quality management system complying with BS EN ISO 9001 that is relevant to the execution of structural steelwork.

There are other certification requirements that are described in Series 1800 that a contractor should have in place and are described in the schedule below.

The schedule is primarily intended as a reference for contractors and for those supervising contracts, to help understand the various certifications that are required by Series 1800. The schedule will also help:

- auditors understand the Series 1800 certification related requirements when auditing contractors for NHSS 20, and
- in answering any queries relating to these requirements that may be raised with the auditors by contractors.

Contact: **Pete Walker**
Email: **pete.walker@bcsa.org.uk**

Certification Requirement	Source of Certification Requirement	Reason for Certification Requirement	Evidence Required to Demonstrate Compliance with the Certification Requirement	Notes
Registration to the Register of Qualified Steelwork Contractors (RQSC) Scheme for Bridgeworks or equivalent.	Series 1800 Clause 1800.4, 1	To demonstrate a constructor's general technical capability and competence for the type and value of work to be undertaken, to satisfy a general assumption in BS EN 1990:2002+A1:2005.	Registration to the RQSC Scheme for Bridgeworks, or equivalent registration or equivalent evidence of technical capability and competence.	Details of the RQSC Scheme and the RQSC Scheme bridgeworks register can be found at: *www.bcsa.org.uk/member-directories/*
Registration to National Highway Sector Scheme 20 (NHSS 20) or equivalent independently certified quality management system complying with ISO 9001	Series 1800 Clause 1800.5.1, 1	To demonstrate that a constructor has an independently certified quality management system complying with BS EN ISO 9001, to satisfy a requirement relating to quality management measures in BS EN 1990:2002+A1:2005.	Certificate of registration to NHSS 20 issued by a Certification Body registered for NHSS 20 and company listing for NHSS 20 on the UKAS CertCheck website, or evidence of independently certified quality management system equivalent to NHSS 20.	The UKAS CertCheck web site can be found at: *www.certcheck.ukas.com*
Certified Welding Quality Management System.	NHSS 20 Clause 8.5.1 (iii); Series 1800 Clause 1807.1	To demonstrate that a constructor is undertaking welding in accordance with the relevant part of BS EN ISO 3834, as required by BS EN 1090-2, Clause 7.1.	Valid BS EN ISO 3834 certificate issued by a Certification Body registered for BS EN ISO 3834.	BS EN ISO 3834-3:2005 certification required for Execution Class 2. BS EN ISO 3834-2:2005 certification required for Execution Classes 3 and 4.
Certificate of Competence in Pre-loaded Bolting (required where pre-loaded bolting is not excluded as an activity in the constructor's NHSS 20 registration).	NHSS 20 Appendix C	To demonstrate that a constructor has a Level 3 Bolting Co-ordinator in place **and** has a bolting quality management system in place which includes training for bolting inspectors and bolting practitioners.	Level 3 Bolting Co-ordinator Certificate of Technical Knowledge, **and** a company Certificate of Competence in Pre-loaded Bolting.	Details of the BCSA training and certification for bolting competency can be found at: *www.bcsa.org.uk/resources/*

Acronyms:

BCSA:	British Constructional Steelwork Association
MCHW:	Manual of Contract Documents for Highway Works
NHSS 20:	National Highway Sector Scheme 20 - The Execution of Steelwork in Transportation Infrastructure Assets
RQSC:	Register of Qualified Steelwork Contractors

Notes:

1. MCHW Series 1800 and NHSS 20 Appendix C have requirements for the qualification of personnel employed by a constructor undertaking specific execution activities.
2. For constructors who undertake the corrosion protection of steelwork, MCHW Series 1900 (Protection of steelwork against corrosion) requires that they are registered to National Highway Sector Scheme 19A – Corrosion protection of ferrous materials by industrial coatings.

AD 500:
Tolerance at cantilever tips

Although BS EN 1090-2 and the National Structural Steelwork Specification (NSSS 7th) include a permitted deviation at the tip of a pre-set cantilever, this AD note recommends that the only reliable way of achieving a consistent alignment of several cantilevering elements is by including provision for adjustment.

The NSSS and EN contain identical tolerances (9.6.20 and Table B.16(5) respectively) for the "deviation Δ from intended pre-set *f* at end of an erected cantilever of length *L*". The NSSS limit is *L*/200, which is the Class 1 limit in the EN.

