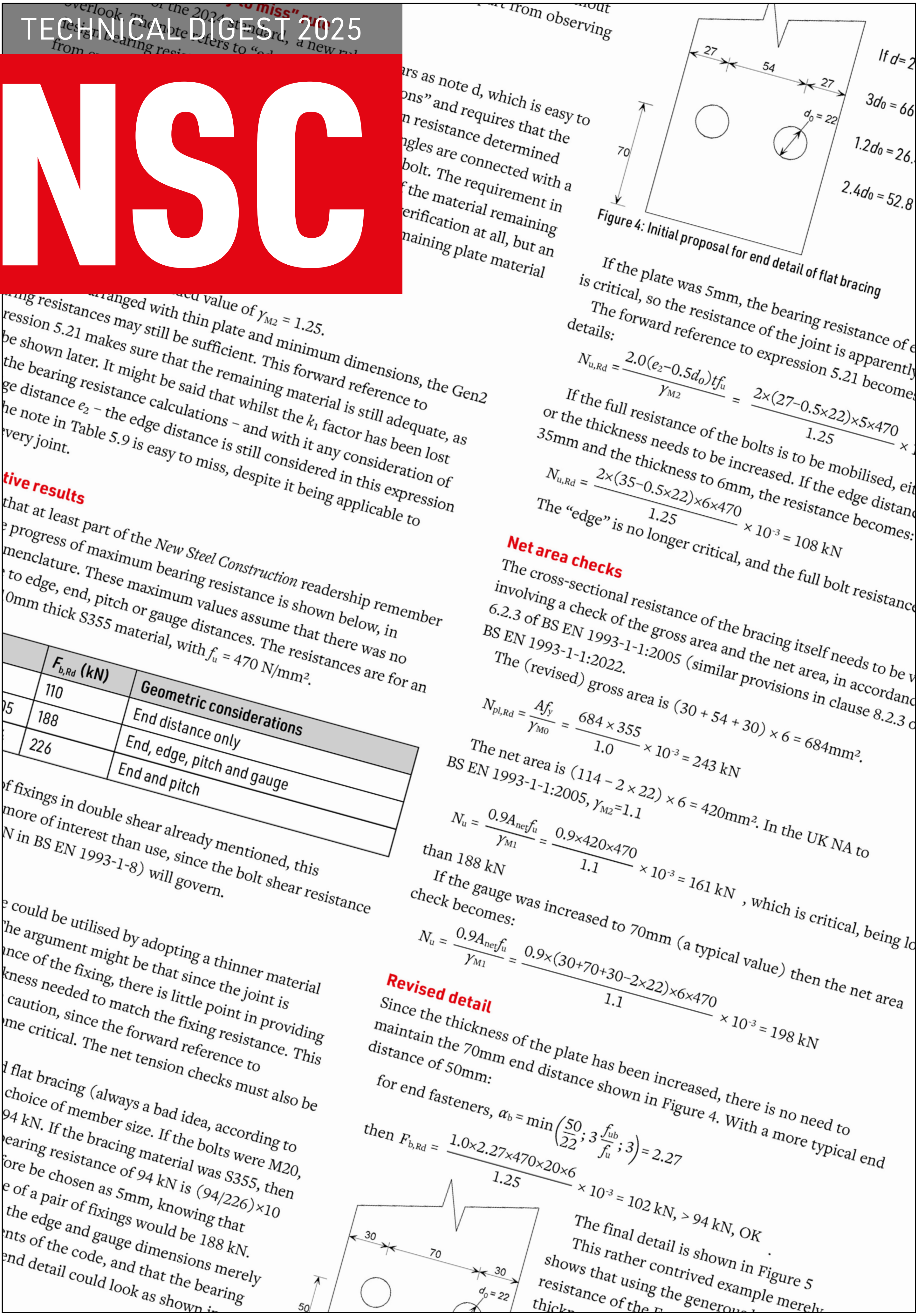
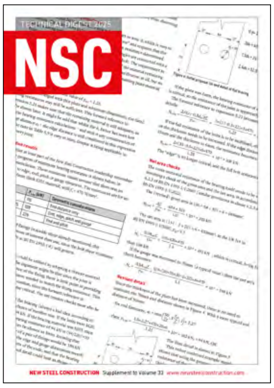
TECHNICAL DIGEST 2025

NSC





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Essential steel construction technical advice for designers

Nick Barrett - Editor

This is the tenth 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).

The Technical Digest was launched after requests from readers that the technical content of NSC be brought together in an easily accessible format, and has earned an established place on the essential reading section of the digital 'bookshelves' of architects and engineers. This Digest brings together all ten Advisory Desk Notes and other Technical Articles published in NSC during 2025, available as a free downloadable PDF at *steelconstruction.info*, 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, ensuring 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 and an increased focus on building safety.

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 through NSC and technical supplements distributed through other specialist construction publications, or at *steelconstruction.info*, a free to use website where everything relevant to steel construction, including cost as well as design guidance, is available. It should be the designer's first port of call for the steel sector's comprehensive 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 provided 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 Digest of value.

New code, new resistance value?

In a continuing series of articles looking at the "Generation 2" Eurocodes, David Brown of the SCI considers the significant changes to the bearing resistance of fasteners.

Introduction

Back in 2005 when the Eurocodes were introduced, one of the most notable changes was the increase in bolt bearing resistance. Although not transparent, BS 5950 was arranged to limit the bearing deformation to 1.5mm at SLS loads – despite the check being undertaken with ULS loads. The 'adjustment' took place in the value of the bearing strength, p_{bs}.

According to BS 5950, if the minimum edge distance, end distance and bolt spacing requirements were satisfied, the only geometrical constraint to consider in the calculation was the end distance – if the end distance was more than twice the bolt diameter, the check was not critical. Based on this, good practice was to make the end distance twice the bolt diameter – so an end distance of 40mm for M20 bolts is adopted for the standardised simple connections in the Green Book. The other well-known rule was that for the (then) common case of Property Class 8.8 bolts in S275, the bearing resistance was equal to the bolt shear resistance if the plate thickness was half the bolt diameter. In those pragmatic days, making end plates at least equal to half the bolt diameter ensured that bearing never governed.

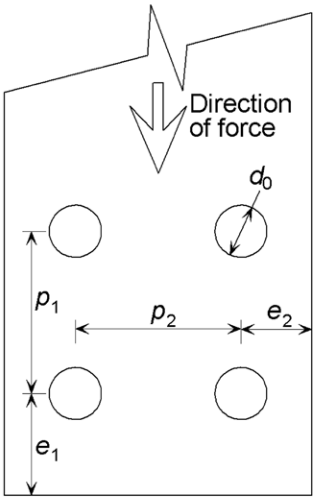


Figure 1: Bolt group nomenclature

In the current version of BS EN 1993-1-8, designers will be aware that the calculations are much more involved. The calculations consider the geometry parallel to the applied load (e_1 and p_1 as shown in Figure 1, and the geometry perpendicular to the direction of the applied load (e_2 and p_2). These considerations are in addition to satisfying the requirements for minimum end distance, edge distance and spacing.

The 'advantage' of the Eurocode was a significant increase in bearing resistance. In common with many other international standards, the Eurocode does not limit the deformation at SLS.

In many cases, having a much higher bearing resistance is no particular advantage, since the shear resistance of the fixing often governs. The one situation where an increased bearing resistance could be of value is when two secondary beams are supported by the web of a primary beam, as shown in Figure 2. The bolts are in double shear, so usually quite capable, but the combined load from each secondary beam must be carried by the primary beam web when a higher bearing resistance could be valuable. Other than this slightly contrived situation, the drama of increased bearing resistances went largely unnoticed back in 2005.

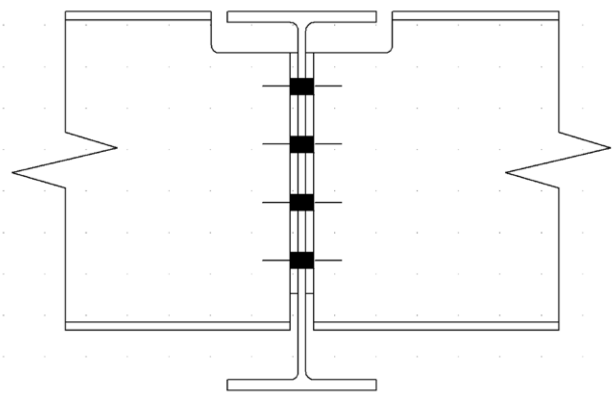


Figure 2: Common web connection

Current EN rules

According to EN 1993-1-8:2005, the resistance of a bolt in bearing is given by:

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$$

where:

α_b is the smaller of α_d, $\dfrac{f_{ub}}{f_u}$ or 1.0

In the direction of load transfer:

For end bolts, $\alpha_d = \dfrac{e_1}{3d_0}$ and for inner bolts, $\alpha_d = \dfrac{p_1}{3d_0} - 0.25$

Perpendicular to the direction of load transfer:

For edge bolts, k_1 is the smaller of $2.8\dfrac{e_2}{d_0} - 1.7$; $1.4\dfrac{p_2}{3d_0} - 1.7$; 2.5

For inner bolts, k_1 is the smallest of $1.4\dfrac{p_2}{d_0} - 1.7$; 2.5

Ever since the Eurocodes were adopted, the description of end, inner and edge bolts has caused some confusion, particularly as "inner" bolts appear both in the expressions for α_d (for bolt geometry parallel to the line of the applied load) and in the expressions for k_1 (for bolt geometry perpendicular to the line of the applied load). If a bolt is not next to an end, it must be an "inner" bolt when calculating α_d. Similarly, if a bolt is not next to an edge, it must be an "inner" bolt when calculating k_1. When calculating k_1, "inner" bolts are not common – typically more than two columns of bolts are required to have "inner" bolts, as shown in Figure 3.

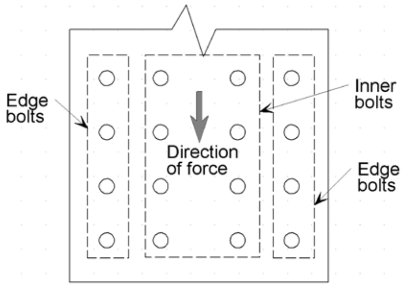


Figure 3: "Inner" and "edge" bolts for the calculation of k_1

Since f_{ub} is generally significantly higher than f_u, the maximum bolt resistance is calculated when $e_2 > 1.5d_0$, $p_2 > 3d_0$, $e_1 > 3d_0$ and $p_1 > d_0$.

If these limitations are respected, the maximum fastener bearing resistance becomes:

$$F_{b,Rd} = \frac{2.5 f_u d t}{\gamma_{M2}}$$

Generation 2 rules for bearing resistance

The drama in BS EN 1993-1-8:2024 is that the edge and gauge dimensions e_2 and p_2 *seem* to have no impact on the bearing resistance. This is not entirely true, as will be shown later. Within the expressions for bearing resistance, the Gen2 formulae make no reference to the k_1 factor which covers dimensions e_2 and p_2 perpendicular to the line of the applied force.

The bearing resistance is given by:

$$F_{b,Rd} = \frac{k_m \alpha_b f_u d t}{\gamma_{M2}}$$

where:

for end fasteners, $\alpha_b = \min\left(\dfrac{e_1}{d_0}; 3\dfrac{f_{ub}}{f_u}; 3\right)$

for inner fasteners, $\alpha_b = \min\left(\dfrac{p_1}{d_0} - 0.5; 3\dfrac{f_{ub}}{f_u}; 3\right)$

For steel grades up to and including S460, k_m = 1.0, and otherwise 0.9.

The maximum fastener bearing resistance is calculated when $e_1 > 3d_0$ and $p_1 > 3.5d_0$

If these limitations are respected, the maximum fastener bearing resistance becomes:

$$F_{b,Rd} = \frac{3f_u dt}{\gamma_{M2}}$$

This is a 20% increase in resistance compared to the 2005 version without any requirement to consider the edge distance or gauge, apart from observing the minimum dimensions.

Edge bolts – the "easy to miss" rule

In Table 5.9 of the 2024 standard, a new rule appears as note d, which is easy to overlook. The note refers to "edge bolts in connections" and requires that the design bearing resistance is no greater than the design resistance determined from expression 5.21. Expression 5.21 is used when angles are connected with a single line of fasteners – so every fastener is an "edge" bolt. The requirement in expression 5.21 is to calculate the ultimate resistance of the material remaining between the bolt hole and the free edge – not a bearing verification at all, but an easily overlooked requirement to verify the fracture of remaining plate material in tension.

Expression 5.21 is given as:

$$N_{u,Rd} = \frac{2.0(e_2 - 0.5d_0)tf_u}{\gamma_{M2}}$$

Note that the recommended value of γ_{M2} = 1.25.

If a detail is arranged with thin plate and minimum dimensions, the Gen2 bearing resistances may still be sufficient. This forward reference to expression 5.21 makes sure that the remaining material is still adequate, as will be shown later. It might be said that whilst the k_1 factor has been lost from the bearing resistance calculations – and with it any consideration of the edge distance e_2 – the edge distance is still considered in this expression 5.21. The note in Table 5.9 is easy to miss, despite it being applicable to almost every joint.

Comparative results

Assuming that at least part of the *New Steel Construction* readership remember BS 5950, the progress of maximum bearing resistance is shown below, in Eurocode nomenclature. These maximum values assume that there was no reduction due to edge, end, pitch or gauge distances. The resistances are for an M20 fixing in 10mm thick S355 material, with f_u = 470 N/mm².

Standard	$F_{b,Rd}$ (kN)	Geometric considerations
BS 5950	110	End distance only
BS EN 1993-1-8:2005	188	End, edge, pitch and gauge
BS EN 1993-1-8:2024	226	End and pitch

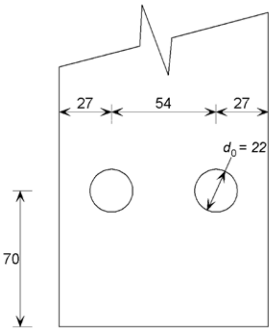
Except in the case of fixings in double shear already mentioned, this increased resistance is more of interest than use, since the bolt shear resistance (92 kN in BS 5950, 94 kN in BS EN 1993-1-8) will govern.

Beware strange details

The high bearing resistance could be utilised by adopting a thinner material (where there is a choice). The argument might be that since the joint is governed by the shear resistance of the fixing, there is little point in providing thicker material than the thickness needed to match the fixing resistance. This policy should be followed with caution, since the forward reference to expression 5.21 is likely to become critical. The net tension checks must also be completed.

Take for example some crossed flat bracing (always a bad idea, according to some) where the designer has the choice of member size. If the bolts were M20, the shear resistance of each bolt is 94 kN. If the bracing material was S355, then the thickness required to deliver a bearing resistance of 94 kN is (94/226)×10 = 4.2mm. The thickness could therefore be chosen as 5mm, knowing that bearing did not govern. The resistance of a pair of fixings would be 188 kN.

Noting the previous comments, that the edge and gauge dimensions merely need to satisfy the minimum requirements of the code, and that the bearing resistance is not reduced if $e_1 > 3d_0$, the end detail could look as shown in Figure 4.



If d = 20, d_0 = 22

$3d_0$ = 66

$1.2d_0$ = 26.4

$2.4d_0$ = 52.8

Figure 4: Initial proposal for end detail of flat bracing

If the plate was 5mm, the bearing resistance of each bolt is 113 kN. Bolt shear is critical, so the resistance of the joint is apparently 2 × 94 = 188 kN.

The forward reference to expression 5.21 becomes critical. With the chosen details:

$$N_{u,Rd} = \frac{2.0(e_2 - 0.5d_0)tf_u}{\gamma_{M2}} = \frac{2\times(27-0.5\times22)\times5\times470}{1.25} \times 10^{-3} = 60 \text{ kN}$$

If the full resistance of the bolts is to be mobilised, either the edge distance e_2 or the thickness needs to be increased. If the edge distance is increased to 35mm and the thickness to 6mm, the resistance becomes:

$$N_{u,Rd} = \frac{2\times(35-0.5\times22)\times6\times470}{1.25} \times 10^{-3} = 108 \text{ kN}$$

The "edge" is no longer critical, and the full bolt resistance may be used.

Net area checks

The cross-sectional resistance of the bracing itself needs to be verified, involving a check of the gross area and the net area, in accordance with clause 6.2.3 of BS EN 1993-1-1:2005 (similar provisions in clause 8.2.3 of BS EN 1993-1-1:2022.)

The (revised) gross area is (30 + 54 + 30) × 6 = 684mm².

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{684 \times 355}{1.0} \times 10^{-3} = 243 \text{ kN}$$

The net area is (114 – 2 × 22) × 6 = 420mm². In the UK NA to BS EN 1993-1-1:2005, γ_{M2}=1.1

$$N_u = \frac{0.9A_{net}f_u}{\gamma_{M1}} = \frac{0.9\times420\times470}{1.1} \times 10^{-3} = 161 \text{ kN},$$ which is critical, being lower than 188 kN

If the gauge was increased to 70mm (a typical value) then the net area check becomes:

$$N_u = \frac{0.9A_{net}f_u}{\gamma_{M1}} = \frac{0.9\times(30+70+30-2\times22)\times6\times470}{1.1} \times 10^{-3} = 198 \text{ kN}$$

Revised detail

Since the thickness of the plate has been increased, there is no need to maintain the 70mm end distance shown in Figure 4. With a more typical end distance of 50mm:

$$\text{for end fasteners, } \alpha_b = \min\left(\frac{50}{22};\ 3\frac{f_{ub}}{f_u};\ 3\right) = 2.27$$

$$\text{then } F_{b,Rd} = \frac{1.0\times2.27\times470\times20\times6}{1.25} \times 10^{-3} = 102 \text{ kN}, > 94 \text{ kN, OK.}$$

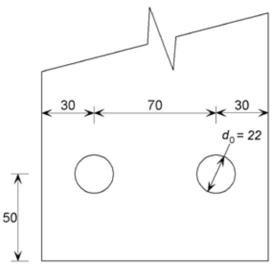


Figure 5: Final detail

The final detail is shown in Figure 5.

This rather contrived example merely shows that using the generous bearing resistance of the Eurocode to reduce material thickness is not always wise.

Reduced bearing resistance to limit deformations

Perhaps acknowledging just how much the bearing resistance increased between BS 5950 and BS EN 1993-1-8:2005, the UK NA has a note to its Table NA.1, suggesting that "in certain circumstances deformation at serviceability might control and γ_{M2} = 1.5 would be more appropriate". If this advice was followed the bearing resistance would be reduced by 17%.

BS EN 1993-1-8:2024 (Gen2) has a revised expression in Table 5.9 "if bearing deformations need to be limited". The reduced bearing resistance is given by:

$$F_{b,Rd,red} = \frac{k_m \alpha_{b,red} f_u d t}{\gamma_{M2}}$$

$\alpha_{b,red}$ is specified in clause 5.9.1(3) as the minimum of 0.8α_b and 2.0 for steel grades up to S460. The standard states that this limits the deformation to $^{d}/_{6}$.

If the bearing resistance was not limited by the edge distance or gauge, then the normal value (assuming no restriction on deformation) of α_b would be 3.0. If bearing deformations needed to be limited then:

$\alpha_{b,red} = \min(0.8 \times 3; 2) = 2$

Continuing the example of M20 bolts in 10mm thick S355 plate, the bearing resistance would become 150 kN, still way in excess of the BS 5950 value of 110 kN, where the (hidden) restriction limited deformation to 1.5mm at SLS.

