TECHNICAL DIGEST 2024

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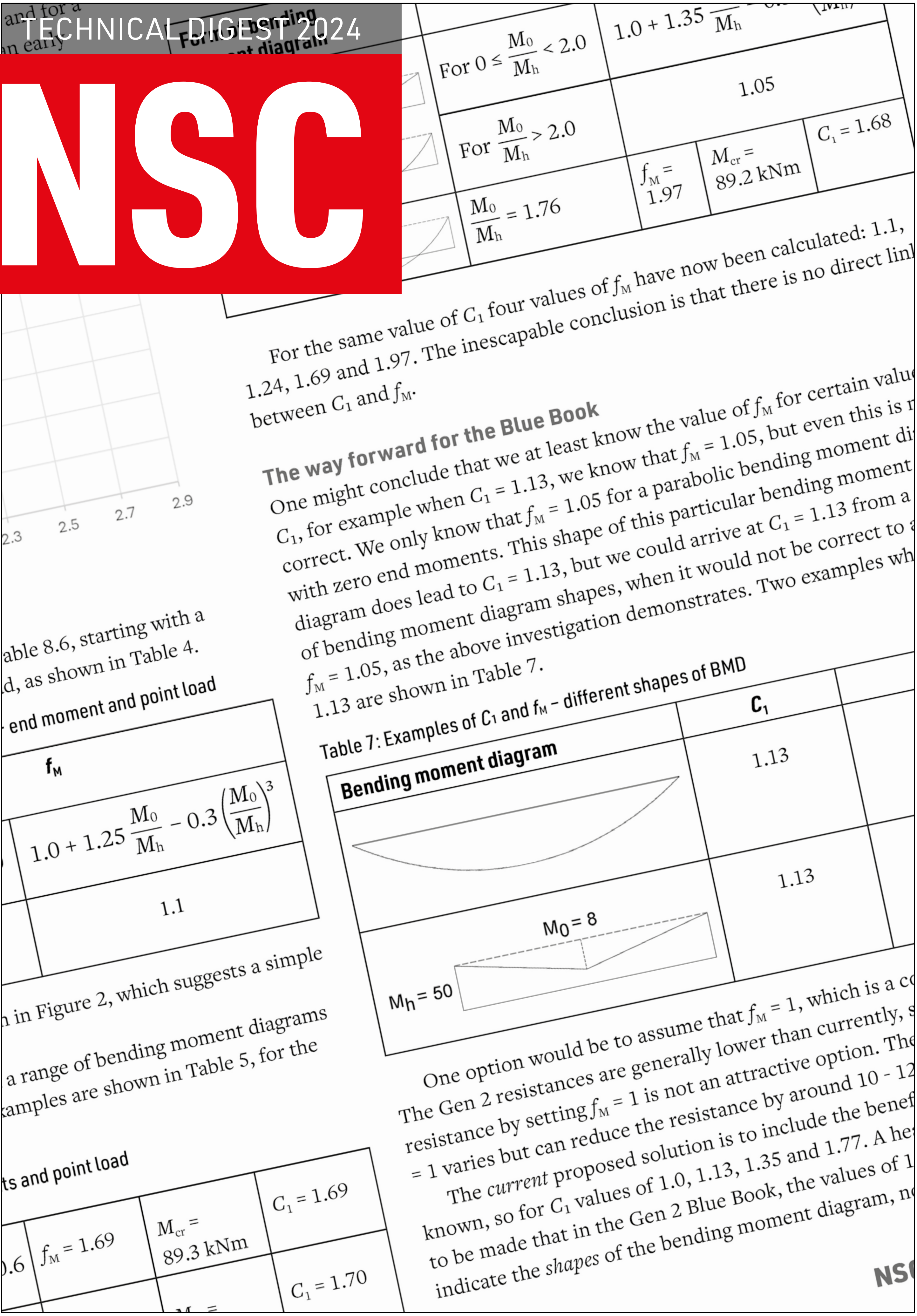