It is presumed that the intended pre-set is to allow for the inevitable deflection of the cantilever, or perhaps to deliberately provide an inclined member (for example, supporting a canopy with a drainage fall back towards the building). The pre-set could be zero, or negative (a fall away from the supporting structure) and would normally be provided by cutting the supported end of the member at a small angle.

The assessment of this permitted deviation is fraught with difficulties. The clause limits the deviation "of an erected cantilever", which means the deflected position is to be assessed. This is a departure from the normal concept that deviations are measured at fixed points such as connections, excluding the effects of gravity. This principle is seen most clearly in the assessment of a truss camber (7.6.1 in the NSSS) which is supposed to exclude the effects of gravity by being measured with the component lying on its side. It may be difficult to do this with some trusses, but the principle is clear.

The position of the cantilever tip after erection depends on a number of uncertain contributions:

- The calculated deflection will assume some stiffness of the connection to the supporting structure, and some stiffness of the supporting structure. Both are unlikely to be as assumed. Any continuity – such as back spans – in the supporting structure will modify the calculated deflection. Any difference in the arrangement at different frames will have an impact on the cantilever tip positions.
- The loading on the cantilever and the supporting structure will affect the position of the tip. If the cantilever tip position is to be verified after erection, which is usually the case and is the requirement in the NSSS, the frame designer should specify the loading condition of the supporting frame and cantilever and the corresponding required position of the cantilever tip.
- The accuracy of the cut angle at the cantilever support and the fit-up between components. A very small difference in the angle of cut can lead to a large difference in tip position.
- The temperature when the measurements are taken. Thermal movement of any back spans or equivalent elements will affect the plumb of the cantilever support and the position of the cantilever tip.
- If cantilevers are connected to an unrestrained beam, the twist will vary along the beam length, leading to variability in the cantilever tip position.

It may be tempting to propose that where possible, each cantilever be connected to its supporting member and the accuracy of the fabrication be measured when the components are lying on their side and unaffected by gravity. However, experience suggests that the positions of the tips of a series of erected cantilevers (such as supporting a canopy) will still not align.

Best practice with cantilever members is to build in provision for adjustment, either with thin shims at the support, or by adjustment at the tip to allow supported members (such as a facia detail at the canopy tips) to be aligned. Expecting good alignment without adjustment is generally unrealistic.

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AD 502:
Design to BS 5950 and other standards

The Advisory Desk is regularly asked whether it is acceptable to carry out structural designs to standards other than the Eurocodes. This note draws attention to the statement in Approved Document A of the Building Regulations 2010, Structure, regarding the use of the guidance it contains.

Page 3 of Approved Document A, headed Use of Guidance states in the second paragraph that the document lists "all the documents that have been approved by the Secretary of State" for the purpose of providing practical guidance with respect to the requirements of the Building Regulations 2010 for England and Wales. The list includes Eurocodes BS EN 1090-2, BS EN 1990, BS EN 1991-1, BS EN 1993-1 and BS EN 1994-1 amongst others for the design of steel and composite steel and concrete buildings. The Eurocodes listed contain the most up-to-date and coherent published guidance available for the design of steel structures.

The third paragraph states that there may well be alternative ways of achieving compliance with the requirements and continues "Thus there is no obligation to adopt any particular solution contained in an Approved Document if you prefer to meet the relevant requirement in some other way."

It is therefore acceptable to use alternative standards to Eurocodes for structural design as long as the requirements of the Building Regulations are met. The Approved Document also includes the possibility of using withdrawn standards to demonstrate compliance, within the subsequent guidance under the heading 'British Standards': "There may be alternative ways of achieving compliance with the requirements and there might be cases when it can be demonstrated that the use of withdrawn standards no longer maintained by the British Standards Institution continues to meet Part A requirements."

Designers should however bear in mind that alternative or withdrawn standards such as BS 5950-1:2000 have not been updated with the most recent developments in the design of steel structures. In addition, there may be contractual requirements to use a particular set of standards for design.

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AD 504:
Web to flange welds in box sections subject to bending and torsion

SCI has recently been asked about sizing welds between webs and flanges of a fabricated box section subject to applied torsion and bending. This AD note gives guidance on sizing the welds.