Groups of bolts

Almost all joints will have inner and end fasteners, often with different bearing resistances. When the load is applied, the plate around the fixings with the lower bearing resistance would (at least in theory) start to deform in a plastic manner, whilst the other bolts picked up more load. The resistance of the bolt group would be reached when all the fixings had reached their bearing resistance – but by this time the material around the fixings with the lowest bearing resistance will have experienced significant plastic deformation. For a joint to behave in a ductile manner, the plastic deformation of the material should take place, rather than the bolts failing in shear. For this reason, clause 5.11(1) of the Gen2 standard states that for ductile behaviour, the shear resistance of the fixings must be more than 80% of the bearing resistance. Yielding of the fastener hole develops at around 80% of its eventual bearing resistance.

If this requirement is not satisfied, then the resistance of the joint is to be taken as the number of fasteners multiplied by the lowest design resistance of any individual fastener. In common situations, the bearing resistance of a bolt is so high that the group resistance will be taken as the sum of the bolt shear resistances. If plates are unusually thin, it is possible to imagine a situation where this clause would catch the unwary. A contrived situation is shown in Figure 6, with a notably thin plate.

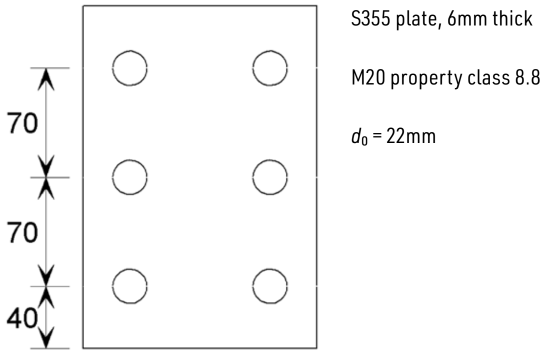


Figure 6: Thin end plate

In this joint:
$F_{v,Rd}$ = 94 kN each bolt
For the end bolts, $F_{v,Rd}$ = 82 kN (80% = 64 kN)
For the inner bolts, $F_{v,Rd}$ = 121 kN (80% = 97 kN)
In this case, the shear resistance does not exceed 80% of the bearing resistance(s), so the joint resistance is 6 × 82 = 492 kN.

Conclusions

The 2024 version of BS EN 1993-1-8 presents significant changes in the expressions for bolt bearing resistance. In many cases the revised resistance is of interest but not significant, since bolt shear is likely to be critical. Joint designers should note that concerns about the edge distance e_2 are not altogether lost – the rather hidden forward reference to expression 5.21 in note d of Table 5.9 may be critical at extreme geometries. ■

What is fire resistance?

Graham Couchman of the Steel Construction Institute charts the history of fire resistance in structural engineering, and concludes that designs, in the most part, have resulted in safe buildings, as failure due to fire is a rarity.

Introduction

Despite the fact that post-Grenfell, much traditional practice is being questioned, hence the numerous recent articles in New Steel Construction, designers in the steel construction sector are very familiar with the concept of designing to achieve fire resistance. Normally, sufficient passive protection is applied to ensure that ambient temperature design governs – there is no need to explicitly design for the fire limit state with reduced loads and reduced material properties. In this article, Dr Graham Couchman of SCI considers what the concept of fire resistance actually means, in particular the use of standard time-temperature curves and standardised resistance periods, and why we try to achieve them. He concludes that whilst the concept of fire resistance is well established and easy to use, we should not be closed to considering other approaches.

The paper *The rise and rise of fire resistance* by Angus Law and Luke Bisby[1] provided much of the background presented in this paper and is gratefully acknowledged. It was published in the Fire Safety Journal and the intention here is to take that knowledge to a wider/different audience, for whom it is equally relevant and valuable. Input from Dr Craig English of Semper is also gratefully acknowledged.

Some background

A key time in the development of the concept of fire resistance was 1903, when following a Fire Prevention Congress in London a paper was published that contained four key concepts. Firstly, that the term 'fire resisting' was more appropriate for use in construction than 'fire proof'. Secondly, that

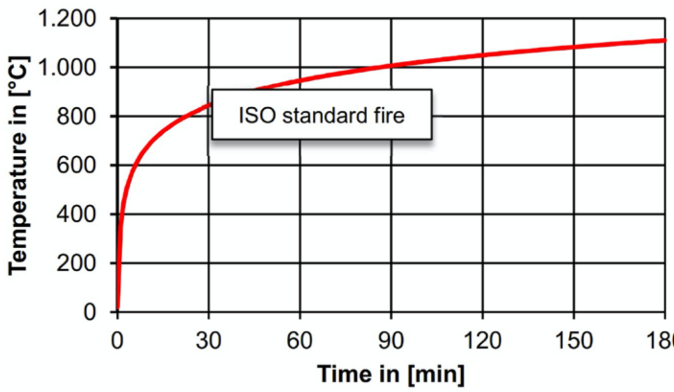


Figure 1: ISO standard 'fire curve'

systems should be classified according to whether they provided 'temporary', 'partial' or 'full protection'. This concept was extended to the third concept of time periods, with resistance for at least 45 minutes, 90 minutes and 150 minutes respectively. Finally, it was proposed that fire testing should be standardised, in terms of duration of exposure, minimum temperature, required loading, and minimum specimen size.

'Full protection' has been interpreted as meaning the structure could survive burn-out of the fire compartment's contents without intervention by fire and rescue services. Options for lower levels of protection were recognised as being practically (commercially) necessary. At the time these definitions were based on a combination of test and real fire experience, which may be a critical point where blurring between real situations and standardised tests started to occur. An obvious example is that the standard time-temperature curve we use in most testing today has temperature that increases up to an asymptote, whereas if contents have burned out then clearly at some point the temperature will start to drop. In 1928 Ingberg made an attempt to link the severity of a real fire to an equivalent period of exposure in a standard fire test – the concept of 'equivalence', which was recognised at the time as having limitations.

Legislation took hold of these concepts, and a century later they are still being widely used. Perhaps this is due to a lack of practical alternatives, but it is still very important to recognise the limitations of such an approach.

Application today

The background summary given above illustrates that the whole area of design for the fire limit state is a bit messy and confused. That confusion seems to be exacerbated in the minds of many by a further blurring, namely that between Building Regulations and Approved Documents (or their equivalent in other nations). Approved Documents were introduced in 1985, and provide ways in which compliance with the Regulations can be demonstrated, for example by testing a specimen in a standard fire test and achieving a stated resistance period. But Approved Document provisions are not the only way of showing regulatory compliance, and indeed in some cases they may even be inappropriate. In the past two years we have seen this dis-joint in the context of load bearing light steel framed walls – Approved Document B[2] (AD-B) requires/allows such walls to be tested with a one-sided fire, but clearly some such types of wall could be exposed to two-sided fire (Figure 2) and simply satisfying the AD-B provisions is now recognised as not then being appropriate[3].

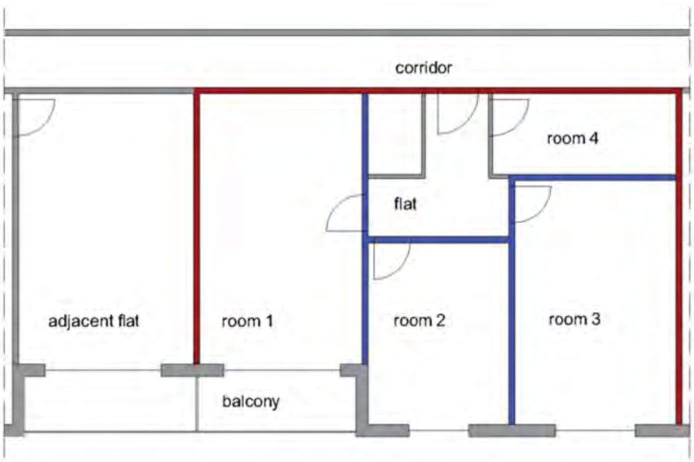


Figure 2: Walls in red would be exposed to fire from one side only. Walls in blue could be exposed to fire from two sides

The fact that periods of resistance recommended in AD-B vary according to building type seems sensible if they have a relationship with burn out of compartment contents. The fact that the resistance period increases with building height appears to be illogical if a relationship with burn out is claimed – an apartment in a multi-storey building will not contain more calorific content than one in a three-storey structure so why does the resistance period go up? Law and Bisby suggest this may have less to do with logic and more to do with harmonising different regulations. However they also note that whether those creating the recommendations appreciated it or not, the adoption of longer periods for taller buildings does increase the effective 'factor of safety'. There is logic to ensuring that taller buildings are more resistant to fires that are not 'average', because of the consequences.

It is worth adding that when sprinklers are provided, the resistance period may reduce.

Alternatives and possible developments

The approach described above has been criticised for several obvious reasons:

- The standard heating curve does not look like a real fire, particularly its lack of a cooling phase.
- Test furnaces are difficult to control, and the thermal and mechanical boundary conditions are unrealistic.
- The 'equivalence' method fails to take into account a number of relevant factors.

Perhaps less obviously, it has long been understood that methods given in typical guidance (AD-B, BS 9999, BS 9991)[2,4,5] provide no explicit measure of building fire safety. The same is true of the deterministic approaches set out in fire engineering codes, such as BS 7974[6]. Not knowing what safety level one's fire design provides is the reason why the Hackitt review recommended the use of outcome-based approaches, and why safety cases are now being prepared for tall residential buildings in order to determine which of them is in a potentially unsafe condition (despite having quite possibly satisfied regulatory requirements).

For the currently very topical case of car parks, a simple alternative would be to consider the heat release rate of different vehicles and how the fire may spread between them[7], and then be able to more accurately quantify the consequences such fires may have on the structure. Those consequences would lead to more informed decisions concerning the level of fire protection, if any, that is required to satisfy life safety, property and environmental objectives.

Despite the obvious logic and potential benefits, rather than the approach described above, it seems likely that future developments in AD-B may include extending fire resistance periods for open sided car parks, and/or requiring sprinklers to reflect the greater fire risks associated with modern vehicles. A requirement for the use of sprinklers could reflect re-consideration of the purpose of Building Regulations – moving towards protecting assets as well as achieving the current objective of saving lives.

More complex fire engineering methods take a more realistic view of how structures behave in fire, not only in terms of fire load but by allowing for variables such as the size of compartments and their ventilation, and the criticality of different structural elements when considering time to failure. Risks should also be assessed in the context of the exit strategy for occupants, access for fire and rescue services etc. Significant savings may be made when such an approach is used, and some structures will more than warrant this level of investment in design.

Conclusions

Design using standard fires and resistance periods is convenient, and it could be argued that this approach has been shown to produce appropriate structures given that structural failures in fire remain a rarity. It is important, however that designers, specifiers, clients and other stakeholders recognise that achieving a certain fire resistance period in a standard test is not always necessary or even appropriate. As we try to construct more 'carbon efficient' structures we should not be content to always use approaches we know to be conservative. ■

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Supporting a boundary elevation

For decades, the detailing of boundary walls of single-storey buildings has followed guidance in P313 and before that, in P087, and going back further, advice from CONSTRADO. Following that advice has been shown to be successful in practice – boundary elevations have stopped the spread of fire to neighbouring property. Increasingly however, questions have been asked about the resistance of the cladding and secondary support steelwork when assessed using the EN 1993-1-2 standard fire curve. To meet this requirement an industry group has prepared a new technical specification for the design and detailing of boundary elevations, available from BCSA (link at end of article). David Brown of the Steel Construction Institute explains the structural engineering in the new guidance.

Single-storey buildings and boundaries

In England and Wales, (and similarly in Scotland and Northern Ireland) Approved Document B (ADB) implements the requirements of the Building Regulations and has a special section addressing single-storey buildings. Because the Building Regulations are mostly concerned about loss of life, there is generally no need for fire resistance of the structural frame in a single-storey building. The Building Regulations *do* want to stop the spread of fire, so if the structure is defined as being near a boundary (the definition depends on several factors), the elevation becomes a "boundary elevation" and must be constructed to prevent fire spreading to a neighbouring structure. The requirement is generally to stop fire spreading out from the inside of the structure, but if the boundary is very close, the elevation must also have fire resistance from the outside.

ADB refers to SCI publication P313, where the concept to provide a boundary elevation is provided by the following features as illustrated in Figure 1:

1. Cladding with appropriate fire resistance, tested to BS EN 1364-1 or BS 476-22 (note that reaction to fire in accordance with BS 476 will be removed from the England Building Regulations from March 2025 and the standard withdrawn entirely in 2029).
2. Primary columns which remain upright by:
   a. Protecting the column from base to eaves level, and
   b. Providing a moment resisting base.

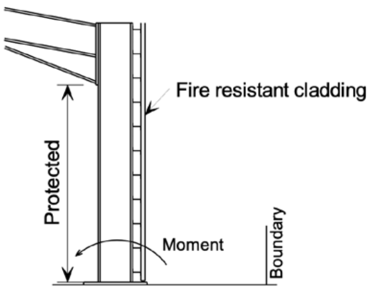


Figure 1: Usual boundary elevation provisions

In some cases the preference is to avoid a moment resisting foundation, which can be expensive. If a moment resisting base and foundation are not provided, the primary frame must be protected to prevent the boundary elevation column from collapsing. Guidance on the extent of the necessary protection was presented in *New Steel Construction*, July 2023.

In a fire, the unprotected rafters lose strength and drop into catenary, applying a force at the top of the column pulling *inwards*, which leads to the calculated overturning moment at the base.

Successful past practice

Despite heightened awareness of fire design and demands for analytically robust solutions, experience in the UK demonstrates that the provisions in P313 to prevent fire spreading to neighbouring properties have been successful. It should also be recognised that this performance is based on a number of engineering assumptions, including:

- The calculation of the overturning moment, which has some engineering basis, but is unlikely to be accurate;
- The entirely empirical assessment that a base moment of 10% the plastic moment of resistance of the column is appropriate for gable columns;
- The extrapolation of cladding performance from a typically 3m × 3m non-loadbearing test in accordance with BS EN 1364-1 or BS 476-22 to the panel sizes used in reality.

Challenged assumptions

In recent years, interest in all forms of fire performance has been heightened. For boundary elevations of single-storey buildings, questions have been asked of the secondary support steelwork – the side rails and their performance in the fire condition. The cladding will have been tested to BS 476-22 or BS EN 1364-1 and the primary steel column will be protected, but what of the light-gauge side rails? If the inside of the structure is assumed to be a compartment, then at the commonly required resistance period of 60 minutes, the temperature of the standard fire (specified in BS EN 1991-1-2) reaches 945°C. At this temperature, the cold-rolled steelwork has only 4% of its original strength (according to Table E.1 of BS EN 1993-1-2), which seems more of a mathematical curiosity rather than something to place undue reliance on.

Of course, there are many potential reasons why a theoretical approach is inappropriate:

- In real fires, the cladding generally remains attached;
- In real fires, purlins often remain in place, despite huge deformation;
- The temperature of 945°C may not be reached, perhaps due to venting through the roof;
- The lower side rails will inevitably be cooler and retain some strength – the temperature is unlikely to be uniform.

Equally, it could be argued that in some circumstances there may be a high fire load in the structure, the roof cladding may remain intact (no venting) and the temperatures reach those predicted by BS EN 1991-1-2. A solution must be put in place that will provide a reliable fire-resistant boundary.

Industry Group

A group of interested parties, who each have a contribution to the fire boundary, was established to prepare recommendations. The group included:

- Steelwork contractors, responsible for the main frames
- Secondary steelwork manufacturers, responsible for the light-gauge steelwork
- Cladding manufacturers, responsible for the integrity and insulation of the cladding (composite panels or built-up systems)
- BCSA and SCI

Whilst steelwork contractors, secondary steel manufacturers and cladding manufacturers will have their own areas of responsibility, a reliable solution requires input from all three parties – collaboration is required. The output from the industry group was to define the essential features of a robust solution with some flexibility over which party provides (and charges for) certain parts of the system. The coordination of the various contributions is a responsibility for the Principal Designer.

System features

The concept for the boundary elevation is simple. It is assumed that in the common fire condition preventing fire spread from the *inside* of the structure, the fire-resistant cladding is attached to and hangs like a curtain from a so-called "capable member", as illustrated in Figure 2. No reliance is placed on the unprotected side rails. Each part of the system is discussed below.

If the property boundary is very close to the structure, it may be necessary to consider fire spread from the *outside* of the structure. In these situations, the secondary steelwork is protected by the cladding and may be assumed to remain competent. It is unlikely that a boundary is *only* required to resist fire from the outside, so in most circumstances the prevention of fire spreading from the inside will dominate the boundary system requirements.

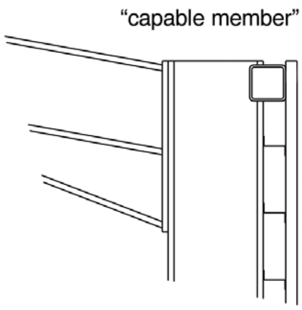


Figure 2: Proposed "capable member" to support cladding "curtain"

"Capable member"

The "capable member" is something at high level to which the cladding is attached. It must be designed to carry the vertical load of the cladding in the fire condition, as an accidental combination of actions. No other variable actions need including as part of the member verification. The capable member could be a hot-rolled member, or a cold-formed member.

Hot-rolled (or hot-finished members) will need to be designed in accordance with BS EN 1993-1-2, for a fire resistance period equal to that of the internal compartment (normally 60 minutes for single-storey buildings). The necessary protection depends on the member utilisation and A/V value for the member. This data must be communicated to the party responsible for specifying the fire protection – it is unacceptable to simply state "the member must be protected". Judicious member selection is important, as the protection of some member types can be practically impossible or prohibitively expensive.

It may be possible to demonstrate, either by physical testing or by analysis, that a cold-rolled member with appropriate protection has sufficient resistance in the fire condition to perform as a "capable member". The limiting temperature for the member must be communicated to the party responsible for providing the protection – it is unacceptable to simply state "the member must be protected".

The capable member will be positioned such that the outside face is on the sheeting line and therefore is almost certain to be supported by some steelwork from the main steel frames. The supports to the "capable member" must also be adequately protected as they are essential to maintain the resistance of the boundary.

In tall elevations, it may be necessary to introduce one or more intermediate "capable members" if the full height of the cladding cannot support itself from a single member.

In some cases, the cladding may be supported from the structure within a parapet, which may be used as the "capable member".

Cladding

The cladding will typically have been tested in a 3m × 3m test furnace, which is clearly not representative of its use in practice. No change is proposed to the assumption that the tested cladding remains equally capable in the large areas of cladding used in practice. No load-bearing tests are proposed to demonstrate that the cladding will support itself when hung from a "capable member" – instead, it is anticipated that cladding companies will demonstrate that in the fire condition, an adequate load path is maintained from the "capable member" into the cladding, and that the cladding is capable of hanging from that support for a specified height. The demonstration of cladding performance may involve some component testing, or analysis, or structural design and is expected to utilise the outer sheet (when the assumed fire is on the inside of the structure) as the main load-carrying component.

Vertically laid cladding

The cladding is to be attached to the "capable member". It should be shown that in the fire limit state the fixings to the capable member (usually screws) either maintain their resistance (since the interface is usually protected between the cladding and the capable member) or the fixings designed on a reduced resistance. In each case, the fixings must be appropriate for the weight of cladding. An adequate load path from the "capable member" to the vertical load-carrying elements of the cladding is required.