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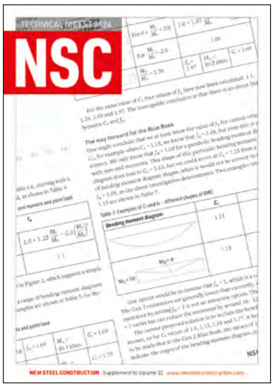

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BCSA

STEEL for life

SCI Steel Knowledge

EDITOR
Nick Barrett Tel: 07973 325417
nick@alignmentmedia.co.uk

DEPUTY EDITOR
Martin Cooper Tel: 07966 904599
martincoopernsc@gmail.com

PRODUCTION EDITOR
Andrew Pilcher Tel: 07365 919818
andrew@alignmentmedia.co.uk

COMMERCIAL MANAGER
Kirsty Barrett Tel: 07525 253316
kirsty@alignmentmedia.co.uk

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

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

Steel for Life Ltd
4 Whitehall Court, Westminster, London SW1A 2ES
Telephone 020 7839 8566

The Steel Construction Institute
Unit 2, The E Centre, Bracknell, Berkshire RG12 1NF
Telephone 01344 636525 Fax 01344 636570
Website www.steel-sci.com
Email hello@steel-sci.com

CONTRACT PUBLISHER & ADVERTISING SALES
Alignment Media
7 Linden Close,
Tunbridge Wells, Kent TN4 8HH
Telephone 07973 325417

Essential steel construction technical advice for designers

Nick Barrett - Editor

This is the ninth 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 the ten Advisory Desk Notes and Technical Articles published in NSC during 2024 in a format that is available as a free downloadable pdf at the steelconstruction.info website, or for online viewing.

The Digest is part of the steel construction sector's long-established commitment to keep designers in steel up-to-date with the latest technical guidance, 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.

Design guidance and other key steel construction information including details of how the steel construction sector is supporting the drive towards net zero carbon is always easily accessible, either 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 Digests of value.

Reaching for the asymptote – what is an optimal design?

Greatly simplifying the complex subject of structural design, one would imagine that an optimally designed element, or structure, would achieve unity factors of 1.0 for all verifications using the minimum amount of material. That may be true if embodied energy is our only concern, and we consider material use as a metric for 'carbon'. It probably isn't true if we are concerned with cost, as there are many reasons why a lower cost solution might well use more material to achieve the same, or better, resistance. Organisations such as SCI have been preaching that 'lowest weight is not lowest cost' for three decades.

Using some examples from the very topical subject of performance in fire, in this article Graham Couchman of SCI will illustrate how different definitions of optimal could lead to very different solutions. The purpose of the article is to encourage designers, and indeed those who procure and construct buildings, to understand what they want and be open to different approaches.

Introduction

Guidance on how to design for more sustainable construction is due for publication by SCI in early 2024 [1]. Its production acknowledges that for the first time in a generation, designers are now having to consider metrics other than (initial) financial cost when striving for an optimal solution. Having spent a career with the mantra of 'lowest weight is not lowest cost' it requires a serious change in mindset. This change of mindset will also require clients to change their approach of choosing the designer proposing the lowest fees, as lowest design fees will rarely lead to the lowest cost solution. The latter point has been well demonstrated by work undertaken at the University of Cambridge in recent years [2], showing how much steel is wasted as a result of reducing design time. Having to consider different definitions of optimum certainly complicates things. Is a 'great' design one that costs the least to build, one that costs the least to design (some aspects of which are what we have traditionally got as a result of how contracts are organised), one that uses least material, one that will be adaptable in the future, or reusable, and so the list goes on. The examples below, both of which relate to design for the fire limit state, illustrate design choices that could be made, and should be informed by the definition of 'optimal'. One is obvious, the other is more subtle.

Composite slabs – is all that concrete needed?

Composite slabs are designed considering three very different scenarios:

- The ability of the decking to support its self-weight, the wet weight of concrete, the self-weight of any mesh, and construction imposed loads,
- The ability of the composite slab, comprising in-situ concrete with mesh and/or fibres and steel decking acting as external reinforcement, to support normal state loads,
- The ability of the composite slab, with resistance reduced due to elevated temperatures, to support reduced loads at the fire limit state.