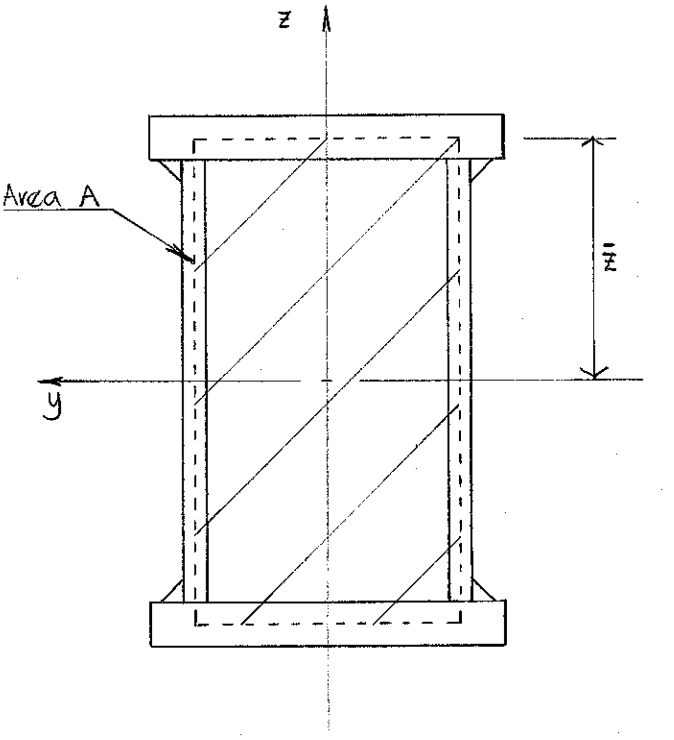
The applied torsion is resisted by shear flow round the box section. The constituent plates deform in shear and complementary shear flows develop parallel to the longitudinal axis of the box and are transferred between the plates by the web to flange welds.

The shear flow s round a box section due to a torsion T is given by:

$$s = \frac{T}{2A} \text{ kN/mm}$$

where A is the area enclosed by the mid-line of the flanges and webs as shown in the figure.

The shear flow between the webs and flanges due to bending is given by the standard formula where A_f is the area of the flange where for two webs and



shear force V parallel to the z axis:

$$s_b = \frac{V A_f \bar{z}}{2 I_y} \text{ kN/mm}$$

The force per mm for sizing the web to flange welds is the sum of the two shear flows:

force per mm = $s + s_b$ kN/mm

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AD 507:
Galvanizing steel of grade S460M

SCI has recently been asked whether the heating of thermomechanical rolled steel of grade S460M when subject to hot-dip galvanizing will affect the properties of the material. This Note addresses this issue in the context of the production and galvanizing processes.

The product standard for structural steel of grade S460M is BS EN 10025-4:2019. Part 4 is titled Technical delivery conditions for thermomechanical rolled fine grain structural steels.

The production process involves a rolling finish temperature of 700°C, lower than the typical rolling finish temperature of 750°C. The lower temperature requires a greater force to roll the material. The process produces a fine grain structure and a tough material which is designated by the letter M. The properties are retained unless the material is reheated above 650°C[1].

Hot-dip galvanizing involves dipping the steel in a bath of molten zinc that commonly has a temperature of about 450°C[2]. The immersion time is typically 4 to 5 minutes but can be longer in certain circumstances. The temperature of the galvanizing bath is therefore below that at which the properties of the steel would be affected.

Galvanizing steels with a yield strength above 650 MPa and steels of high hardness is addressed in SCI Publication P432[3].

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1. https://www.steelconstruction.info/Steel_material_properties
2. *The Engineers & Architects' Guide: Hot Dip Galvanizing*, The Galvanizers Association. https://www.galvanizing.org.uk/publications/
3. Baddoo, N, Chen A, *High strength steel design and execution guide*, (P432), SCI, 2020

AD 509:
Non-slip connections in wind bracing

SCI have received reports that frame designers are specifying non-slip connections for wind bracing – typically on the elevations or in the roof – noting that such connections are subject to load reversal.