If the cladding is not continuous over the full height of the elevation, the joints must be shown to be capable of carrying the design vertical actions in the fire condition. The internal liner and fixings may be critical as they are exposed to the compartment fire. It may be possible to show that the load at joints can be carried by transfer to the outer sheet and its lap connections, or by bracketry within a built-up cladding system. If a joint cannot be detailed to be adequate, an additional capable member should be introduced to carry the weight of the lower cladding.

Horizontally laid cladding

Horizontally laid cladding is generally attached to vertical members, running between horizontal rails. The vertical members are not continuous, so if they are to be used to carry force to the "capable member" they must be verified at elevated temperature. The joints between vertical members where they are interrupted by the horizontal rails, and the more heavily loaded connection to the "capable member" must also be verified at elevated temperature.

At elevated temperature, the cladding must be shown to span between the vertical supports. It may be that the outer sheet provides adequate resistance. If cladding systems rely on internal bracketry in the fire limit state, an adequate load path to the capable member must be demonstrated.

Slotted side rail connections

For many years, some authorities have insisted that the conditions in a fire test - which generally have slots at the supports to allow expansion and contraction - are reproduced in practice and therefore insist that slots should be provided at the connections of the side rails to the primary steelwork. Both SCI and BCSA consider that the opportunity for side rails to buckle over their length (and thus accommodate expansion) means that slots are not required. In the normal design condition, side rails provide restraint to the column, so providing slots is detrimental to their performance.

The slot length generally provided is much less than the theoretical expansion. Where two side rails meet, the gap to allow expansion is generally in the order of 60mm, implying that the cladding, which is fixed to the side rails at intervals, can accommodate 60mm of crushing at the cleat locations as illustrated in Figure 3.

The recommendation from SCI and BCSA is that slots need not be provided at the connections between the side rails and the primary steelwork.

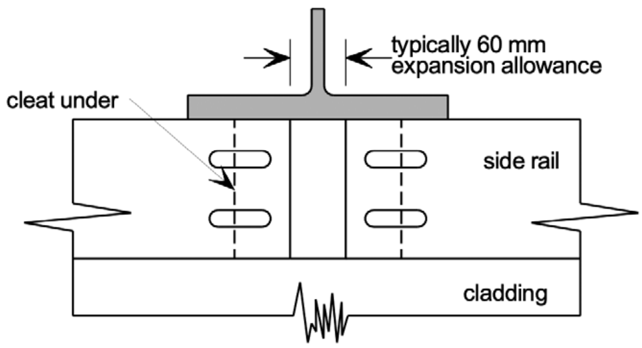


Figure 3: Provision for expansion at the side rail connections (but not cladding)

Allocation of responsibility and information exchange

Whilst the foregoing recommendations define the features of the system, the implementation of a competent boundary requires coordination by the Principal Designer. Responsibilities of the various parties are identified below, reflecting typical practical arrangements (which may differ between contracts).

Steelwork contractor

- Design of the primary steelwork (and sometimes the secondary steelwork);
- Design of the "capable member" if hot-rolled / hot-finished sections and communicating the member details to other parties;
- Performance specification for the fire protection of the primary steelwork and "capable member".

The steelwork contractor will need design loading from the cladding manufacturer and secondary steelwork manufacturer, with any requirements for intermediate “capable members”.

Secondary steelwork manufacturer

- Design of the secondary steelwork, including the performance of any load-carrying members assumed to act in the fire limit state;
- Design of the “capable member” if cold-formed and communicating the member details to other parties;
- Performance specification for the fire protection of the “capable member”.

The secondary steelwork manufacturer will need design loading from the cladding manufacturer, with any requirements for intermediate “capable members”.

Cladding manufacturer

- Justification of the cladding hanging as a curtain from a capable member;
- Justification of the fixings to the capable member and at laps (if any).

Main contractor

- The design of any moment-resisting foundations;
- The design and application of fire protection systems.

Protection of the primary steelwork

Although the new guidance primarily concerns the secondary steelwork and cladding, it is self-evident that the primary steelwork must also be adequately protected. The specification of adequate protection requires the calculation of a critical temperature, which will depend on the utilisation of the member and is determined by the designer of the structure. It should be noted that in the fire condition, the main columns may be highly utilised, since the calculated base moment already includes reduced partial factors to reflect the accidental limit state.

Conclusions

Although the recommendations of P313 appear to be adequate in practice, it is clear that the secondary steelwork – which supports the elevation cladding – cannot be verified for the usual fire resistance period of one hour, if the temperature within the structure follows the standard fire curve specified in BS EN 1991-1-2.

The new guidance proposes the engineering justification of a load path to ensure the cladding remains supported by the structure. The proposed solution requires collaboration between the main parties involved in construction, with the essential coordination the responsibility of the Principal Designer. ■

The technical specification for the design and detailing of boundary elevations is available from BCSA at *https://bcsa.org.uk/resources/fabrication-technical-design/industry-specifications/*

Design at elevated temperatures – unrestrained beams

Previous articles on the design of members at elevated temperatures[1,2], covered restrained beams and columns. This third article in the series by David Brown of the SCI covers the verification of unrestrained beams.

Introduction

Unrestrained beams are rather like columns – the simple reduction in design strength which was satisfactory for restrained members is not appropriate. For both unrestrained beams and unrestrained columns, buckling behaviour – which is non-linear even at ambient temperatures – is impacted by changes to the yield strength and changes to the modulus of elasticity. The overall process is to firstly calculate the reduced loading in the fire limit state, which was covered previously. The LTB resistance can be calculated at any given temperature, which must of course be greater than the design effects (the bending moment in the fire limit state). The critical temperature is when the design resistance just exceeds the design effects. This critical temperature together with the A_m/V ratio can be used by a fire protection company to specify the necessary thickness of their product.

Changes to the resistance calculation

The changes specified in BS EN 1993-1-2 are straightforward.

A revised non-dimensions slenderness $\bar{\lambda}_{LT,\theta,com}$ is required, given by

$$\bar{\lambda}_{LT,\theta,com} = \bar{\lambda}_{LT,}[k_{y,\theta,com}/k_{E,\theta,com}]^{0.5}$$

The values of $k_{y,\theta,com}$ and $k_{E,\theta,com}$ adjust the material strength and modulus of elasticity (Young’s Modulus) respectively and are taken from Table 3.1 of BS EN 1993-1-2.

Although many designers will know that the strength of steel does not reduce until after 400°C, the modulus of elasticity is modified as soon as the temperature reaches 200°C. This means that the adjustment factor $[k_{y,\theta,com}/k_{E,\theta,com}]^{0.5}$ is significant at relatively low temperatures. The relationship between the adjustment factor and temperature is shown in Figure 1. Since the adjustment factor is greater than 1.0, the slenderness is increased, leading to a more significant reduction factor.

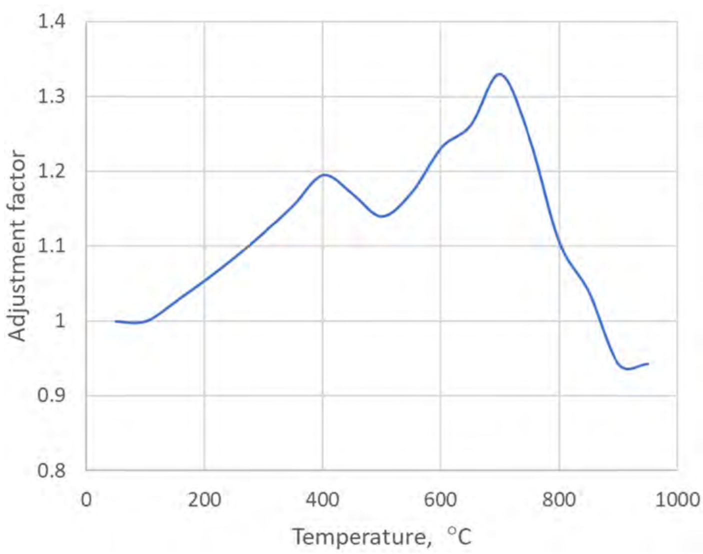


Figure 1: Variation of $[k_{y,\theta,com}/k_{E,\theta,com}]^{0.5}$ with temperature

The second change is that the imperfection factor α is no longer related to the cross section geometry, but is a fixed value given by:

$$\alpha = 0.65\sqrt{235/f_y}$$

The final changes are subtle alterations to the formulae to calculate the reduction factor $\chi_{LT,fi}$. In particular, there is no plateau length within the expressions, so the reduction applies even at low values of slenderness.

The final resistance is given by:

$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{pl,y} k_{y,\theta,com} f_y / \gamma_{M,fi}$

(for Class 1 and Class 2 sections, hence the use of W_{pl})

Due to the combined impact of the changes in the formulae, even at 20°C there is a marked reduction in the calculated LTB resistance. Table 1 shows the difference, for a 533 × 210 × 82 UB in S355 with a C_1 value of 1.0. In a perfect world one would have hoped the resistance at 20°C was the same as the "cold" value, but the differences in the formulae preclude this.

Table 1: Comparison of LTB resistance

Beam buckling length (m)	"Cold" Resistance (kNm)	"Hot" resistance (kNm) – at only 20°C
9	193	147
6	312	238
3	577	451

Design example

Verification at ambient temperature:

7m span beam, unrestrained, nominally pinned supports, subject to a UDL. The variable action is from office loading.

G_k = 3.0 kN/m²; Q_k = 3.3 kN/m². Beams spaced at 3.6m centres.

Using expression 6.10 from BS EN 1990, the design combination of actions is:

1.35 × 3.0 + 1.5 × 3.3 = 9.0 kN/m²

The design load on the beam = 9.0 × 3.6 = 32.4 kN/m and the maximum bending moment $= \dfrac{32.4 \times 7^2}{8} = 199$ kNm

Looking in the Blue Book, a 406 × 178 × 74 UB in S355 appears appropriate.

$M_{b,Rd}$ = 218 kNm, > 199 kNm, OK.

Deflection is unlikely to be a critical check with unrestrained beams, but for completeness is verified.

Characteristic variable load on beam = 3.3 × 3.6 = 11.9 kN/m

$$\delta = \frac{5 \times 11.9 \times 7000^4}{384 \times 210000 \times 27300 \times 10^4} = 6.5\text{mm}$$

$$\text{Allowable} = \frac{7000}{360} = 19.4, \text{OK}.$$

In preparation for the verification at elevated temperature, the non-dimensional slenderness $\bar{\lambda}_{LT}$ is required. The non-dimensional slenderness requires M_{cr}, which must be calculated using software or from a formula (for example as given in P362).

In this instance, the formula has been used, and M_{cr} = 253 kNm

$$\text{Then } \bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{1500 \times 10^3 \times 355}{253 \times 10^6}} = 1.45$$

(noting that the section is Class 1, so $W_y = W_{pl}$ and the flange is 16.0mm, so f_y = 355 N/mm²)

Verification at elevated temperature

Firstly, the value of the actions in the fire limit state is calculated using the expressions in BS EN 1993-1-2.

The reduction factor to be applied to the design loads, η_{fi}, is given by:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_k}{\gamma_G G_k + \gamma_Q Q_k} = \frac{3 + 0.5 \times 3.3}{1.35 \times 3 + 1.5 \times 3.3} = 0.52$$

The design effects in fire $E_{d,fi}$ are therefore $E_{d,fi} = \eta_{FI} E_d$ = 0.52 × 199 = 104 kNm.

The calculation steps follow those outlined above - calculate a revised value of α (for f_y = 355, α = 0.53) and then for each temperature:

- Calculate a revised value of the non-dimensional slenderness, $\bar{\lambda}_{LT,\theta,com}$
- Calculate a new reduction factor $\chi_{LT,fi}$
- Calculate a new buckling resistance $M_{b,fi,t,Rd}$ – remembering that the factor $k_{y,\theta,com}$ appears in the final calculation as well as the revised slenderness calculation.

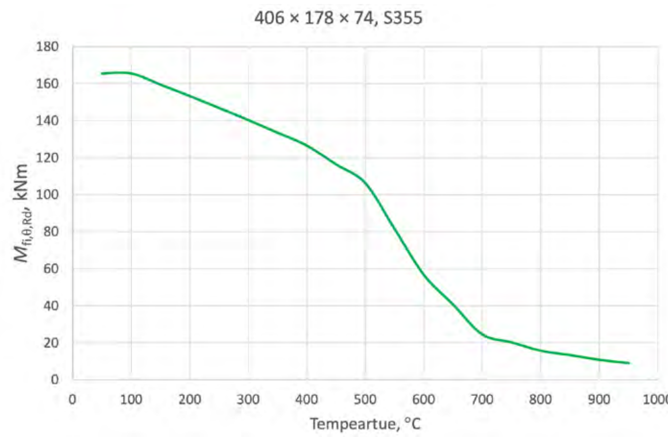


Figure 2: LTB resistance at elevated temperature

A spreadsheet will facilitate these calculations. For the selected beam, the relationship between the moment resistance $M_{b,fi,t,Rd}$ and temperature is shown in Figure 2.

From Figure 2, it can be seen that the design resistance falls below 104 kNm at around 500°C. The precise figure is 505°C, which is used in the following calculations.

Interpolating from Table 3.1 of BS EN 1993-1-2:

$k_{y,\theta}$ = 0.765

$k_{y,\theta}$ = 0.586

$$\text{Then } \bar{\lambda}_{LT,\theta,com} = \bar{\lambda}_{LT,}[k_{y,\theta,com}/k_{E,\theta,com}]^{0.5} = 1.45 \times \left(\frac{0.765}{0.586}\right)^2 = 1.66$$

With α = 0.53

$\phi_{LT,\theta,com}$ = 0.5 × [1 + 0.53 × 1.66 + 1.66²] = 2.32

and

$$\chi_{LT,fi} = \frac{1}{2.32\ \sqrt{2.32^2 - 1.66^2}} = 0.25$$

then

$M_{fi,t,Rd}$ = (0.25 × 1500 × 10³ × 0.765 × 355)/(1.0 × 10⁶) = 102 kNm

(or, with more precision in the intermediate values, 103.7 kNm).

Shear resistance

Just as deflection is unlikely to be critical with unrestrained beams, so is shear. The shear resistance at elevated temperatures is given by:

$V_{fi,t,Rd} = k_{y,\theta,web} V_{Rd}\ [\gamma_{M0}/\gamma_{M,fi}]$

Which uses the same value of $k_{y,\theta}$ calculated previously.

From the Blue Book, V_{Rd} = 858 kN

$V_{fi,t,Rd}$ = 0.765 × 858[1/1] = 656 kN

The design shear load in the fire condition is 0.52 × 32.4 × 7/2 = 59 kN, OK.

Conclusions

In this particular example, the critical temperature was 505°C. The unrestrained condition is intuitively more onerous than when the beam is restrained, so the tabulated critical temperatures for restrained beams (discussed in part 1) should not be used. In the fire condition, the example beam is "utilised" at 104/166 = 0.63. The resistance of 166 kNm is the LTB resistance at 20°C. Incorrectly using Table NA.1 of the UK NA to BS EN 1993-1-2, the critical temperature might be assessed as around 548°C, which is not adequate.

Table 18 of the ASFP Yellow Book includes "beams not carrying concrete floor slabs" and presents a limiting temperature of 585°C for offices, which is similarly not adequate. The tabulated values in these two documents, which are for restrained beams, should not be used for unrestrained beams.

The correct calculation process for unrestrained beams is not difficult and is readily facilitated in a spreadsheet. ■

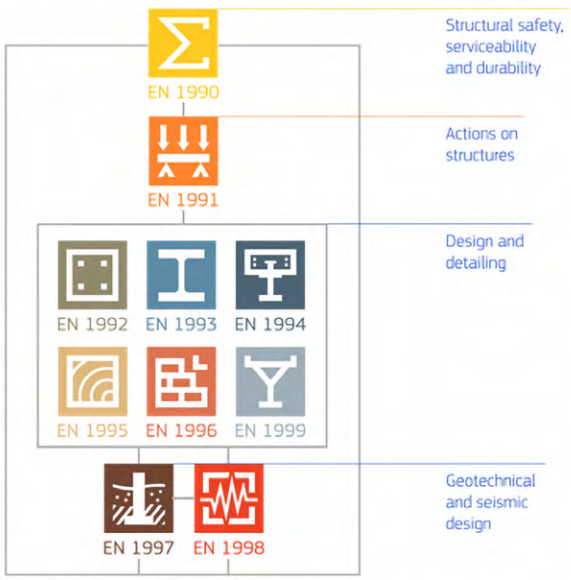
1. Fire protection of steelwork, NSC, March 2024
2. Critical temperatures for fire design: Part 2 – Columns, NSC, April 2024

What's changing in the updated FprEN 1993 Eurocode 3 – Design of Steel Structures?

A European Design Standard, with country specific annexes and design guides

Eurocode 3 provides both general and structure specific recommendations for the design of steel structures that can be used by design engineers, fabricators and manufacturers to create safe, durable, and sustainable steel structures. It was first published between 2002 and 2007 to enable the design of building and civil engineering works, and to determine the performance of structural construction products.

It has been adopted throughout Europe as the design standard for steel structures. It provides a common set of design rules to be used with a country's National Annex. There have been significant advances in research, product performance and state-of-the-art practices, hence the review and updates which will be rolled out in the forthcoming years.



When will the second generation be published and mandated?

They are due to be published by the end of September 2027, with a period of coexistence where the first generation of Eurocodes are current and the second generation are available, but not implementable until the date of withdrawal in March 2028.

General changes in Eurocode 3

The most general changes to the second generation of Eurocode 3 are:

- A revision of the table of contents, which means that designers will have to relearn where things are that they need to reference for design calculations
- An extension of the scope to steel grades up to S700
- EN 1993-1-12 will now include additional rules for steel grades up to S960
- Clarification on the use of verbs to indicate how rigorously a clause should be applied by designers when using the recommendations given in the standards
- There are two new parts. EN 1993-1-13 provides rules for beams with large web openings and EN 1993-1-14 outlines a common approach for the design of steel structures designed using finite element analysis

Some of the most important specific changes

EN 1993-1-1: Design of Steel Structures. Part 1-1 – General Rules and Rules for Buildings

EN 1993-1-1 deals with the structural design of individual components such as beams, columns, and the design of whole structures. It includes recommendations on the types of steel to be used and the material properties that should be used in the design.

The revised version of EN 1993-1-1 includes an extension of the scope to allow steel grades up to S700 to be used. As a result, the ductility recommendations need to be revised to reflect the reduced ductility of higher strength steels, particularly when considering the design resistance of a section with holes.

The revision also includes a new method for determining the lateral-torsional buckling of beams.

Other changes include; the design of elliptical hollow sections, the methods for structural analysis have been refined and summarised in a flowchart, a new method for the design of semi-compact sections (Class 3), an improvement in the effects of torsion on the resistance of cross-sections and members, a simplified design approach for fatigue, an annex providing statistical data on material and dimensional properties that were used for the calibration of the default partial factors.

EN 1993-1-2: Design of Steel Structures. Part 1-2 – Structural Fire Design

EN 1993-1-2 deals with the design of steel structures for the accidental situation of fire exposure with reference to the load bearing function and only identifies differences from, or supplements to, normal temperature design. It is only concerned with passive forms of fire protection and also covers cold-formed members.

In revised EN 1993-1-2, nominal fires are applicable to steel grades up to and including S700. However, physically based thermal actions are only applicable to steel grades up to and including S500.

EN 1993-1-3: Design of Steel Structures. Part 1-3 – Cold Formed Sections and Sheeting

EN 1993-1-3 deals with the design of cold-formed sections and sheeting.

The list of steel grades given in EN 1993-1-3 has been expanded.

Other changes include: rules added for the design of sinusoidal sheeting, the design of trapezoidal sheeting in axial compression and the bending moment resistance of liner trays. Clarification on the design formulae for cross-sectional resistance of sections in combined axial force, bending moment, shear force and torsion and of the design provisions at serviceability limit states. Minor specifications and explanations added for the buckling design of sections in combined compression and bending. New and special provisions for the design of trapezoidal sheeting with overlaps and special provisions for fasteners made of stainless steel in relation to the corrosion environment deleted.

EN 1993-1-5: Design of Steel Structures. Part 1-5 – Plated Structural Elements

EN 1993-1-5 provides design rules for stiffened or unstiffened steel plates that are subject to forces applied within the plane of the plate. It covers structural elements such as I section girders, box sections and plated components used in tanks and silos.

The scope of the standard has been extended to cover non-rectangular panels.

Another change concerns the resistance of steel plate girders subjected to patch loading, with a new calculation for the reduction factor on the design resistance, recommended in Clause 6.4(1).

EN 1993-1-8: Design of Steel Structures. Part 1-8 – Joints

EN 1993-1-8 advises on the design of steel joints. This includes bolted joints, such as end plates, fin plates, and welded joints. It also covers tubular joints.

The revised standard has been extended to include the design of nominally pinned connections, with recommendations given in Annex C.

The standard also includes a new Annex D for the design of column bases with fasteners between steel and concrete.

Want to find out more about the changes to the second generation FprEN 1993 Eurocode 3 – Design of Steel Structures?

The BCSA hosted a webinar on this subject, presented by Dr Ana Girão Coelho on 30 April 2025.

The presentation included details on the evolution of the Eurocodes, where they are now, what to expect in the second generation, and where we are at in the process ahead of the withdrawal of the current Eurocodes in March 2028.

Dr Ana Girão Coelho gave a brief overview and answered questions at the end of the presentation.

To watch the webinar visit https://www.youtube.com/watch?v=FB0eQunhjTs

Fire protection guidance from ASFP

The Association for Specialist Fire Protection (ASFP) has recently published the 6th Edition of the 'Yellow Book', Volume 1. David Brown of the Steel Construction Institute reviews the updated guidance.

The "Yellow Book", Volume 1

ASFP has for many years published guidance on various aspects of fire performance. Volume 1 of the "Yellow Book" covers fire protection for structural steelwork, providing background information and recommended values of critical temperature in different situations. Fire protection must be provided to ensure the critical temperature is not exceeded at the specified period of fire resistance.

In January 2025, the 6th Edition of Volume 1 was published. Sadly, the cover (Figure 1) is no longer yellow. Although SCI's name appears inside the front cover (with the former office address), SCI did not contribute to the revised edition and was not invited to comment as the revised guidance was developed.

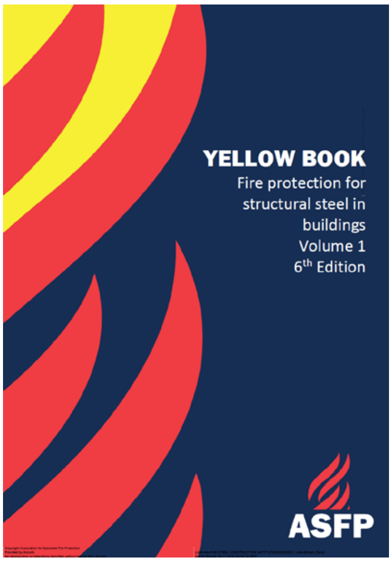


Figure 1: Cover of Yellow Book, 6th edition

Default critical temperatures

One important change in the 6th Edition is the consolidation of the critical temperatures presented in previous versions into one simple table. The new guidance is shown below (reproduced exactly as presented in Table 7 in the revised edition).

Simple Description	Structural Use / Description of Member	Default critical temperature (°C)
3-sided beam	I section and Hollow section beams in bending supporting concrete slabs or composite slabs.	580
4- sided beam	I section and hollow section beams in bending not supporting concrete slabs	550
Hangers and tension braces	Members in tension only of any shape	550
Columns (including Hollow Columns), trusses and other bracings	Compression members of any shape	500

An important introduction to the table is that "the appropriate design load level for the fire scenario is taken to be no less than 0.6 × the capacity of the member at ambient temperature".

The statement *should* read that "the load level is taken to be *no more* than 0.6 × the capacity...", as more highly utilised members require the temperature to be limited to a lower value.

The identical table reappears in Annex B.2, where the introduction to the table states (the temperatures) "have been compiled based upon structural loading limitations to utilisation (in fire) values of no greater than μ_0 = 0.6 in all scenarios". Hopefully, the mismatch between the two statements will be corrected in a future revision.

The utilisation of a member depends on the ratio

$$\eta_{fi} = \frac{\text{design actions in fire}}{\text{design actions in normal conditions}}$$

and how hard the member was working at ambient temperature. In many cases, members are not working at 100% of their resistance at ambient temperature, which reduces their effective utilisation in the fire limit state – they already had "spare" resistance.

The value of η_{fi} was discussed in a previous article[1], but is worth repeating here, as illustrated in Figure 2. The values of ψ_1 = 0.9, 0.7 and 0.5 relate to storage, shopping and office categories of loading respectively. The ASFP limiting value of η_{fi} = 0.6 is also shown.

If the utilisation were *more* than 0.6 in the fire limit state, the limiting temperature tabulated by ASFP would not be conservative. Figure 2 indicates that the value of η_{fi} for storage category of load (ψ_1 = 0.9) is always greater than 0.6. In these situations, the ASFP guidance is not conservative unless the beam had "spare" resistance in the normal condition. Note 2 to Figure 2.1 in BS EN 1993-1-2 recommends a maximum value of η_{fi} = 0.65 usually and η_{fi} = 0.7 for storage classification, so the 0.6 assumed by ASFP is a little optimistic.

If Q_k/G_k = 1 in the storage category, η_{fi} = 0.667 (if the original design values of actions had used expressions 6.10a and 6.10b from BS EN 1990).

If the utilisation was 0.667, and the beam is exposed on four sides (what the ASFP table refers to as a "four-sided beam") the resulting critical temperature is 535°C (from equation 4.22 of BS EN 1993-1-2), somewhat more onerous than the ASFP table. It must be pointed out that if the member in the preceding example was only working at 90% of its resistance in the normal condition, the tabulated critical temperature would be satisfactory. In most practical cases therefore, the limiting temperatures provided by ASFP for beams will be satisfactory.

From Figure 2, it will be seen that the assumed utilisation = 0.6 is conservative in very many usual cases. If Q_k/G_k = 1.5 in the office category, η_{fi} = 0.5. If the beam was working at 85% of its resistance in the normal condition, the utilisation becomes 0.425 and the critical temperature increases from 550°C tabulated by ASFP to 610°C, which requires less protection.

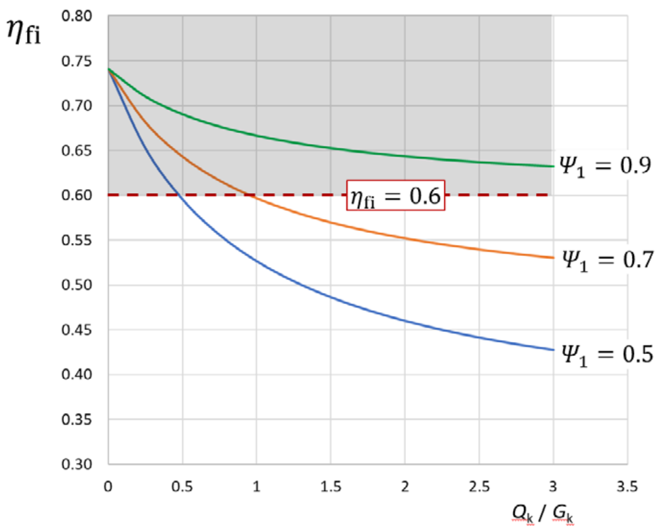


Figure 2: Reduction factor η_{fi}

These two examples should remind designers that, rather than adopting default values, best practice is to calculate the critical temperature, which is not difficult. One very important point is that even though Table 7 of the 6th Edition describes "4-sided Beam" (which is a member exposed to fire on four sides) and indicates the structural use as "beams not supporting concrete slabs", the beams are still fully restrained. The critical temperatures in Table 7 are not appropriate for unrestrained beams, which is unfortunately not clarified in the document. The structural resistance of unrestrained beams requires a more involved assessment[2].

Members in tension

The calculation for members in tension is straightforward, as BS EN 1993-1-2 states that the tension resistance $N_{fi,\theta,Rd}$ is given by:

$N_{fi,\theta,Rd} = k_{y,\theta} N_{Rd} [\gamma_{M,0} / \gamma_{M,fi}]$

Since the partial factors are currently both 1.0 in the relevant UK National Annex, the resistance is simply the resistance at ambient, multiplied by the reduction factor, $k_{y,\theta}$ from Table 3.1 of BS EN 1993-1-2.

If the reduction factor is limited to 0.6 (so a 40% loss of strength) the critical temperature is 558°C by interpolation of Table 3.1 in BS EN 1993-1-2, which appears in the ASFP table as 550°C, so perhaps some modest rounding down. If Equation 4.22 is used, the critical temperature is calculated to be 554°C. In common with beams and columns, calculating the actual utilisation in fire and using any "spare" resistance at ambient could be useful in reducing the protection needed.

Compression members

The final category in the ASFP table covers "Columns (including hollow sections), trusses and other bracings", but also "compression members of any shape", with a critical temperature of 500°C.

In the 5th Edition, the tabulated critical temperatures for columns in compression were appropriate for columns in multi-storey buildings and used a buckling length equal to 0.7*L* (*L* being the storey height)[3]. This reduced buckling length is permitted by the code and 0.7*L* is a conservative choice – for anything other than the top storey the buckling length may be taken as 0.5*L*.

In the 6th Edition, the critical temperatures are stated to be based only on limiting the utilisation to no more than 0.6 in the fire condition, neglecting any adjustment of the buckling length. The basis for the ASFP values for "compression members of any shape" is the UK National Annex to BS EN 1993-1-2 where for a utilisation of 0.6, the lowest (most onerous) critical temperature for all tabulated values of non-dimensional slenderness is given as 500°C. ASFP have – quite reasonably – adopted this most onerous value for their stated utilisation. Unfortunately, Table NA.1 in the UK NA has a limited scope of application which is not transparent and therefore impacts the ASFP guidance.

UK NA critical temperatures for compression members

The critical temperatures presented in Table NA.1 assume a fixed value of the imperfection factor α as 0.49, selected from buckling curve 'c' in Table 6.2 of BS EN 1993-1-1. The value of $\alpha = 0.49$ is appropriate for UC sections buckling about their minor axis – which is probably the common situation in multi-storey buildings. If the imperfection factor is less than 0.49, the tabulated critical temperatures are not conservative. Hot finished hollow sections in S355 are buckling curve 'a', which can result in a considerably lower critical temperature.

Table 1 illustrates a comparison between a UC section and a hot finished SHS. The buckling lengths have been selected so that the non-dimensional slenderness is 1.2 for each section. In each case the temperature when the buckling resistance reduces to 60% of the ambient resistance is examined.

Table 1 demonstrates that whilst for UC sections buckling in the minor axis the UK NA tabulated values are conservative, this is not the case for other sections.

Table 2 illustrates the critical temperatures for the ASFP stated utilisation of 0.6 for different imperfection factors. The value highlighted in green is the critical temperature adopted in the ASFP guidance. The values highlighted in orange are the two examples considered in Table 1. Any temperature in Table 2 less than 500°C indicates an unconservative situation.

Table 2: Critical temperatures for 0.6 utilisation

	UK NA and ASFP		
Non-dimensional slenderness	$\alpha = 0.49$	$\alpha = 0.34$	$\alpha = 0.21$
$\lambda = 0.4$	526	519	513
$\lambda = 0.6$	518	504	482
$\lambda = 0.8$	510	481	436
$\lambda = 1.0$	505	462	403
$\lambda = 1.2$	502	454	391
$\lambda = 1.4$	500	453	399
$\lambda = 1.6$	500	456	408

Common situations where the UK NA tabulated values are not conservative include:

- UC sections in S460;
- UC sections in S355 where due to the arrangement of restraints the critical buckling mode is in the major axis;
- Hot finished hollow sections in S355, S420 and S460 ;
- UB sections.

A further complication is that the tabulated critical temperatures are appropriate for Class 1, 2 and 3 cross sections only. Many UB sections are Class 4 in pure compression and more sections become Class 4 at elevated temperature (because ε is modified). Clause 4.2.3.3 of BS EN 1993-1-2 recommends a critical temperature of 350°C for Class 4 sections. ASFP properly exclude Class 4 sections from the scope of their tabulated critical temperatures.

Recommendations by the ASFP

Section B.2 of the 6th Edition recommends that "the critical temperature should be fully evaluated by a detailed engineering calculation by a suitably qualified structural engineer". The SCI endorses that sound advice. An AD will be issued addressing the limitations of the UK NA and the ASFP guidance. ■

1 Fire protection of steelwork, NSC, March 2024

2 Design at elevated temperatures – unrestrained beams, NSC, April 2025

3 Critical temperatures for fire design: Part 2 – Columns, NSC, April 2024

Table1: Comparison of critical temperatures

	203 UC 60, S355	180 × 180 × 16 SHS, S355
Length	4765 mm	6090 mm
Area	7640 mm²	10200 mm²
Non-dimensional slenderness λ	1.2	1.2
Ambient $N_{b,Rd}$	1176 kN	1919 kN
Utilisation	0.6	0.6
Target $N_{b,fi,t,Rd}$	0.6 × 1176 = 706 kN	0.6 × 1919 = 1151 kN
Temperature	502°C	393°C
$k_{y,\theta}$	0.773	1.0
$k_{E,\theta}$	0.593	0.707
λ_θ	$1.2 \times \left(\frac{0.773}{0.593}\right)^{0.5} = 1.370$	$1.2 \times \left(\frac{1.0}{0.707}\right)^{0.5} = 1.472$
α	$0.65 \times \sqrt{\frac{235}{355}} = 0.528$	$0.65 \times \sqrt{\frac{235}{355}} = 0.528$
ϕ_θ	$0.5 \times (1 + 0.528 \times 1.37 + 1.37^2) = 1.800$	$0.5 \times (1 + 0.528 \times 1.427 + 1.427^2) = 1.895$
χ_{fi}	$\frac{1}{1.800 + \sqrt{1.800^2 - 1.370^2}} = 0.337$	$\frac{1}{1.895 + \sqrt{1.895^2 - 1.472^2}} = 0.318$
$N_{b,fi,t,Rd}$	$\frac{0.337 \times 7640 \times 0.773 \times 355}{1 \times 10^3} = 706$ kN	$\frac{0.318 \times 10200 \times 1.0 \times 355}{1 \times 10^3} = 1151$ kN
Conclusion	Critical temperature is 502°C	Critical temperature is 393°C

Calculation of α_{cr} for unbraced frames

In this article, Dr. Yigit Ozcelik of the Steel Construction Institute (SCI) presents a simple yet efficient hand method to estimate the global stability parameter, α_{cr}, for unbraced frames.

Introduction

In accordance with BS EN 1993-1-1[1] Clause 5.2.1, either a first-order or second-order analysis can be used to determine internal forces and moments, providing the criteria for the chosen method are satisfied. For first-order elastic analysis, the criterion requires that the factor, α_{cr} be greater than or equal to 10. This factor represents the multiplier by which the design loading would need to be increased to cause elastic instability in a global mode (see Equation (1)). If α_{cr} falls between 3 and 10, second-order effects may be considered using an approximate second-order analysis. However, for structures where α_{cr} is less than 3, a rigorous second-order analysis is required.

$$\alpha_{cr} = \frac{F_{cr}}{F_{ed}} \qquad (1)$$

where F_{Ed} is the design loading on the structure

F_{cr} is the elastic critical buckling load for global instability mode based on initial elastic stiffness

To calculate F_{cr} (and α_{cr}) precisely, a linear buckling analysis is normally needed; however, BS EN 1993-1-1[1] introduces an approximate method to estimate α_{cr} on a storey-by-storey basis within a building:

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) \qquad (2)$$

where H_{Ed} is the total design horizontal load transferred by the storey

V_{Ed} is the total design vertical load on the frame transferred by the storey

$\delta_{H,Ed}$ is the horizontal displacement at the top of the storey relative to the bottom of the storey when the frame is loaded with horizontal loads

h is the storey height

Similar to the linear buckling analysis, the approximate method also requires the use of structural analysis software to calculate horizontal displacements at storey levels. While such software is indispensable in modern engineering practice, a lack of understanding of its underlying assumptions can lead to erroneous results. Therefore, hand calculations remain a valuable practice to verify software output. In this article, a simple hand method based on first principles is introduced to calculate α_{cr} for unbraced frames.

Background on elastic critical buckling load

The elastic critical buckling load, N_{cr}, is defined as the compressive load at which an elastic column will suddenly bend and buckle.

$$N_{cr} = \frac{\pi^2 EI}{L^2} \qquad (3)$$

where E is the modulus of elasticity

I is the second moment of area

L is the length

Equation (3) was derived by Leonhard Euler in 1744, writing the equations of equilibrium of a pin-ended column in the deformed configuration, using the Euler-Bernoulli beam theory, which describes the relationship between deflection and applied load.

The effective length factor, K, commonly referred to as the K-factor, is a multiplier that enables the calculation of an artificial column length that allows the use of Euler's equation to evaluate the elastic critical buckling load of a column with relatively general support conditions (Figure 1). This leads to the general form of Euler's formula:

$$N_{cr} = \frac{\pi^2 EI}{(KL)^2} \qquad (4)$$

K-factors were determined for idealised end conditions such as pinned–pinned, fixed–fixed, pinned–fixed, and fixed–free, and are widely available in literature. However, these ideal cases have limited practical value in real-world applications, where support conditions and stiffness distributions are more complex.

For braced frames, a conservative design approach typically assumes K=1 for most situations. In practice, K<1.0 can be achieved in systems with very high lateral stiffness, but the use of unity is often recommended for simplicity and safety.

In contrast, determining appropriate K-factors for unbraced frames is more complex. In such cases, the K-factor can theoretically vary from 1.0 up to infinity, depending on the degree of rotational restraint provided by the surrounding frame. As a result, no universally applicable approach exists.

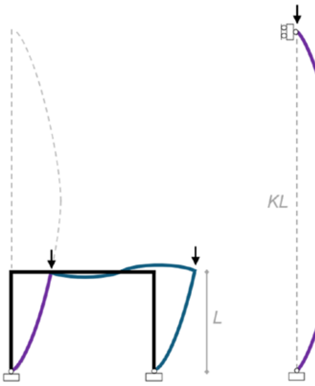


Figure 1: Column length (L) vs column effective buckling length (KL)

One approach to determining K-factors is the alignment chart that is a well-established graphical tool widely used by engineers. There are two nomographs available — one for braced frames and one for unbraced frames. The nomograph applicable to unbraced frames is shown in Figure 2.