Clause 6.1.7.2 of BS 5950 identifies that when load reversal is solely due to wind, preloaded assemblies to produce non-slip joints are not necessary. The guidance is equally appropriate to structures designed to the Eurocodes.

Non-slip joints are more expensive to prepare than connections with ordinary bolts, the fasteners themselves are more expensive and the installation will cost more than connections with ordinary bolts.

In some cases, such as site connections of large trusses or moment resisting connections in plate girder splices, non-slip joints are necessary, but as has been demonstrated by decades of successful practice, this is not the case for wind bracing.

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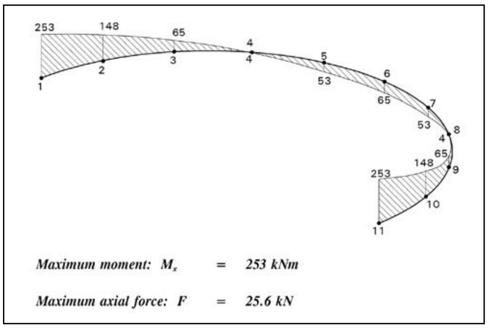
AD 510:
P281 worked example of beams curved on plan

SCI publication P281 was published in 2001 covering the design of curved steel members, in accordance with BS 5950. It is clear that this guide is still used, as SCI receive occasional questions. The most common question, repeated recently, concerns example 6 which covers the verification of a universal beam curved on plan.

The design process starts by applying the vertical load to the curved beam, which produces a bending moment diagram as reproduced from the example:

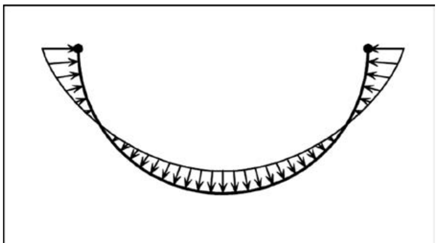


Designers following the example generally question how the axial force of 25.6 kN shown below the bending moment diagram has been determined. The unfortunate answer is that the determination of this axial force should have come later in the process – the value is correct, but the location of the text causes confusion.

The bending moment as shown above is converted into axial forces in the flanges, simply by dividing the moment by the lever arm between the flanges. If the top flange is considered, the flange force is tension near the supports varying to compression at the furthest part of the curved member.

Since the top flange is curved on plan, the axial force just calculated has a radial component of varying intensity – the component is "inward" adjacent to the supports, and "outward" when the flange force is compression.

This varying radial force is shown below (again taken from P281).



Radial load on top flange

The next step is to analyse the curved member again, with the loading shown above. This produces a bending moment (given as 149 kNm in the example) and an axial force. The value of this axial force is the 25.6 kN, which has been quoted at the earlier location in the example.

The process is described in steps in section 8.5.4. As there are two forces "*F*", it may be helpful to identify them separately. In Steps 1 and 2, the equivalent flange force – which leads to the radial components, might be defined as F_1.

Steps 3 and 4 cover the analysis of the member subject to the radial loads, which produces an axial load which might be defined as F_2. In this example, F_2 = 25.6 kN. Referencing this force within Steps 1 and 2 of the numerical example has led to the confusion identified earlier in the Note.

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AD 512:
Eurocode requirements for wind bracing connections

AD 509 reminded designers that according to BS 5950, preloaded assemblies are not required when reversal is solely due to wind loading. The AD recommended that this guidance is equally appropriate for design to the Eurocode.

In fact, BS EN 1993-1-8 provides the same advice in clause 2.6(3). The Eurocode notes that for wind and/or stability bracings, bolts in Category A connections may be used. Category A connections are "bearing type", more commonly known as ordinary bolts in clearance holes. No preloading is required for Category A connections

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AD 515:
Welding of structural nuts and bolts

SCI's Advisory Desk has recently received queries from designers asking if it permissible to weld structural nuts and bolts - typically for situations where there is no access to one side of a connection.

The high temperatures reached during welding will affect the material properties of the nut or bolt and can cause the nut or bolt to become distorted, therefore welding fasteners is generally not permitted. Clause 8.2.1 of BS EN 1090-2 specifies that bolts and nuts shall not be welded, unless otherwise specified, however it is difficult to think of circumstances where welding fasteners would be appropriate.