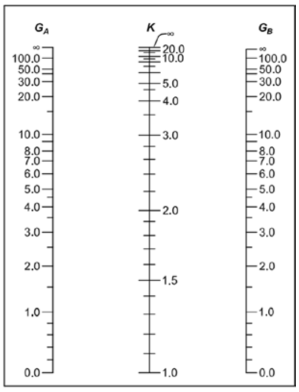


Figure 2: Alignment chart – unbraced frames

To use the nomograph, the degree of restraint at both ends of a column — denoted as G — must first be calculated using Equation (5):

$$G = \frac{\Sigma(I_c/L_c)}{\Sigma I_b/L_b} \qquad (5)$$

where $\sum(I_c/L_c)$ is the sum of the ratio of the second moment of area to the length of all columns connected to the joint

$\sum(I_b/L_b)$ is the sum of the same ratio for all beams connected to the joint

As an alternative to the graphical nomograph, the following closed-formed equation may be used to calculate K-factors for unbraced frames:

$$\frac{G_A G_B\left(\frac{\pi}{K}\right)^2 - 36}{6(G_A G_B)} - \frac{\left(\frac{\pi}{K}\right)}{\tan\left(\frac{\pi}{K}\right)} = 0 \qquad (6)$$

where G_A is the degree of restraint at one end of the column (see Equation (5))

G_B is the degree of restraint at the other end of the column (see Equation (5))

It is important to recognise that the alignment chart is derived from an elastic sidesway stability analysis of a highly idealised frame under simplified loading conditions. These assumptions, along with the modifications to the alignment chart, for unbraced frames will be explored in a forthcoming article by SCI.

Worked example 1

In this example, an unbraced frame subjected to two equal vertical point loads acting at beam-column joints was evaluated to determine the critical vertical load, N, that leads to instability of the frame.

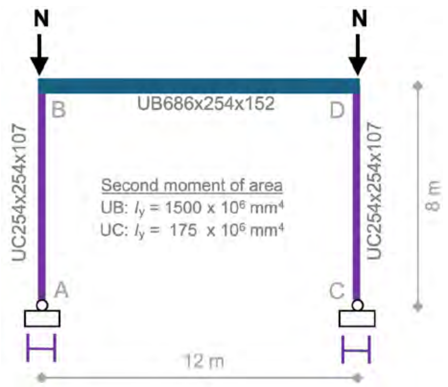


Figure 3: Worked example 1

The degree of restraint for Column AB at Point B, G_B, is:

$$G_B = \left(\frac{\Sigma I_c/L_c}{\Sigma I_b/L_b}\right)_B = \frac{175 \times 10^6\text{mm}^4 / 8\text{m}}{1500 \times 10^6\text{mm}^4 / 12\text{m}} = 0.175 \qquad (7)$$

where I_c is the second moment of area of Column AB
L_c is the length of Column AB
I_b is the second moment of area of Beam BD
L_b is the length of Beam BD

Due to the pinned base, the degree of restraint for Column AB at the column base (Point A), G_A, is infinity.

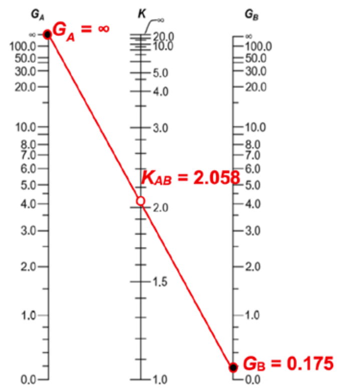


Figure 4: Effective length factor for Column AB

Entering G_A and G_B into the alignment chart, the effective length factor for Column AB, K_{AB}, is 2.058.

Using Equation (4), the elastic critical buckling load for Column AB, $N_{cr,AB}$ is:

$$N_{cr,AB} = \frac{\pi^2 EI_c}{(K_{AB}L_c)^2} = \frac{\pi^2\,(210\text{kN/mm}^2)\,(175 \times 10^6\text{mm}^4)}{(2.058 \times 8000\text{mm})^2} = 1338\text{ kN} \qquad (8)$$

Accordingly, N=1338 kN.

The unbraced frame was also analysed using MASTAN2, a free structural analysis program capable of performing linear buckling analysis. The results of the analysis yielded a critical vertical load of N=1335 kN, which suggests the simple hand calculation provided an accurate prediction of the critical load, closely matching the numerical results.

Worked example 2

In this example, the unbraced frame considered in the Worked Example 1 was modified to the extent that the load distribution among the columns is different, while the total load acting on the frame remains the same.

As the alignment chart used to determine the K-factor does not account for individual column loads, the K-factor remains unchanged. Consequently, $N_{cr,AB}$ = 1338 kN also remains unchanged.

Given that the elastic critical buckling load for Column CD was calculated using the alignment chart, $N_{cr,CD}$, is equal to $N_{cr,AB}$, one might argue that Column CD would buckle first, as it is

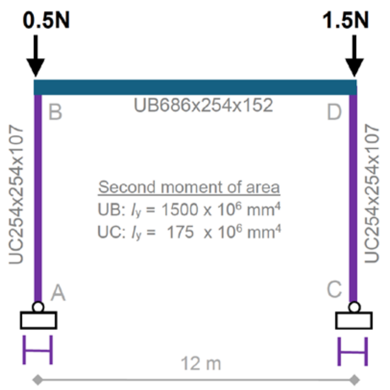


Figure 4: Worked example 2

subjected to a larger vertical load than Column AB. This would suggest that N should be lower than in the Worked Example 1. However, the linear buckling analysis of the frame with modified loads yielded the same critical vertical load: N=1335 kN.

This outcome can be explained by the fact that, when Column CD is onset of buckling, Column AB – being subjected to a smaller vertical load – still has reserve load-carrying capacity. This reserve capacity contributes to the overall stability of the frame by effectively *helping* Column CD to resist a larger load than its $N_{cr,CD}$ value. This phenomenon is known as the *ΣP Concept*[3], which describes how, in sway buckling, *some* columns *help others* while *others* reduce the capacity of *some*, until all columns buckle together in a global sway mode. Therefore, it is not suitable to assess the sidesway stability of columns in isolation; rather, the stability of the entire storey in the sway mode must be evaluated.

According to the results of the linear buckling analysis, the critical vertical load of the frame (or storey) is $2N$=2670 kN. Using this value, the effective length factors of Column AB and Column CD, (K_{AB} and K_{CD}, respectively) were back-calculated:

$$K_{AB} = \sqrt{\frac{\pi^2 EI_c}{0.5NI_c^2}} = \sqrt{\frac{\pi^2\,(210\text{kN/mm}^2)(175 \times 10^6\text{mm}^4)}{(0.5 \times 1335\text{kN})(8000\text{mm})^2}} = 2.914 \qquad (9)$$

$$K_{CD} = \sqrt{\frac{\pi^2 EI_c}{1.5NI_c^2}} = \sqrt{\frac{\pi^2\,(210\text{kN/mm}^2)(175 \times 10^6\text{mm}^4)}{(1.5 \times 1335\text{kN})(8000\text{mm})^2}} = 1.618 \qquad (10)$$

Notably, the K-factor determined from the alignment chart in the Worked Example 1 differs significantly from the values obtained in Equations (9) and (10). However, the elastic critical buckling load of the frame (or storey), $N_{cr,storey}$ – calculated as the sum of the elastic critical buckling load of each column estimated using the alignment chart according to the ΣP concept – matches the result from the linear buckling analysis. This leads to an important conclusion: the elastic buckling load of an individual column in an unbraced frame determined using an alignment chart K-factor, should be interpreted not as the maximum load that column can support, but rather as its contribution to the overall storey's buckling stiffness. Hence, $N_{cr,storey}$ can be accurately estimated using the alignment charts even if the K-factors for individual columns are not accurate:

$$N_{cr,storey} = \sum N_{cr,i} \qquad (11)$$

where $N_{cr,i}$ is the elastic critical buckling load of Column i using the alignment chart K-factor

However, it is important to note that the restraint (or *help*) provided by *some* columns to *others* is limited by the elastic buckling resistance of *other* columns in the no-sway mode – that is, assuming K = 1.0. In other words, each column must be able to support its own vertical load in isolation in the no-sway mode, without relying on the help. It is worth mentioning that elastic buckling of a column (which is part of a stability system) in the no-sway mode is quite unlikely for orthodox frame configurations.

Similar to the approximate method given in BS EN 1993-1-1[1] (see Equation (2)), α_{cr} can be calculated on a storey-by-storey basis within a building:

$$\alpha_{cr} = \frac{N_{cr,storey}}{V_{Ed}} \qquad (12)$$

Conclusion

In this article, a simple hand method is presented for calculating the global stability parameter, α_{cr}, of unbraced frames based on the fundamentals of the stability theory and effective length factors obtained from the alignment chart. The method allows engineers to estimate α_{cr} without relying on structural analysis software.

Through two worked examples, it was shown that the elastic critical buckling load of a storey for global instability mode – and therefore the calculated α_{cr} – remains accurate despite observing that the elastic critical buckling load of individual columns of the storey calculated using the alignment chart might be incorrect.

The method enables accurate estimation of α_{cr} and offers a valuable verification tool for engineers. ■

1 British Standards Institution. (2005). *BS EN 1993-1-1:2005 - Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings*. BSI.
2 Ziemian, R. D., McGuire, W., & Liu, S. (2015). *MASTAN2: Interactive structural analysis program*. Retrieved from https://www.mastan2.com/
3 Yura, J. (2011). *Five useful stability concepts* [PDF]. American Institute of Steel Construction. Retrieved from https://www.aisc.org/globalassets/continuing-education/quiz-handouts/five-useful-stability-concepts-handouts_2-per-bw.pdf

New SCI publication on steel castings

Modern steel construction continually pushes boundaries, demanding greater complexity and enhanced aesthetic appeal, while maintaining or even increasing material efficiency. As architects and engineers envision more ambitious structures, the limitations of traditional fabrication methods can sometimes present challenges at the joints between members. Structural steel castings offer a viable alternative to conventional fabrication for such connections and components, but they remain an often underutilised solution. The Steel Construction Institute's (SCI) new Publication P441, *Structural Steel Castings*, provides a comprehensive technical resource for engineers, designers, and fabricators. This publication represents a significant update to the earlier SCI guide P172 *Castings in Construction*, reflecting almost three decades of advances in analysis, manufacturing, and application. Max Cooper of the SCI offers an overview of key guidance within P441, examining the engineering advantages, practical applications, procurement considerations, and quality management associated with structural steel castings.

Engineering Advantages of Structural Steel Castings

SCI Publication P441 extensively details the engineering merits of employing steel castings, which can yield significant benefits in terms of structural performance, design optimisation, and construction efficiency.

Optimised Geometries and Material Utilisation

The casting process allows the creation of components with highly complex, three-dimensional geometries that are challenging or unachievable through standard fabrication techniques. This allows for the optimal shaping of connections to align with stress paths, for instance, by gradually tapering wall thicknesses to meet varying load demands or transitioning cross-sectional shapes to match the incoming members. Such optimisation can minimise stress concentrations and improve fatigue performance. Material can be distributed precisely where structurally required, enabling efficient load transfer and potentially reducing overall component weight compared to fabricated alternatives built up from standard sections and plates. This is particularly evident where multiple members with different profiles or orientations converge at a single point.

Figure 1: Example of cast steel base connection at Charlotte Douglas International Airport
© CAST CONNEX

Enhanced Structural Characteristics

Cast steel components typically exhibit isotropic material properties, providing uniform strength and ductility in all directions. This is particularly advantageous for connections subjected to multi-axial stress states where the directional properties of wrought materials might be a limiting factor. The monolithic nature of castings eliminates bolted or welded joints within the component itself, which can be sources of stress concentration or points of initiation for fatigue cracking. Well-designed castings, often benefiting from smooth fillets and gradual changes in section, can offer superior fatigue resistance compared to equivalent fabricated connections, leading to extended service life for the structure.

Fabrication Simplification and Construction Precision

The use of castings can simplify the fabrication of complex nodes by reducing the number of individual pieces and the extent of complex welding. Welds are typically relocated from highly stressed zones within the node to simpler, more accessible interfaces between the casting and adjoining steel members. This not only improves weld quality but can also reduce inspection burdens.

Dimensional accuracy of critical interfaces, such as mating surfaces or bolt hole locations, can be achieved through post-cast machining, often utilising CNC processes to tight tolerances. This precision leads to improved fit-up on site, reduced erection times, and lower risks of costly rework. Features such as integrated lifting lugs, temporary erection supports, or specific weld preparations can also be incorporated directly into the casting design, further streamlining site operations.

Understanding Steel Castings: The Process and Product

At its core, a steel casting is a component formed to a near-net shape by pouring molten steel into a mould containing a cavity of the desired geometry. Once the metal cools and solidifies, the mould is removed, and the casting undergoes various finishing processes. While different casting methods exist, sand casting is the most common for structural steel applications.

The typical sand-casting process involves several key stages:

1. **Pattern Making:** A pattern, which is a replica of the final part (oversized to account for metal shrinkage during cooling), is created. Patterns can be made from materials like wood, plastic, or metal and are commonly CNC machined.
2. **Mould Creation:** The pattern is used to create a cavity in a refractory moulding material, which is typically sand mixed with a bonding agent. For complex internal geometries, cores (also usually made of sand) are placed within the mould. The mould is often made in two or more parts to allow for pattern removal and subsequent assembly.
3. **Melting and Pouring:** Steel scrap, with precise additions of virgin alloys to achieve the desired grade, is melted in a furnace (commonly an Electric Arc Furnace) to temperatures around 1600°C. The molten steel is then poured into the assembled mould cavity through a carefully designed gating system, which controls the flow of the molten steel and helps prevent defects.
4. **Solidification and Cooling:** The molten steel solidifies within the mould, taking the shape of the cavity. The rate of cooling is a critical factor influencing the final microstructure and properties of the casting.
5. **Shakeout and Finishing (Fettling):** Once cooled, the casting is removed from the mould (shakeout). Excess material, such as the gating system and any feeders (reservoirs of molten metal that compensate for shrinkage), is removed. The casting surface is cleaned, typically by shot blasting.
6. **Heat Treatment:** Most structural castings undergo heat treatment processes (e.g., annealing, normalising, quenching and tempering) to refine the grain structure of the metal and to achieve the specified mechanical properties, such as strength, toughness, and ductility.
7. **Inspection and Machining:** The casting is then inspected using various non-destructive testing (NDT) methods. If required, critical surfaces or features like bolt holes are machined to achieve precise dimensional tolerances.


Figure 2: MSG Exosphere © Mike McNulty


Figure 3: Close-up of cast node in MSG Exosphere © Mike McNulty

This process, though complex, allows for the creation of monolithic components with geometries that are often impractical or impossible to achieve through traditional fabrication from steel plates and sections.

Case Studies from SCI P441

The practical application and benefits of steel castings are illustrated in SCI P441 through several notable case studies, showcasing both bespoke and standardised solutions.

The MSG Sphere, Las Vegas

The construction of this large-diameter spherical structure required cast steel nodes to connect the circular hollow sections of its exosphere. The complex, multi-planar geometry of these connections, each typically joining six members, presented a significant fabrication challenge. The adoption of 368 bespoke cast steel nodes supplied by Cast Connex, with individual weights up to 6.5 tonnes, resulted in substantial advantages over fabricated alternatives. These included a reported 57% weight reduction, a 76% decrease in the number of bolts, and a 76% reduction in surface area requiring coating for a typical node. CNC machining of the casting flanges ensured the geometric accuracy required for assembly. The project also utilised standardised or “off-the-shelf” cast field-bolted splice components for circular hollow sections (also from Cast Connex), demonstrating the versatility of casting solutions.

Western Concourse, King’s Cross Station, London

For the roof structure of the Western Concourse, 1.5-tonne multi-planar cast steel nodes were employed to connect the tapering elliptical columns to the steel diagrid shell. The castings were instrumental in realising the complex, organic architectural forms while fulfilling the structural load transfer requirements. The alternative of fabricating these nodes would have involved extensive and highly complex welding operations, making the aesthetic and structural vision difficult to achieve with the same level of refinement.

Procurement and Design Considerations

SCI P441 provides detailed guidance on the procurement process and design considerations specific to custom steel castings, an area where early decisions significantly impact project outcomes.

Procurement Routes

The publication discusses two primary procurement models. For project teams with limited prior experience in castings, the engagement of a specialist ‘casting designer-supplier’ is recommended. Designer-suppliers, such as companies like Cast Connex, provide expertise from conceptual design assistance and detailed engineering through to foundry liaison, quality assurance, and supply. This integrated approach can reduce risk in the process for the main project team.

The alternative, traditional route involves direct engagement between the steelwork contractor and the foundry, necessitating a higher degree of specialist knowledge within the contractor and principal design team. Regardless of the chosen model, early engagement with casting specialists or foundries is emphasised to ensure technical feasibility, define clear performance requirements, and establish realistic programmes and cost estimates.

Design and Specification Process

The design of structural castings is an iterative process that must balance structural performance, architectural intent, and manufacturability (castability). SCI P441 highlights the use of Finite Element Analysis (FEA) for optimising casting geometry and verifying structural adequacy under design loads. This often involves multiple iterations to refine shapes, minimise weight, and ensure stress levels are within permissible limits. Early collaboration between the structural engineer, architect, steelwork contractor, and the casting specialist is crucial to harness the full potential of castings. The development of comprehensive documentation, including a Performance Requirements Report (defining loads, service conditions, and applicable standards) and a Casting Design Report (detailing the proposed geometry, material grade, and analysis results), is crucial. The product standard for structural castings in the UK is BS EN 10340 *Steel Castings for Structural Uses*, which covers a range of carbon and stainless steel grades and their associated mechanical properties.


Figure 4: Western Concourse, Kings Cross - steel castings at the fabrication shop © John McAslan + Partners


Figure 5: Cast nodes connect the ‘branches’ to the ‘trunk’ columns © John McAslan + Partners

Quality Management and Sustainability

Effective quality management is critical to the successful use of steel castings and is thoroughly addressed in the new guide. Sustainability is also an important consideration in the use of steel castings.

Quality Assurance and NDT

SCI P441 underscores the importance of robust quality control measures throughout the casting production process. This includes foundry accreditations (e.g., BS EN 10340 and ISO 9001 quality management systems) and a well-defined inspection and testing plan, agreed between the specifier and the foundry. The publication details various non-destructive testing (NDT) techniques applicable to castings, such as visual testing (VT), magnetic particle testing (MT), liquid penetrant testing (PT), ultrasonic testing (UT), and radiographic testing (RT). Crucially, it advises on the specification of appropriate NDT methods and acceptance criteria (severity levels) related to the service conditions and criticality of the specific casting, cautioning against overly stringent blanket requirements that may lead to unnecessary cost. A well-defined NDT programme, tailored to the casting's function, provides vital confidence to all project stakeholders.

Sustainability Considerations

Steel castings offer several sustainability advantages. The casting process allows for 'near-net shape' manufacturing, optimising material utilisation and minimising the generation of waste material compared to subtractive manufacturing processes. The reduction in welding achieved by using monolithic cast nodes can also lead to a decrease in welding consumables and associated energy consumption. Furthermore, the potential for enhanced durability and fatigue life in cast components can contribute to extended structural service lives and reduced whole-life carbon impacts. The optimised, often lighter cast components can also lead to secondary benefits such as reduced transportation emissions and reduced demands on supporting structures and foundations. The smoother profiles of castings can also improve the longevity of protective coating systems, reducing maintenance interventions.

The role of SCI P441

Structural steel castings provide a valuable engineering solution for complex structural components and connections, offering benefits in design flexibility, structural efficiency, and construction. The effective specification, design, and procurement of these components requires specialist knowledge. SCI Publication P441, developed with support from industry leaders Cast Connex, serves as a significant technical reference for the constructional steelwork industry. It equips professionals with all the necessary information to assess the suitability of castings and to implement them successfully in projects.