Mechanical properties of structural fasteners made from carbon steel and alloy steel are given in BS EN ISO 898-1. Annex B of the standard explains that elevated temperatures can cause changes in the mechanical properties and in the functional performance of a fastener.

The Corrigenda to the 7th edition of the National Structural Steelwork Specification for Building Construction (NSSS), published on 3rd April 2023 and which came into force on 2nd October 2023, makes the use of the *Model specification for the purchase of structural bolting assemblies and holding down bolts* mandatory, which in turn states that bolting assemblies shall not be welded.

In situations where access is not possible, various solutions are available which do not involve welding the fastener. Cages which are welded to the plate, constraining the nut, are one solution. Various types of expanding anchors and gravity operated toggle bolts are available for one-sided ("blind") fixing applications.

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AD 517:
Heavy concentrated loads on composite slabs

This note revisits the subject of concentrated loads on composite slabs. Earlier notes AD450 and AD477 considered how to design the slab, and particularly the transverse reinforcement needed, when such loads may start to be significant. In this note we consider the implications of trying to support 'very large' concentrated loads, and the tricky issue of defining 'very large'. The guidance is applicable to both permanent and temporary concentrated loads.

The shear connection found in composite beams normally comprises shear studs, welded to the steel beam at regular intervals. This is effectively a generic solution because the resistance of each stud can be determined by reference to a design code such as Eurocode 4. Using this resistance, the total force that can be transferred between steel and concrete due to 'composite action' can be easily calculated. For composite slabs the steel element is proprietary profiled decking, and interaction with the concrete is achieved through a combination of embossments rolled into the decking, and any re-entrant parts of the profile shape. This means that the 'composite action' that can be achieved is specific to each deck and is determined by tests undertaken by the manufacturer.

BS EN 1994-1-1 Annex B describes how decking tests should be undertaken (Figure 1), and the results analysed. It is worth noting that the test procedure includes some initial load cycles to break down any chemical bond and ensure only mechanical interlock (which can be guaranteed every time a load is applied to a slab) is taken into account in design. Loading then comprises the self-weight, which is of course a UDL, plus concentrated imposed loads at quarter span points. The results are used to determine either the *m* and *k* values, or $\tau_{u,Rd}$. These are used in two different approaches, but in both cases

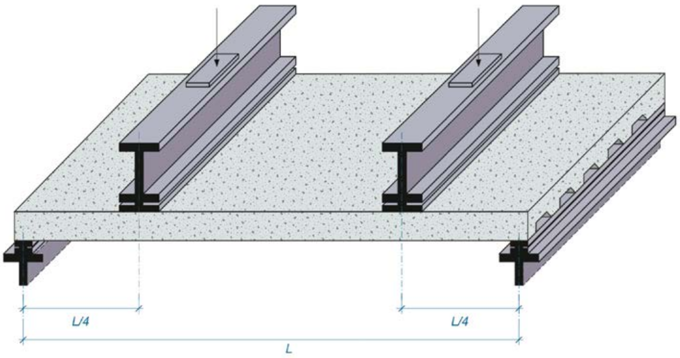

Figure 1: Composite slab test specimen used to determine shear bond, according to Eurocode 4

determine the force that can be transferred between steel and concrete, which is therefore specific to a given deck. In theory they should give the same end result.

The basic means of force transfer between steel and concrete is the same for beams and slabs. As the element bends strains occur in the two materials. For beams, where the position of the interface is clearer, the bottom of the concrete is often in tension and the top of the steel in compression. So, at the interface we get what is known as ‘slip strain’, which is a step in the distribution of strain over the depth of the composite section (Figure 2). This slip strain considered over half a span leads to the end slip that one can see in a composite beam, where the concrete tries to ‘ride over’ the steel. Infinitely stiff shear connection would be needed to prevent this slip.