SCI P441: *Structural Steel Castings* is available as a free PDF download for all SCI members through SteelBiz, and can be found at www.steel-sci.com. ■

Composite column design

Although not commonly used in the UK, composite columns can, from a structural, fire resistance and accidental loading perspective, be advantageous, as they combine the benefits of steel and concrete. They are widely used in tall buildings because of the high resistance-to-footprint ratio they enable. In this article Dr Graham Couchman outlines the process for ambient temperature design given in EN 1994-1-1[1], with a focus on simplified methods for cross-section and member resistance.

Introduction

In order to improve clarity, design complications, such as a need to consider long terms effects, transverse shear, and second order effects, are dealt with by references to relevant clauses. The upcoming Generation 2 EN 1994-1-1[2] presents the same approach, with minor changes to some notation and factors, and of course clause numbering.

Design to EN 1994-1-1

Ultimate limit state (ULS) design is covered in Section 6.7. Alongside a General Method, which I will not discuss, a Simplified Method with a scope limitation broad enough to not inhibit its use for most practical design is given (6.7.3). The scope is limited to cross-sections that are doubly symmetric and uniform over the length of the member. Limits related to concrete cover of embedded sections, aspect ratio of the cross-section, slenderness, amount of reinforcement and the nature of the steel element (you can have built-up sections, but you can't have multiple unconnected steel sections) are also defined in 6.7.3.

Cross-section resistance

The plastic resistance to compression is defined in 6.7.3.2. It is simply the sum of the resistances of the three components (structural steel, concrete and

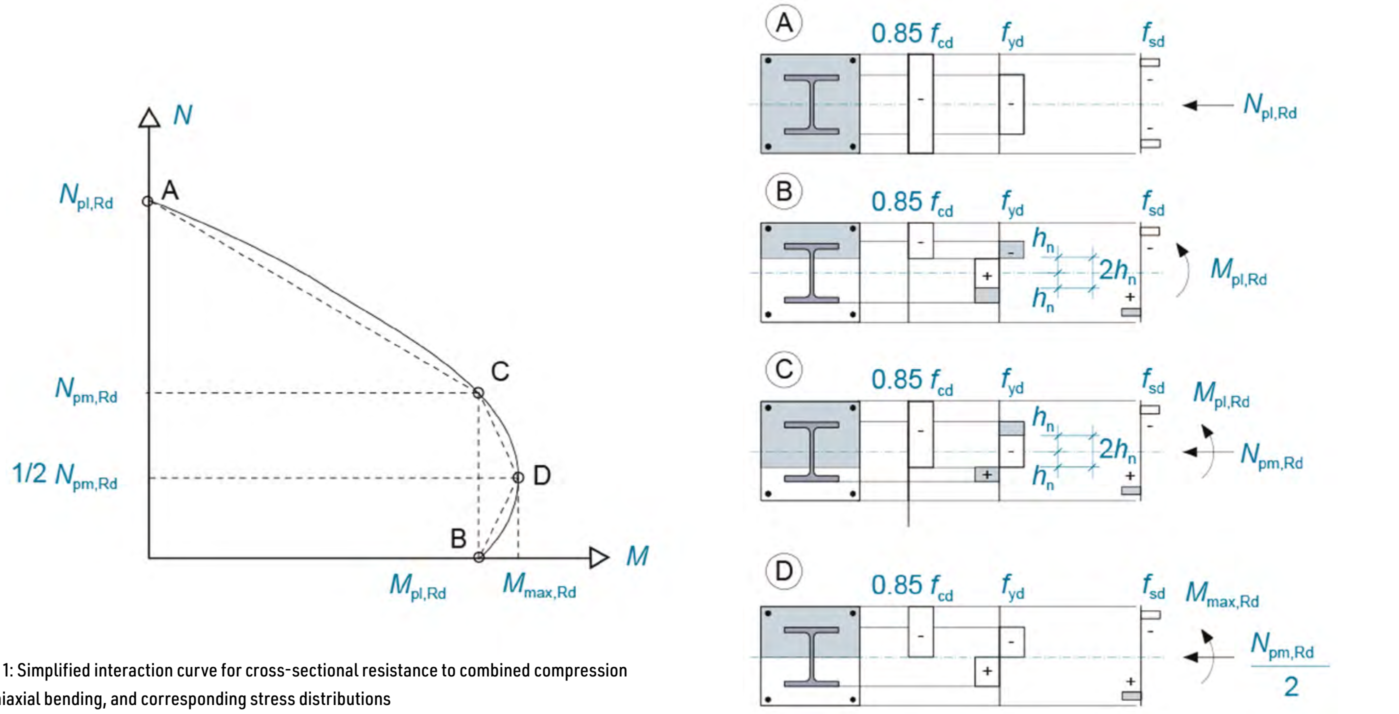


Figure 1: Simplified interaction curve for cross-sectional resistance to combined compression and uniaxial bending, and corresponding stress distributions

reinforcement). For an encased steel section:

$$N_{pl,Rd} = A_a f_{yd} + 0.85 A_c f_{cd} + A_s f_{yd}$$

For composite element notation, the subscripts a, c and s refer to structural steel, concrete and reinforcing steel respectively. For a concrete-filled hollow section the value of 0.85 is replaced by 1.0, presumably to reflect the benefits of concrete confinement, although 6.7.3.6 gives an enhanced axial resistance for concrete-filled hollow sections within a certain slenderness limit.

The impact of transverse shear, which we will assume is negligible, is considered in 6.7.3.2 (3) and (4).

A simplified method for considering the interaction of compression and bending on the cross-section resistance is given in 6.7.3.2(5) and illustrated in Figure 1 on the previous page. Different neutral axis positions are considered:

- Point A is pure compression
- Point B is pure bending
- Points C and D consider equilibrium of forces, with Point D showing that the presence of some axial compression can – like pre-stressing – enhance the moment resistance

Stiffness, slenderness and member analysis

Stiffness, slenderness and member analysis are covered in 6.7.3.3 and 6.7.3.4. For the determination of the relative slenderness and elastic critical force, the stiffness is taken as the sum of the stiffnesses of the three components:

$$(EI)_{eff} = E_a I_a + E_s I_s + K_e E_{cm} I_c$$

K_e is an approximate correction factor to allow for concrete cracking, that should be taken as 0.6. Second moments of area are for the plane of bending being considered, and the uncracked concrete value should be used.

The relative slenderness λ for the chosen plane of bending is given by:

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}}$$

$N_{pl,Rk}$ is the characteristic value of the plastic resistance (characteristic values are used rather than design strengths of materials).

N_{cr} is the elastic critical normal force for the relevant buckling mode. For flexural buckling the elastic critical normal force is given by the Euler load:

$$N_{cr} = \frac{\pi^2 (EI)_{eff}}{L_{cr}^2}$$

L is the effective length, which may vary depending on the buckling mode being considered.

For the determination of internal forces, the stiffness is reduced using the factors defined below (note the values of 0.5 and 0.9 are defined in the code as variable calibration and correction factors respectively, with these values recommended):

$$(EI)_{eff,II} = 0.9(E_a I_a + E_s I_s + 0.5 E_{cm} I_c)$$

The concrete stiffness E_{cm} should be reduced to allow for any long-term effects, using 6.7.3.3(4).

Second order effects and imperfections are considered in 6.7.3.4(3) (4) and (5). For second order effects, (5) defines a simple magnification factor that multiplies the greatest first-order bending moment M_{Ed}:

$$k = \frac{\beta}{1 - N_{ed}/N_{cr,eff}} \geq 1.0$$

The critical normal force for use in this check, $N_{cr,eff}$, is determined using the effective stiffness, as defined above, but with an effective length taken as the column length. The equivalent moment factor β is taken from EN 1994-1-1 Table 6.4.

Member resistance

Members in pure axial compression are considered in 6.7.3.5(2). Member resistance is the cross-sectional resistance reduced by the factor χ according to EN 1993-1-1[3], 6.3.1.2, as a function of the relevant buckling curve and relative slenderness. The verification is therefore:

$$\frac{N_{ed}}{\chi N_{pl,Rd}} \leq 1.0$$

EN 1994-1-1 Table 6.5 identifies which buckling curve to use for different types of cross-section, and each axis of bending. It also defines member imperfections for each case.

Member resistance in combined axial compression and uniaxial bending is considered in 6.7.3.6. The maximum applied moment is compared with a moment resistance that is reduced to allow for the level of axial force present, using the following verification:

$$\frac{M_{ed}}{\mu_d M_{pl,Rd}} \leq \alpha_M$$

According to 6.7.3.6(2), the reduction factor μ_d is derived from the curve describing the cross-sectional resistance to combined compression and uniaxial bending (as shown in Figure 2). The factor α_M is taken as 0.9 for steel grades not more than S355, and 0.8 for S420 and S460.

The added complication of combined compression and biaxial bending is considered in 6.7.3.7, using the same principles as described above, (and therefore not repeated here).

Conclusions

Composite beams and composite slabs are widely used in the UK, as the benefits of combining the properties of steel and concrete are widely recognised. Composite columns are much less used, despite the performance benefits to be gained. This may be at least partly due to the frame erection process implications of combining the two materials in a column. Off-site manufactured elements could help to address this.

References

1. BS EN 1994-1-1:2004. *Eurocode 4: Design of composite steel and concrete structures.* Part 1-1: General rules and rules for buildings. Incorporating corrigendum April 2009. BSI, 2009.
2. Fpr EN 1994-1-1:2025. *Eurocode 4: Design of composite steel and concrete structures.* Part 1-1: General rules and rules for buildings. CEN, 2025.
3. BS EN 1993-1-1:2005. *Eurocode 3: Design of steel structures.* Part 1-1: General rules and rules for buildings. Incorporating corrigenda February 2006 and April 2009. BSI, 2010.

Recommended reading

- PN006 NCCI: Design of reinforced concrete filled, hot finished structural steel hollow sections in fire

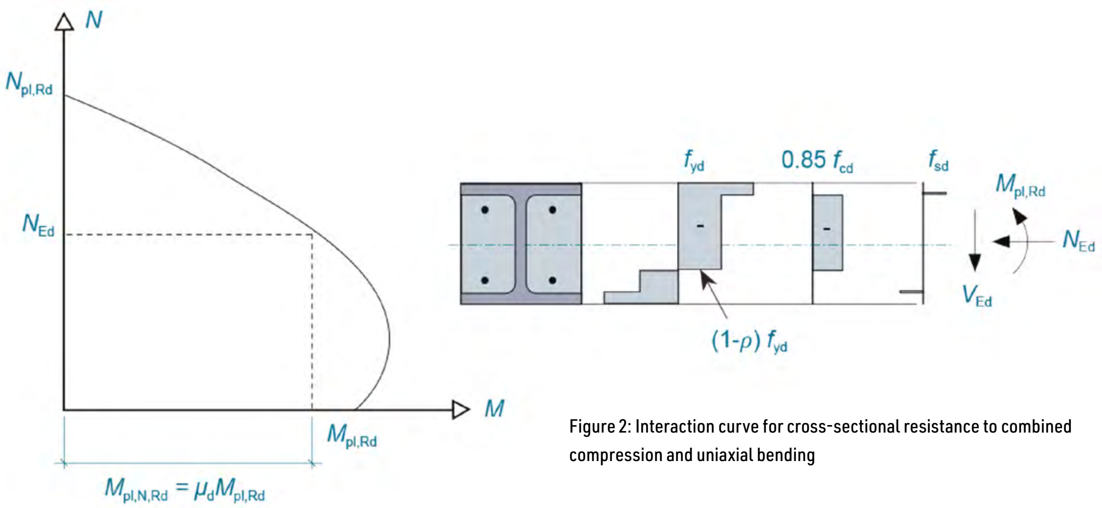


Figure 2: Interaction curve for cross-sectional resistance to combined compression and uniaxial bending

Floors with heavy loading – what are the implications?

In this article, Dr Graham Couchman considers the implications of high levels of loading, and how they can change expected failure modes and the design rules that should be applied. As usual, his focus is on composite floors, recognising their commercial significance to the UK steel construction sector.

Introduction

More and more clients are requiring their buildings to be designed for heavy imposed floor loads. A recent enquiry aimed at our Advisory Desk concerned a uniformly distributed load (UDL) of 22 kN/m^2! We are also seeing many cases where there are heavy point loads (PL), often numerous and sometimes at close centres. Although in some cases these onerous loads are specified simply because the client wants to keep their options open to avoid delays while the real loads are determined, sometimes it seems the loads are realistic.

Heavy uniform loading

Many floors are designed for an unfactored UDL of 5 kN/m^2. In terms of what a typical floor will experience, that is already pretty high, 2.5 kN/m^2 is recommended by the British Council for Offices, resulting in wasted money and wasted material. But for some types of building even 5 kN/m^2 is not enough, and that means that design rules and failure modes may change from what is normally used and expected. Some examples of things that may change are considered below.

Composite slabs

Design of the vast majority of composite slabs is governed by the ability of the decking to support the wet weight of concrete (and coincident construction stage imposed loads). The weight of concrete usually results from the need for a certain depth in order to satisfy the fire insulation criterion for the finished slab. Propping the decking to help it support the wet concrete is not normally considered because of the detrimental process implications of propping. However, with heavy final stage imposed loading the composite slab design may become more critical than the construction stage decking design, either at normal temperature or for the fire condition. The latter is likely to be particularly critical – decking losing almost all its resistance in fire is why, having ignored any end continuity for normal temperature design, we nevertheless take it into account in fire because the slab needs all the help it can get. Some sagging resistance comes from the very weakened deck, with a good lever arm, plus the more performant upper mesh with a small lever arm. The mesh is more performant because it is at a much lower temperature, being insulated by the concrete, so retains reasonable strength.

Even with the reduced level of loading that is associated with the fire limit state, this combination of sagging and hogging moment resistances may not be enough, particularly under high imposed load. In such cases, and in cases where there is no end continuity, therefore no hogging resistance, an obvious solution would be to place reinforcement bars in the decking troughs. These bars provide additional tensile resistance in span, with a good lever arm and relatively good strength as they are insulated by surrounding concrete. It is worth adding that some software may not allow bars and decking to be combined when determining tensile reinforcement and thereby sagging resistance. The so-called 'mesh and deck' method, which has a justification for the deck contribution that is based on tests, does not allow bars to act also.

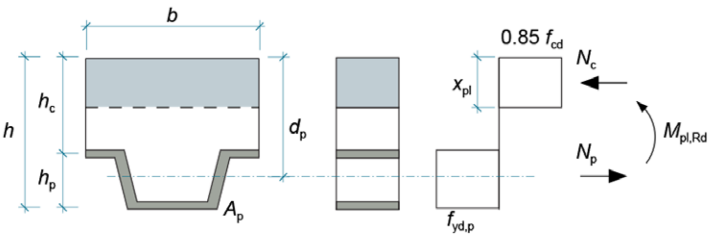


Figure 1: Stress blocks and lever arms for a composite slab cross-section (no bars in troughs so the only tensile reinforcement is the decking). Notation as per BS EN 1994-1-1

Composite beams

Many composite beams are designed using the rules given in SCI publication P405 *Minimum degree of shear connection rules for UK construction to Eurocode 4*[1], which very significantly reduce the required minimum degree of shear connection compared to BS EN 1994-1-1[2]. This reduction makes many designs that would not be possible – you simply cannot fit enough shear studs on the beam when you have transverse decking with ribs at a given pitch – very efficient. P405 rules are adopted in most composite beam software used in the UK.

The reason why we have minimum degree of connection rules is to ensure that the combined stiffness of the studs on a beam is enough to prevent the slip at the beam ends, where it is a maximum, exceeding the stud's slip capacity. Slip capacity is typically taken as either 6mm, for studs in solid slabs, with parallel decking, or transverse re-entrant decking, or 10mm for transverse trapezoidal decking. This is therefore a strength check used to verify a stiffness requirement, and is not transparent.

The minimum degree of connection rules in P405 differentiate between levels of imposed loading. For normal cases the factored imposed UDL shall not exceed 9 kN/m^2. When loading is not just UDL, as an alternative this limit can be satisfied by limiting the moment due to factored imposed loading to 70% of the moment due to factored total loading. More onerous rules are given to cover factored imposed loading up to 12 kN/m^2, which is defined in P405 as 'heavy loading'.

As an example, for a 16m span symmetric beam with transverse trapezoidal decking (studs have a slip capacity of 10mm), in S355 and assuming the beam is fully utilised in bending, the minimum degree of connection is:

- Normal loading 43%
- Heavy loading 91%

The reason for this differentiation in the level of imposed loading can be understood by considering the background to the rules. Numerical analyses were undertaken by SCI, using ANSYS to model composite beams with springs representing appropriately defined load-slip characteristics for the shear studs – initial stiffness, resistance, slip capacity (Figure 2, over page). For a given beam the number of studs was increased until the end slip no longer exceeded the slip capacity, then that number of studs was defined as a degree of shear connection. By analysing multiple beams, and considering different slip capacities for studs it was possible to define new degree of shear connection rules as a function of span, steel strength, slip capacity etc. An additional variable for unpropped beams was the relative level of imposed load.

Two phenomena differentiate how an unpropped beam behaves compared to a propped beam. Phenomenon 1 is that self-weight has no impact on the shear studs – the fluid concrete merely 'runs' around them. Only the imposed loads result in forces applied to the studs, transferred via the now hard concrete, as the beam deflects. The relative level of dead to imposed loads, as well as the total load, is therefore important as it affects how much the studs will slip, so it can be readily seen how relative load levels will affect the minimum degree of connection that is needed. A complication, which is why numerical analysis is

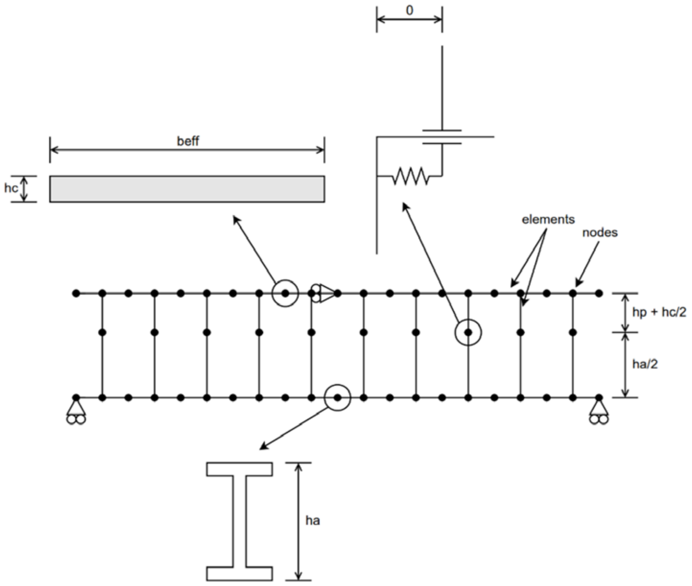


Figure 2: Schematic of ANSYS model of a composite beam (the upper line of nodes and elements represents the slab, the lower level the beam, with springs representing stud behaviour)

needed to model beams, is Phenomenon 2. The steel beam experiences higher strains under self-weight when unpropped compared to the same section when propped, because a composite beam typically has over twice the stiffness of the steel beam it is based on. The lower stiffness of the (bare steel) beam in an unpropped situation results in greater deflection and curvature, which means the extreme fibres get closer to their elastic limit under self-weight. When imposed loading is subsequently applied to the composite beam some of the steel then goes beyond its elastic limit (i.e. loses stiffness) 'sooner', deflections are greater as is end slip. This partly counters the lower slip due to Phenomenon 1.