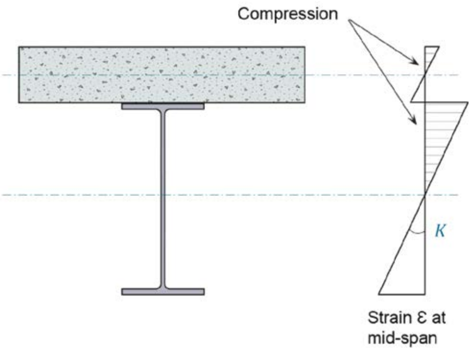

Figure 2: Strain as a function of depth for a concrete slab on a steel beam cross-section

Consideration of the way the load is applied to a slab in the test used to determine the shear bond means that care should be taken using the predicted values for a slab that is subject to a ‘very large’ concentrated load. ‘Very large’ here means one that will sufficiently change the deflected shape of the slab (the component due to self-weight will not change) so that strains in steel and concrete are no longer represented by those that occurred in the test specimen. The location of the load could be an issue, not just its magnitude. If the ‘slip strain’ is lower, then less force might be generated. If the slip is greater, it could exceed the capacity of the shear connection, and this is a more serious problem.

Trying to quantify the strains that the test specimen experienced, and those that the slab would experience due to a significant point load, is possible but certainly not easy. Then even knowing the difference between the two it would be impossible to accurately predict what that would mean for the force transfer between steel and concrete.

One thing that can be easily quantified is the deflection that occurs under different types of load, and because deflections are clearly related to curvatures one could imagine that the slip is a function of the deflection. The imposed loads in a standard test set up will only cause three quarters of the deflection due to the same total load concentrated at mid-span. Deflections due to UDL will be significantly less. So, for the same magnitude of load, slip will be greater for a central concentrated load. Although the level of utilisation in bending would also be higher for the concentrated load, this design output could be misleading – if the greater slip caused the shear bond to pass its slip capacity, then the degree of utilisation would be understated.

Given all this complexity a more pragmatic approach may be preferable:

- Experience tells us that whilst unusual, it is not uncommon for composite slabs to be subject to concentrated loads up to 40 kN. This suggests that any impact on shear bond is limited.
- The slab could be assessed considering the decking as permanent formwork only, i.e. ignoring any shear bond. This would give a good indication of the reliance on the decking, noting that the majority of composite slab designs are governed by the construction stage so a reduced composite resistance may not affect the spanning ability.
- Provide trimming steel below the slab if its resistance alone is insufficient.

Whilst appreciating that designers do not always have the flexibility they would like, some points of good practice are also worth noting:

- Place significant concentrated loads over (or adjacent to) supporting beams whenever possible, to avoid loading the slab in bending.
- Use a large stiff bearing area to reduce the demands on punching shear resistance and maximise the width of slab ‘strip’ that carries the load. Unfortunately, this might result in a stiff loading length that does not follow the deflected shape of slab so some localised crushing of the upper surface could occur.

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A milestone in advisory desk notes

The issue of AD 500 marks a significant achievement in the provision of technical advice to the steel construction industry. Advice was issued from 1988 within SCI’s own journal. When New Steel Construction was initiated in 1992 the advisory desk note was already at number 126, so about 20 were issued per year over that initial period. BS 5950 was relatively “new” at the time, so perhaps there was plenty of advice needed. Since 1992 advisory desk notes have become less frequent (around 12 per year) but hopefully still relevant and helpful.

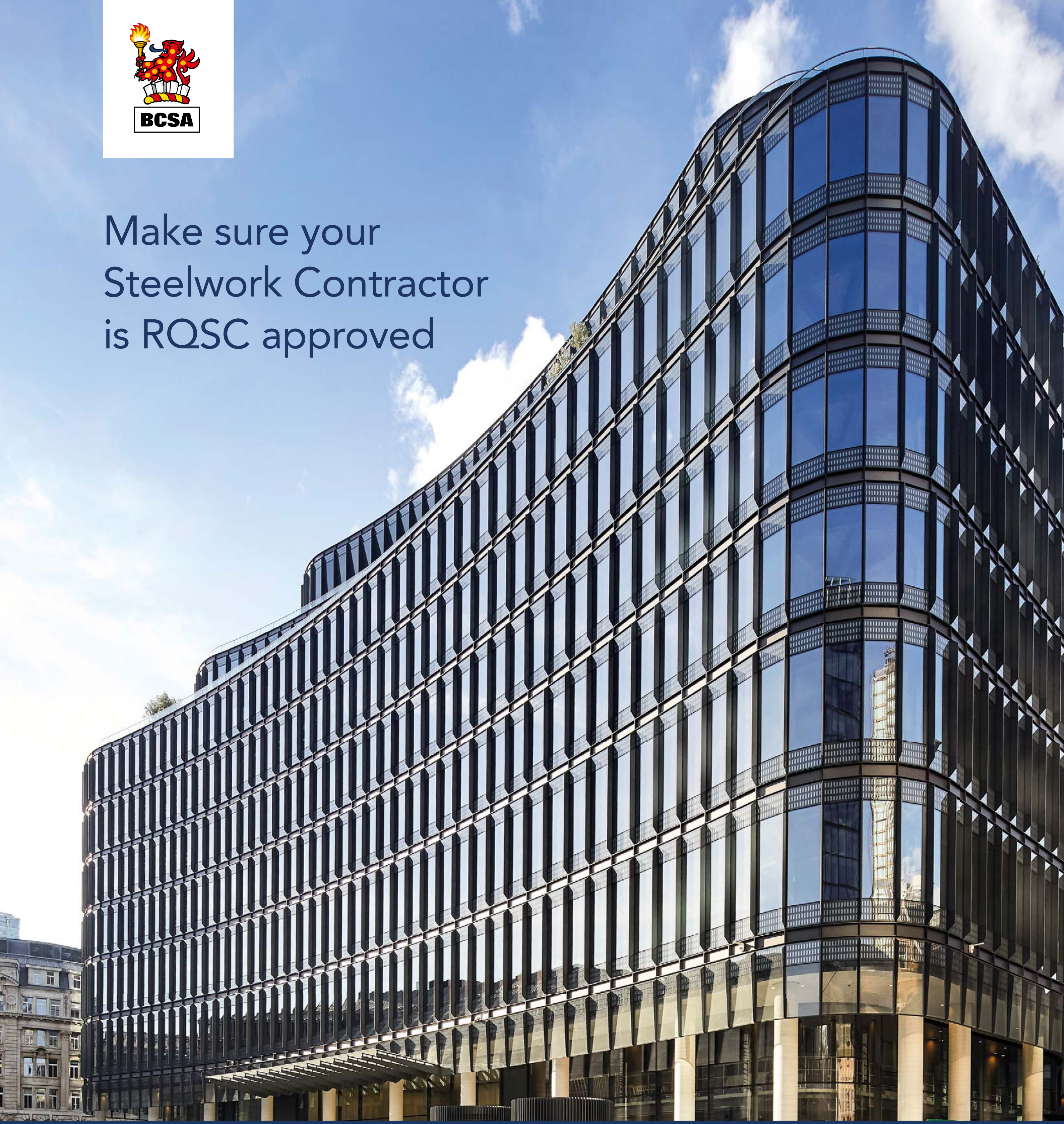
AD 001, which was issued in April 1988 is entitled “guidance on compactness” and is really about the classification limits which must have seemed quite new at the time. The introduction to the AD refers to the “many” queries on the subject. AD 002 commences a theme which reoccurs in AD 006 and continues to the present time – correcting mistakes and other errors in the codes (and sometimes in SCI publications!).

Different writing styles can be seen over the years – some more formal and some rather more conversational. AD 003 refers to “Pundits of BS 449” – an expert in their field frequently called upon to give their opinion. AD 008 refers to “unnecessary beefing up”, which would probably appear as “over-conservatism” these days.

Presumably AD 100 was also a significant milestone around 1990. AD 100 looks backwards to BS 449 and the clauses covering separators and diaphragms. Advice on withdrawn (but still used) design standards is another theme which continues to the present time.

Looking forward to the next 500, the wholesale revisions to the Eurocode suite will no doubt inspire plenty of AD notes. Most AD notes are prompted by questions sent to the SCI’s advisory team, so SCI members are encouraged to keep the enquiries flowing.

David Brown, SCI



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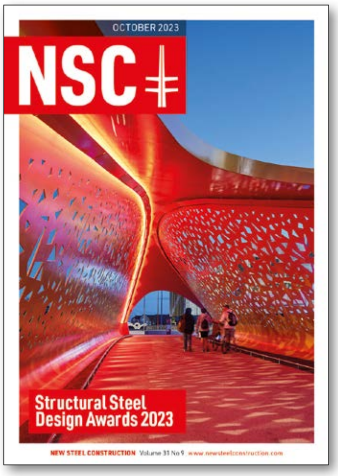

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