The example numbers given above – 43% versus 91% - show how sensitive the required minimum degree is to the level of imposed loading. It is therefore very important that the P405 rules are not used out of scope, as they would be with a factored imposed load over 12 kN/m² (unfactored in excess of say 8 kN/m²).

BS EN 1994-1-1 gives rules for minimum degree of connection in 6.6.1.2. They do not distinguish between propped and unpropped construction, which means that the ratio of dead to imposed load is irrelevant (they cover the worst case where all loads are assumed to have an impact on the shear studs – i.e. propped construction). These rules may therefore be used for any level of imposed load, although care should always be taken when applying codified rules to very unusual situations as the rules may be based on tests and/or experience that were not representative of the situation actually being designed. Note this does not mean that the stud resistances from BS EN 1994-1-1 may also be used – they have been shown by test to be inaccurate and indeed unconservative when there are two studs per trough.

Unlike its predecessor, Generation 2 EN 1994-1-1[3], 8.6.3.3 does distinguish between propped and unpropped construction by using a variable k_{up} to adjust the minimum degree of shear connection that is required. This variable has a value of 1.0 for propped, and could drop to 0.85 at its extreme for unpropped. There is no mention of the relative level of dead to imposed loading, but as the benefit of unpropped is limited to only a 15% reduction in minimum degree, getting it right is a much less significant problem than when P405 rules are used.

Heavy and numerous point loads

The subject of heavy and/or numerous point loads has been covered in several recent articles and AD Notes from SCI. Heavy PLs place onerous requirements on a slab in terms of the transverse bending they cause, and the ability of what may be a relatively narrow strip of slab to support them[4,5]. Loads near a support, for example caused by a MEWP, tend to push the slab's vertical shear resistance to the limit. Local punching through the slab, which is normally never critical, may also be a concern.

Because composite slabs are designed as one-way spanning, the biggest danger with numerous PLs may be that a designer does not appreciate all the loading that will be present on the 1m wide strip being designed[6]. A PL placed near centre span may well mobilise a strip of slab that is greater than 1m wide, and if it is sufficiently close (transversely) to another PL they could interact and both need to be considered when determining the input loads. The designer should take care to consider loads near to, not just centred, on the line of the strip being designed.

Conclusions

Experienced designers can often judge whether a design output feels right or not, based on their past experience. For example, anyone familiar with composite slab design will know that the ability of the steel decking to support the wet weight of concrete governs slab design most of the time. However, self-evidently when faced with an unusual situation past experience loses some of its value. In this article we have considered how the current demands from numerous clients to consider heavier floor loads than have been typical, and multiple point loads on some floors, can affect key aspects of composite floor design.

References

1. P405 Minimum degree of shear connection rules for UK construction to Eurocode 4, SCI, 2015
2. BS EN 1994-1-1:2005, Eurocode 4 - Design of composite steel and concrete structures - Part 1-1: General rules and rules for buildings, BSI, 2005
3. Fpr EN 1994-1-1:2025, Eurocode 4 - Design of composite steel and concrete structures - Part 1-1: General rules and rules for buildings, CEN, Unpublished
4. AD450 Resistance of composite slabs to concentrated loads, SCI, 2020
5. AD477 Transverse bending of composite slabs subjected to point loads. SCI, 2021
6. Dealing with multiple point loads on a composite slab. NSC Nov/Dec 2024

Members with axial load and moments – at elevated temperatures

In what could be the final article on structural steel design at elevated temperatures (at least until Gen2 in 2028!) David Brown of the SCI looks at the general case of members subject to combined axial and bending.

A reader of *New Steel Construction* has noted that the design of members subject to bending and members subject to an axial force had been covered – but both effects in splendid isolation, and observed that the general case was to have combined moment and axial effects. Never one to shirk a challenge, this article is the result.

Overall plan

Designers will know that in normal design (at ambient temperatures) the resistance of members to combined axial load and bending is covered by expression 6.61 and 6.62 of BS EN 1993-1-1. Reference to that pair of expressions is usually sufficient to dampen any further enthusiasm – the expressions are painful to work through by hand.

At elevated temperatures, there are similar looking pairs of expressions in BS EN 1993 1-2. Expressions 4.21a and 4.21b cover Class 1 and 2 sections, and expressions 4.21c and 4.21d cover Class 3 sections. As might be expected, the expressions for Class 3 are the same as those for Class 1 and 2, but utilising the elastic modulus in place of the plastic modulus. Class 4 sections are not covered in the same way at all – SCI advice is generally to choose a different section.

Within the expressions 4.21a and 4.21b, the ratios follow the familiar form of $\frac{\text{effect}}{\text{resistance}}$, both terms being affected by reductions in the fire limit state. The calculation of resistance in bending and in axial has been covered in previous articles. The final answer is the summation of the ratios for axial, major axis bending and minor axis bending.

In the same way that expressions 6.61 and 6.62 have interaction factors, expressions 4.21a and 4.21b have factors k_{LT}, k_y and k_z. The factors depend on the shape of the bending moment diagram and the utilisation in compression, so are in principle familiar to anyone who has looked in detail at the ambient design.

The expressions

Expressions 4.21a and 4.21b are reproduced below:

$$\frac{N_{\mathrm{fi,Ed}}}{\chi_{\mathrm{min,fi}} A k_{y,\theta} \frac{f_y}{\gamma_{\mathrm{M,fi}}}} + \frac{k_y M_{\mathrm{fi,Ed}}}{W_{\mathrm{pl},y} k_{y,\theta} \frac{f_y}{\gamma_{\mathrm{M,fi}}}} + \frac{k_z M_{z,\mathrm{fi,Ed}}}{W_{\mathrm{pl},z} k_{y,\theta} \frac{f_y}{\gamma_{\mathrm{M,fi}}}} \le 1 \qquad 4.21a$$

$$\frac{N_{\mathrm{fi,Ed}}}{\chi_{z,\mathrm{fi}} A k_{y,\theta} \frac{f_y}{\gamma_{\mathrm{M,fi}}}} + \frac{k_{\mathrm{LT}} M_{y,\mathrm{fi,Ed}}}{\chi_{\mathrm{LT,fi}} W_{\mathrm{pl},y} k_{y,\theta} \frac{f_y}{\gamma_{\mathrm{M,fi}}}} + \frac{k_z M_{z,\mathrm{fi,Ed}}}{W_{\mathrm{pl},z} k_{y,\theta} \frac{f_y}{\gamma_{\mathrm{M,fi}}}} \le 1 \qquad 4.21b$$

Within the first term of expression 4.21a, the minimum value of the reduction factor χ_{fi} will usually be $\chi_{z,\mathrm{fi}}$ in the minor axis. It would be unusual to have the minimum slenderness in the major axis.

Considering the second term, it seems almost certain that 4.21b will be critical, since the reduction factor $\chi_{\mathrm{LT,fi}}$ appears in the denominator and is always 1.0 or (significantly) less. It may be possible that this ratio in 4.21a is critical, but only at very short lengths – recognising the values of k_y and k_{LT} have not been considered yet and could potentially change the conclusion.

The third term is the same in both expressions, so a casual review suggests that 4.21b is the likely candidate to be critical.

A numerical example

It seems that some readers value a numerical example, if only to check their own spreadsheet calculations. This example considers a 203 UC 60 in S355, 4m long, at 500°C. The bending moments in the fire condition are at one end of the column, diminishing to zero at the other end. The shape of the bending moment diagram means that C_1 = 1.77.

Classification

The first step is to classify the section to determine which pair of expression should be verified. Member class may change at elevated temperature, because the value of ε is modified.

At elevated temperature, $\varepsilon = 0.85\sqrt{\frac{235}{f_y}} = 0.85 \times \sqrt{\frac{235}{355}} = 0.69$

The Class 2 limit for the flange is 10ε = 10 × 0.69 = 6.9

The actual $\frac{c_f}{t_f}$ = 6.2, so the flange is at least Class 2.

The classification of the web in combined axial load and bending requires the axial load, which is 650 kN in the fire limit state.

From the expression in P362,

$$\alpha = ½\left(1 + \frac{N_{\mathrm{Ed}}}{f_y c t_w}\right) = ½\left(1 + \frac{650 \times 10^3}{355 \times 160.8 \times 94}\right) = 1.106$$

therefore α takes the limiting value of 1.0.

As α > 0.5 then the Class 2 limit is given by:

$$\frac{456\varepsilon}{13\alpha - 1} = \frac{456 \times 0.69}{13 \times 1.0 - 1} = 26.2$$

The actual $\frac{c_w}{t_w}$ =17.1 so the web and the whole section is at least Class 2.

For Class 2 sections, the pair of expressions 4.21a and 4.21b must be verified.

Design data

The design resistances at ambient and at 500°C are shown below, calculated as shown in previous articles, with the design effects in the fire limit state.

Ambient	$N_{\mathrm{b,z,Rd}}$ = 1450 kN	$M_{\mathrm{b,Rd}}$ = 233 kN	
At 500°C	$N_{\mathrm{b,z,fi,Rd}}$ = 893 kN	$M_{\mathrm{b,fi,Rd}}$ = 123 kN	
Effects	$N_{\mathrm{fi,Ed}}$ = 650 kN	$M_{\mathrm{y,fi,Ed}}$ = 40 kN	$M_{\mathrm{z,fi,Ed}}$ = 10 kN

The calculation of $N_{\mathrm{b,z,fi,Rd}}$ does not apply any reduction to the buckling length that would be permitted in Figure 4.1 of BS EN 1993-1-2. The value of 650 kN selected for $N_{\mathrm{fi,Ed}}$ is relatively low. If the column was utilised to around 85% at ambient, a reasonable value for $N_{\mathrm{fi,Ed}}$ might be 0.85 × 1450 × 0.65 = 801 kN. The reduction of 0.65 is based on the simplification given in Note 2 to Figure 2.1 of BS EN 1993-1-2. Previous articles have noted that this reduction is conservative, so the value of 650 kN can be seen as a more realistic value.

Intermediate values in compression

At 500°C, the value of $k_{y,\theta}$ is 0.78 and the value of $k_{E,\theta}$ is 0.60.

Following the approach recommended in previous articles, the salient calculation values are shown below, at a temperature of 500°C. Note that the values in the major axis will be required later in the calculations. As the steel is S355, α = 0.529.

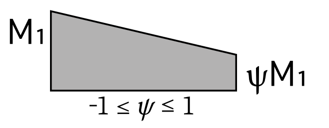
	Minor axis	Major axis
$\bar{\lambda}_{\mathrm{fi}}$	1.148	0.667
χ_{fi}	0.422	0.666
$N_{\mathrm{b,fi,Rd}}$	893 kN	1410 kN

Intermediate values in bending at 500°C

	Minor axis
$\bar{\lambda}_{LT,fi}$	0.644
$\chi_{LT,fi}$	0.678
$M_{b,fi,Rd}$	123.3 kN

Interaction coefficients

The values of coefficients k_y, k_z and k_{LT} involve the shape of the bending moment diagram and an equivalent uniform moment factor β_M. In very many cases, the bending moment diagram in the column will be linear (with no significant loading applied along the length of the column). In the case of a linear bending moment diagram, the value of β_M is shown below.

	$\beta_{M,\psi} = 1.8 - 0.7\psi$

In this instance, since in both axes $\psi = 0$ then $\beta_{M,\psi} = 1.8$

The coefficients are straightforward to calculate, but designers should pay attention to the latest version of the code. In earlier versions of the code, the expressions for μ_y and μ_z were unfortunately reversed and modified.

Then $k_{LT} = 1 - \dfrac{\mu_{LT} N_{fi,Ed}}{\chi_{z,fi} A k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}}$ with $\mu_{LT} = 0.15\bar{\lambda}_{z,\theta}\,\beta_{M,LT} - 0.15 \leq 0.9$

$\mu_{LT} = 0.15 \times 1.148 \times 1.8 - 0.15 = 0.16$

In the expressions for the interaction coefficients, it is disappointing that the denominator is not simply shown as the relevant resistance at elevated temperature.

$$k_{LT} = 1 - \frac{0.16 \times 650}{893} = 0.88$$

The second coefficient is k_y, given by:

$$k_y = 1 - \frac{\mu_y N_{fi,Ed}}{\chi_{y,fi} A k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} \leq 3$$

with $\mu_y = (2\beta_{M,z} - 5)\,\bar{\lambda}_{y,\theta} + 0.44\beta_{M,y} + 0.29 \leq 0.8$ and $\lambda_{y,20°C} \leq 1.1$

In this case, the non-dimensional slenderness was 0.585 at ambient temperature, so the limiting value of 1.1 does not apply.

$\mu_y = (2 \times 1.8 - 5) \times 0.667 + 0.44 \times 1.8 + 0.29 = 0.149$

Note that the non-dimensional slenderness in the major axis is required.

$$k_y = 1 - \frac{0.149 \times 650}{1410} = 0.931$$

The final coefficient is k_z, given by:

$$k_z = 1 - \frac{\mu_z N_{fi,Ed}}{\chi_{z,fi} A k_{y,\theta} \dfrac{f_y}{\gamma_{M,fi}}} \leq 3$$

with $\mu_z = (1.2\beta_{M,z} - 3)\bar{\lambda}_{z,\theta} + 0.71\beta_{M,y} - 0.29 \leq 0.8$

$\mu_z = (1.2 \times 1.8 - 3) \times 1.148 + 0.71 \times 1.8 - 0.29 = 0.024$

$$k_z = 1 - \frac{0.024 \times 650}{893} = 0.983$$

Bringing it all together

Substituting all the information into expressions 4.21a and 4.21b, the results are:

$$\frac{650}{893} + \frac{0.931 \times 40 \times 10^6}{656 \times 10^3 \times 0.78 \times 355} + \frac{0.983 \times 10 \times 10^6}{305 \times 10^3 \times 0.78 \times 355} = 1.049$$

and

$$\frac{650}{893} + \frac{0.88 \times 40 \times 10^6}{0.678 \times 656 \times 10^3 \times 0.78 \times 355} + \frac{0.983 \times 10 \times 10^6}{305 \times 10^3 \times 0.78 \times 355} = 1.131$$

In this case, at the end of the process, the column is unsatisfactory. In practice, designers will not start the process with a temperature, but will know the selected column and the effects in the fire limit state - and need to calculate the critical temperature. This is easy if the calculations are embedded in a spreadsheet. The critical temperature is found to be 441°C in this instance. The specification for the necessary protection is therefore to limit the temperature of this steel member to no more than 441°C at the required period of fire resistance.

Conclusions

The primary purpose of this article is to help those designers wishing to correctly determine a critical temperature, especially those preparing their own spreadsheet solution. The process is very similar to the verification at ambient temperature with a pair of interaction expressions to satisfy after determining intermediate values. After inspecting the expressions in BS EN 1993-1-2, it does seem likely that expression 4.21b will be critical.

This example also serves as a reminder that the critical temperatures in Table NA.1 of the UK NA to BS EN 1993-1-2 are limited to the case of members in pure compression – and should not be used if any moment is introduced to the section. ■

Advisory Desk 2025

AD 536:
In-plane member buckling lengths for portal frames

The verification of members in portal frames leads to a common question about in-plane buckling lengths, especially when designers are using general software. General software requires an effective length factor or buckling length in both axes, leading designers to question what the in-plane buckling length is. The subject was covered in *New Steel Construction* of June 2020[1] so this AD serves as a summary reminder and a commentary on other potentially misleading guidance.

The in-plane verification of members in a portal frame is completed by verifying the in-plane stability of the entire frame. BS 5950 provided clear advice in 5.2.3.1 (2nd para) and 5.5.2. Once frame stability is verified and second-order effects are allowed for if necessary – by calculating $\alpha_{cr,est}$, the only in-plane verification is the resistance of the cross-section.

Out-of-plane verifications are of course necessary, using expression 6.62 of BS EN 1993-1-1. Comprehensive guidance on stability verifications is given in P399.

P397, also covering portal frame design, was written before Eurocode guidance had been fully developed. P397 includes guidance and an example covering the in-plane verification of portal frame members, which clearly conflicts with the above advice and should be ignored. A warning in the foreword to P397 points out that the guidance relating to in-plane buckling was likely to change – it did, and is clarified in P399.

SCI members have pointed out that SN031a (available on Steelbiz) and Table 6.2 of P360, both refer to buckling lengths in portal buildings. It's likely that the original guidance in SN031a referred to the widespread continental practice of "portalised" frames with columns and flat roof beams (perhaps with pinned ends), rather than the pitched roof portals that dominate UK practice. The guidance in SN031a and Table 6.2 of P360 is not appropriate for pitched roof portals.

Sometimes, buildings with roof trusses are "portalised" by connecting both top and bottom chords to the column, thus providing in-plane stability. Designers should note that the buckling lengths of the columns in these frames may be well over twice the system length, depending on the depth of the truss and clear height of the column. The Steel Designers' Manual contains helpful charts to determine the appropriate factor.

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[1] D. G. Brown: In-plane stability of portal frames, *New Steel Construction*, June 2020

AD 537:
Web resistance

The Blue Book contains values of web resistance for various stiff bearing lengths, s_s. During the work to prepare for the "Generation 2" changes, it has become clear that one limitation in BS EN 1993-1-5:2006 was not observed in the original calculations. This omission means that, for smaller beams, some web resistance values are not correct at longer stiff bearing lengths. This AD explains the omission and advises on the work around.

Clause 6.3(1) of BS EN 1993-1-5:2006 limits the maximum length of stiff bearing, s_s to be no larger than the depth between flanges, h_w. This limit was not observed when the tabulated values were calculated.

As an example, consider a 203 × 133 × 25 UB. The depth between flanges, h_w is 203.2 – 2 × 7.8 = 187.6mm. The stiff bearing length, s_s should have been limited to this maximum value – but the calculated resistances use the tabulated values including lengths between 200mm and 350mm. The effect of the omission varies with beam size, weight and steel grade. Only beams of 356 serial size depth and smaller are affected, since for deeper sections, h_w > 350mm.

For cases when s_s exceeds hw, it is conservative to use the values when ss ≤ h_w. For the 203 UB example, the tabulated resistances for s_s = 150mm, being less than 187.6mm, will be conservative.

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AD 539:
Locking devices for fasteners subject to vibration

SCI is occasionally asked about nuts coming loose if joints are subject to vibration.

A number of solutions are possible, of which the following are examples. Each application should be carefully considered and an appropriate solution chosen to satisfy the design requirements.

Solutions include:

- Simple spring washers, which are a ring of square or rectangular cross section which has been cut and deformed into a spiral. Spring washers introduce a small preload into the fastener and may also "bite" into the nut and component surface, inhibiting rotation.
- Counter nuts (often referred to as Palnuts®), which are manufactured from relatively thin sheet steel. They have a number of protruding tabs which flatten and lock in place during their installation. In structural steelwork Counter nuts may be used (for example) to prevent bolts vibrating loose during transportation.
- Wedge lock washers, which comprise a mating pair of washers with cams on their mating surfaces and serrations on the external faces of the pair. The serrations on the external faces of the mating washer pair bind against the nut face and component surface. If the nut were to rotate, the overall thickness of the washer pair must expand due to the wedge effect, increasing the reaction between the nut and component, which introduces preload in the fixing to stop the nut loosening.
- Nuts with nylon inserts, which increases friction on the threads.
- Lock nuts, which may also be known as half nuts, and as the name suggests have a height approximately half the height of a standard nut. Half nuts should be plated on the bolt first, followed by the standard nut.
- Various bespoke specialised lock nuts, not commonly used in the UK, but well-known in the United States. These include nuts with special cuts, so that the nut deforms into the threads of the bolt, nuts with an internal wedging action similar to wedge lock washers, nuts with serrated faces and nuts with a locking pin which deforms the threads to prevent removal.
- Using preloaded assemblies.

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AD 541: Imposed roof loads in combination with PV

Photovoltaic arrays (PV) are becoming common on roofs. They're increasingly being added to new structures and retrofitted to existing ones.

The UK National Annex to BS EN 1991-1-1 specifies the imposed load on roofs with a slope less than 30° to be 0.6 kN/m². Within the Eurocode system, the snow load is a separate variable action.

It would seem extremely unlikely that an imposed roof load of 0.6 kN/m² would exist over the entirety of a roof at the same time as the PV. Some level of load must however be allowed for, since it is to be expected that the PV will need maintenance. SCI recommends that when the PV load and imposed roof load are considered in combination, the imposed roof load should be taken as 0.4 kN/m². The value of 0.4 kN/m² has some provenance, since it is the recommended value of imposed load on the roof in BS EN 1991-1-1.

The combinations to be considered include:

Permanent + roof imposed load at 0.6 kN/m²
Permanent + PV + roof imposed load at 0.4 kN/m²
Permanent + PV + snow

Further combinations including wind will be necessary. It should be noted that the roof imposed load is not considered in combination with either snow or wind (BS EN 1991-1-1 clause 3.3.2 (1)). PV may be considered as a permanent action.

If they are not flat on the roof slope, snow drifting within and around PV arrays should be considered. The draft version of BS EN 1993-1-3 proposes an increased snow load shape coefficient covering the area of the "tilted" panels and a distance all around the array. Some aspects of the requirement may be modified by the National Annex so the final detail may change, but the principle is clear.

When more than one variable action is included in the combination, one variable action should be identified in turn as the "leading" variable action. The remaining variable actions attract their respective ψ_0 factor.

The weight of the PV should be carefully determined, including the supporting structures and ballast. A nominal weight of 0.15 kN/m² should not be assumed as weights of 0.35 kN/m² have been reported.

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AD 543: Deflection of composite slabs

Occasionally, SCI's Advisory Desk is asked about the simplified rules for controlling the deflection of steel-concrete composite slabs by limiting their span-to-depth ratios. The question relates to what one should do when these ratios are exceeded. The purpose of this note is to provide clarification.

Limiting span to depth ratio

Clause 9.8.2 (4) of BS EN 1994-1-1:2004 permits calculations of the deflection of composite slabs to be omitted if both the following conditions are satisfied:

- the span to effective depth ratio does not exceed the limits given in clause 7.4 of EN 1992-1-1:2004, for lightly stressed concrete, and
- the load causing an end slip of 0.5 mm in the (long span) tests (used to determine the level of shear connection) on composite slabs exceeds 1.2 times the design service load. In reality this is something that a designer is likely to just assume, as it is highly unlikely they will have access to the manufacturer's test results.

The span to depth limits of BS EN 1992-1-1:2004 are specified in Table 7.4N where the UK NA to EN 1992-1-1:2004 refers to Table NA.5. The ratios in BS EN 1992-1-1:2004 are based on the effective depth, which for a composite slab with steel sheeting as reinforcement, is the distance from the top of slab to the centroid of the profile. The limits for different span conditions are shown in Table 1.

Table 1: General rules for the slab maximum span-to-depth ratios in accordance with BS EN 1992-1-1:2004

	Normal weight concrete	Lightweight concrete
Single spans	20	18.8
End spans	26	24.5
Internal spans	30	28.3

It's noted that the second-generation of EN 1994-1-1 only includes a single value of 26 for the limiting ratio of the span to the effective depth for normal weight concrete. Whilst the second generation of EN 1992-1-1 have new span to effective depth limits with a wider range of applicability, the rules in EN 1992-1-1 are no longer referred to in the second-generation of EN 1994-1-1, which only gives one value presumably as a simplification.

Where the limits of Table 1 are exceeded, the deflections should be calculated.

Calculating deflections

In accordance with clause 9.8.2 (3) of BS EN 1994-1-1:2004 deflections due to loading applied to the composite member should be calculated using elastic analysis.

Although end continuity is ignored for ULS verifications, for SLS it may be taken into account. Clause 9.8.2 (5) of BS EN 1994-1-1:2004 states that for internal spans that this can be achieved by using an average of the cracked (concrete below the neutral axis is ignored) and uncracked second moments of area. We recommend that this approach may also be used for end spans. For single spans a more complex and accurate approach may be justified, although it is very unlikely to be a critical check.

For typical building structures, the effects of creep may be taken into account by replacing concrete areas, A_c by effective equivalent steel areas A_c/n, where n is the modular ratio. The value of n allows for a typical combination of short-term and long-term loading.

where:

$$n = 2\frac{E_s}{E_{cm}}$$

E_s is the modulus of elasticity of structural steel
E_{cm} is the modulus of elasticity for the concrete

Traditionally, the modular ratio has been taken as 10 for normal weight concrete and 15 for light weight concrete.

Although clause 9.8.2(3) of BS EN 1994-1-1:2004 states that deflections of slabs should be calculated neglecting the effects of shrinkage, the second-generation of EN 1994-1-1 now requires that the additional deflections caused by shrinkage are included, and provides simplified formulae for single span slabs and continuous slabs:

$$\delta_{sh} = 0.15\varepsilon_{sh}\frac{L^2}{h} \text{ for single span slabs}$$

$$\delta_{sh} = 0.1\varepsilon_{sh}\frac{L^2}{h} \text{ for continuous slabs}$$

where
ε_{sh} is the shrinkage strain of the concrete
h is the overall depth of the composite slab
L is the span length

Subclause 3.1(4) of BS EN 1994-1-1:2004 states that where composite action is taken into account in buildings, the effects of autogenous shrinkage may be neglected in the determination of stresses and deflections. For dry environments within buildings, Annex C of BS EN 1994-1-1:2004 states that the total final free shrinkage strain may be taken as:

$\varepsilon_s = 325 \times 10^{-6}$ for normal weight concrete
$\varepsilon_s = 500 \times 10^{-6}$ for lightweight concrete

Composite slabs are normally unpropped during construction and the sheeting alone resists the self-weight of the wet concrete and construction loads. So if/when checking total deflection the part of the deflection due to the self weight of the slab is determined based on the stiffness of the sheeting, whereas that due to imposed loads is based on the stiffness of the composite slab.

If the sheeting is propped, the deflections will be greater the earlier the props are removed due to the lower stiffness of the 'immature' concrete. This immaturity would need to be reflected in a higher modular ratio.

Eurocode 4 does not specify deflection limits for composite slabs. BS 5950-4 gives a limit of $L/350$ or 20mm for the deflection of a composite slab due to imposed loads. Deflection due to the total load (less the deflection due to the self-weight of the slab plus, when props are used, the deflection due to prop removal) should be limited to $L/250$. When considering whether the deflection is acceptable, it may be necessary to consider the deflection of the supporting beams.

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AD 544: Washers for preloaded bolting assemblies

The SCI Advisory Desk sometimes receives questions about washers for preloaded bolting assemblies. When preloaded bolts are tightened and unhardened washers are used, if the contact pressure acting on the joint is too high, the washers may indent and deform when the bolt is fully tightened. If this occurs after the tightening is completed, the bolt preload will reduce, potentially affecting the integrity of the joint[1]. Clause 8.2.4 of BS EN 1090-2:2018[2] and clause 6.3.1 of the NSSS[3] require that hardened washers are to be used as follows:

- For property class 8.8 bolts a washer shall be used under the bolt head or the nut, whichever is to be rotated;
- For property class 10.9 bolts used with steel grade S235 washers shall be used under both the bolt head and the nut (it is noted that the use of steel grade S235 is highly unusual in the UK);
- For property class 10.9 bolts used with steel grades above S235, washers shall be used under the bolt head or the nut whichever is to be rotated, unless the use of washers under both the bolt head and the nut is specified.

Hardened plain chamfered washers according to BS EN 14399-6:2015[4] shall be used under the heads of preloaded bolts and positioned with the chamfer towards the bolt head and towards the nut when fitted under the nut. Hardened plain (unchamfered) washers according to BS EN 14399-5:2015[5] shall only be used under nuts (because of the corner radius between the bolt shank and head).

Both BS EN 14399-5:2015 and BS EN 14399-6:2015 note that washers according to those standards are not intended to be used in direct contact with oversized or slotted holes. As explained in AD 522, plate washers shall be used for connections with slotted and oversized holes. Hardened washers (as described above) are required on top of the plate washer.

1 *Joint Bearing Pressure and the Use of Unsuitable Washers*, Bolt Science, October 2024
2 BS EN 1090-2:2018 *Execution of steel structures and aluminium structures*, BSI
3 *National Structural Steelwork Specification for Building Construction 7th Edition*, BCSA
4 BS EN 14399-6 *High-strength structural bolting assemblies for preloading - Plain chamfered washers*, BSI
5 BS EN 14399-5 *High-strength structural bolting assemblies for preloading - Plain washers*, BSI

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AD 546: Critical temperatures for compression members in the UK NA to BS EN 1993-1-2

The UK NA to BS EN 1993-1-2 contains values for critical temperatures for compression members, presented in Table NA.1 as a matrix of values for combination of non-dimensional slenderness λ and utilisation μ_0. The non-dimensional slenderness is the value at ambient temperature; the utilisation is the value in the fire limit state.

The critical temperatures assume that the relevant column buckling curve at ambient is curve 'c' and the imperfection factor α is therefore 0.49 (see BS EN 1993-1-1 Tables 6.1 and 6.2). This assumption is appropriate for most UC sections in S355, buckling in the minor axis, as would commonly be found in multi-storey buildings.

If the imperfection factor is less than 0.49, the critical temperatures in Table NA.1 are not conservative. Common situations where the imperfection factor is less than 0.49 include:

- UC sections in S460 (α = 0.21)
- UB sections in S355 with t_f < 40mm and buckling in the minor axis (α = 0.34)
- UB sections in S460 (α = 0.21 or 0.13)
- Hot finished hollow sections in S355 (α = 0.21)

If used as columns within multi-storey buildings, an additional factor may be applied to reduce the buckling length at elevated temperatures which will have a beneficial effect (see BS EN 1993-1-2 clause 4.2.3.2(5)). The reduced buckling length is $0.7L$ for intermediate storeys, where L is the storey height.

If the reduced buckling length of $0.7L$ is used, the NA values are conservative for columns in multi-storey buildings, even when the imperfection factor is less than 0.49. However, for other members in compression, such as found in trusses and bracing, designers should note that Table NA.1 is not appropriate.

For the common cases of hot finished hollow section compression members in S355 where α = 0.21, replacement values are given in the table below.

Tables of critical temperature for other combinations of steel design grade and imperfection factor may be downloaded from Steelbiz.

The 6th Edition of the ASFP "Yellow Book" limits the utilisation μ_0 to no more than 0.6 and in Table 7 (and Table B.1) recommends a critical temperature of 500°C for all compression members. This temperature of 500°C is the lowest in Table NA.1 for a utilisation of 0.6. The ASFP critical temperature of 500°C is for "Column (including hollow columns), trusses and other bracings" and "Compression members of any shape". Because the ASFP table adopts the value from the UK NA, the same concern applies – the ASFP recommended value is not conservative if α < 0.49.

As can be seen from the replacement table when α = 0.21, a more onerous temperature of 391°C would be appropriate for hot finished hollow sections in S355.

Designers should note that the UK NA and ASFP guidance is only appropriate for UC sections buckling in their minor axis, with an imperfection factor of 0.49.

α = 0.21 and S355	Critical temperature (°C) for utilisation factor μ_0					
Non-dimensional slenderness	0.7	0.6	0.5	0.4	0.3	0.2
λ = 0.4	461	513	551	589	637	688
λ = 0.6	412	482	530	571	617	673
λ = 0.8	251	436	510	553	597	657
λ = 1.0	192	403	497	543	588	647
λ = 1.2	211	391	495	542	585	644
λ =1.4	241	399	501	544	586	644
λ = 1.6	266	408	504	546	588	646

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AD 548:
High shear regions for large web openings as defined in SCI P355

SCI's Advisory Desk has been asked about the definition of a 'high shear' region in SCI P355 *Design of Composite Beams with Large Web Openings*.

In P355, Table 2.1 provides practical geometric limits for beams with web openings. It includes limits on the maximum stiffened and unstiffened opening lengths and the minimum width of the web-post. Different limits are specified for 'high shear' and 'low shear' regions, with stricter limits required for openings in high shear regions.

A note below the table says that "A high shear region is where the design shear force is greater than half the maximum value of design shear force acting on the beam".

$$\frac{V_{\mathrm{ed}}}{V_{\mathrm{Ed,max}}} > 0.5$$

Examples of high and low shear regions for simply supported beams with uniformly distributed loads, and with point loads are shown in Figure 1. Notably the determination of the 'high shear' region bears no relation to the shear capacity of the beam itself.

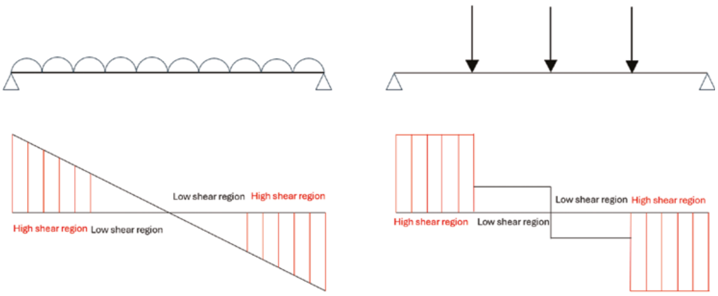

Figure 1: Examples of high and low shear regions for simply supported beams

The practical geometric limits given in Table 2.1 were derived based on typical composite beam designs with large web openings. The 'low shear' limits allow for larger openings and narrower web-posts. In addition, as suggested in P355, the geometric limits given in Table 2.1 are practical limits for beams within the scope of the publication. Openings that exceed these limits may be used, based on the guidance given in the publication, provided the design is justified by appropriate calculations. Therefore, the limits in Table 2.1 may be exceeded if the engineering checks are met.

It's noted that the new second generation of Eurocode, BS EN 1993-1-13:2024 *Beams with large web openings* includes similar geometric limits for unstiffened and stiffened web openings; however, no distinctions between high and low shear exist. In contrast to P355, the limits provided in BS EN 1993-1-13 apply in all cases unless the National Annexes (NA) permit otherwise. Work on the NA will start shortly.

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AD 549:
Steel decking and composite slab span types

Design software, or load-span information, provided by floor decking manufacturers will normally be used to verify the steel decking and composite slab, as its performance during construction and after completion is complex and certain design parameters are best determined from tests. It is not normally necessary for designers to understand the design methodology in detail, although the principles should be well understood. It's particularly important that the spanning conditions of the decking and the spanning conditions of the composite slab, the two of which may well be different, that are used in the design reflect what will be built on-site. The purpose of this AD note is to explain the different span types of the decking and spanning conditions of composite slabs, and how they affect the design of the steel decking during construction and composite slab after completion respectively.

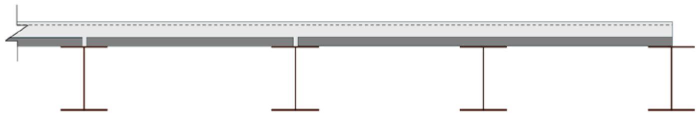

Figure 1: Different steel decking and composite slab span types

Decking span types

Single-span decking

This arrangement consists of a single steel deck that spans between two supports with no continuity over either support.

Double-span decking

This arrangement consists of a single steel deck that spans over three supports comprising two bays, with continuity over the internal support.

Multi-span decking

This arrangement consists of a single steel deck that spans over four or more supports comprising three or more bays.

Slab span types

Single

In this condition, the reinforced concrete is not continuous over either support.

End

In this condition, the reinforced concrete is continuous over one support.

Internal

In this condition, the reinforced concrete is continuous over both supports.

Construction stage

The construction stage concerns the design of the decking. Only the decking span type is relevant at this stage. The reaction forces, shear forces, bending moments and deflections are calculated by determining the critical load case for the applied actions considering the appropriate span type. Multiple load cases may need to be considered for ultimate limit state and the serviceability limit state, positioning imposed loads to maximise bending, shear, or the combination of the two. The number of cases will depend on the span type.

Normal stage

The normal stage concerns the design of the composite slab. For the ultimate limit state, composite slabs are usually designed as single span simply supported members, with no account taken of any hogging resistance resulting from reinforced concrete that is continuous over the supports. This is the case whether there is physical continuity or not, and has nothing to do with the span type of the decking during construction.

When determining the deflection of a composite slab for the serviceability limit state, when the slab is either 'end' or 'internal' the continuity at one or both ends respectively may be taken into account by using an average of the cracked and uncracked second moments of its area. For single spans the uncracked value may be used.

Fire stage

This stage concerns the design of the composite slab. Unlike the normal stage design, any physical continuity of the slab over internal supports is taken into account.

For single spans, which have no end continuity at either side, only the sagging moment resistance is considered. In such cases a bar will be needed in the troughs to ensure the sagging resistance is adequate. Design on this basis is sometimes called the Bar Method.

For end spans, which have continuity at one end, the sagging resistance is enhanced by allowing for hogging moment resistance at that end. The addition of hogging resistance means that the sagging resistance may be adequate even without bars in troughs. Design on this basis is sometimes called the Mesh and Deck Method.

For internal spans, which have end continuity at both ends, the sagging and hogging resistances are combined to determine total moment resistance.

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AD 550: Stiffness classification for welded beam to column joints

If a bolted beam to column joint is to be classified as "rigid", practical recommendations are contained in the Green Book on moment-resisting joints (P398). To be classified as rigid, the critical mode for the top row of bolts is to be Mode 3 (bolt resistance is critical rather than modes involving flexure of the plate) and the column web panel shear force must not exceed 80% of the design shear resistance.

This AD provides complementary advice for welded beam to column connections, which in due course will appear in a revised version of the Green Book.

For a welded beam to column connection to be classified as rigid without recourse to calculations or analysis by software, the joint should meet the following requirements:

- Stiffeners, of equal (or greater) width and thickness as the beam flange, should be provided across the full width of the column web.
- The beam to column flange welds should be of equal strength to the beam flange, and the welds between the stiffeners and the column flange should have an equivalent resistance to the beam flange welds.
- The column web panel shear force must not exceed 80% of the design shear resistance.

Alternatively, by using limited calculations, the following approach may be adopted:

- The detail should satisfy the requirements for welding to unstiffened flanges, or stiffeners aligned with the beam flanges should be provided in the web of the column. Although rules for welding to unstiffened flanges are provided in BS EN 1993-1-8, the UK Connections Group recommend that the requirements given in clause 6.7.5 of BS 5950 be adopted.
- When stiffeners are required, they should be of equal (or greater) width and thickness as the beam flange, unless smaller stiffeners are proven by calculation.
- The welds to the beam flange should be designed for the applied loads applied over the effective width of the flange, or may be sized to be of equal strength to the beam flange.
- The welds between stiffeners and the column flange should have an equivalent resistance to the beam flange welds.
- The column web panel shear force must not exceed 80% of the design shear resistance.

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