For the first two scenarios both resistance and stiffness of the decking and slab respectively must be adequate. Stiffness is not explicitly considered for fire design. In over 80% of cases the first of the scenarios above, the construction stage, governs. This indicative figure is for the UK market, despite our common practice of thru-deck welding of the shear studs, which facilitates making the decking sheets continuous and therefore undergoing significantly reduced deflections compared to simply supported sheets. When the ability of the decking to support construction dead and imposed loads limits the spans that can be achieved, it is clear there is not point in say adding depth to the slab in order to increase the composite resistance. Indeed that would be counterproductive as it would add to the wet weight of concrete. Table 1 distinguishes the spans that can be achieved for each design condition for a typical slab with the attributes below:

- 150mm total depth (composite stage assumes, as is almost universally done in the UK, that the slab is discontinuous at both ends despite the physical reality)
- 60mm trapezoidal decking, 0.9mm gauge (assumed continuous at one end)
- REI 60 minutes (concrete assumed to be continuous at both ends)

Consideration	Maximum span (m)	Governing verification
Construction	3.17	Deflection[1]
Normal composite	4.91	Moment resistance
Fire (composite)	4.06	Moment resistance

[1] It is worth noting that whilst deflections are considered an SLS check, deflections of decking at the construction stage may increase the depth of slab and have 'ultimate' consequences.

Table 1 – Spans that can be achieved for the construction, normal composite and fire stages for a typical slab

However, it is worth considering why the design of an element such as a composite slab is so often governed by the construction stage. Developments in decking profiles over the past 25 years have been driven by a desire to reduce the volume, and therefore wet weight, of concrete. The difference in the volume of the voids formed in a slab with either nominal 50mm re-entrant decking or nominal 60mm trapezoidal decking is immediately clear when the decking geometries are considered (Figure 1). However, to achieve adequate performance in fire requires a certain depth of concrete. Requirements for acoustic attenuation may also dictate the depth (mass) of the slab. In fire, the depth is needed to ensure that the unexposed upper surface of a slab with a fire below stays below a certain temperature (to satisfy the insulation criterion in Approved Document B[3]). In some situations, such as short spans with low imposed loads at the final stage, a more optimal solution might use less concrete, and other means of assuring fire and acoustic performance.

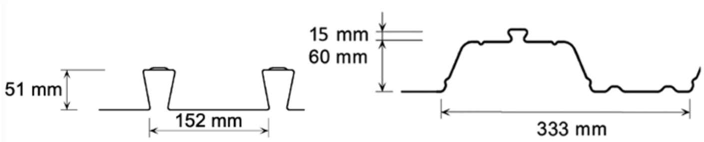

Figure 1 – Cross-sections of a nominal 50mm re-entrant deck and a nominal 60mm trapezoidal deck

Depending on what metric defines optimal, this could be through the provision of extra layers rather than just increasing the thickness of concrete. So the real condition governing design in numerous cases is actually fire, but this is not transparent as it manifests itself in software etc. as a 'minimum' acceptable slab depth that then impacts construction stage design.

Composite beams – what performance is important?

When considering the fire protection of a composite beam, the cross-section is typically broken down into elements. BS EN 1994-1-2 Clause 4.3.4.2[4] gives rules for un-encased downstand beams and notes how the section should be divided into parts (flanges, web, slab), assuming no heat transfer between them (Figure 2). Various sub clauses identify how to calculate the increase in temperature of a given part, as a function of time. Having established the temperature of each part for the required time period (the temperature varies, amongst other things, with the ratio of surface area to volume of the part), the amount of fire protection can then be determined. Clause 4.3.4.2.2 (9) considers the issue of transverse decking laid on the beam, and recognises that when voids thus form above the steel beam top flange, only part of which is then in direct contact with the slab, their 'size' can affect the flange temperature. The code assumes that more than 85% direct contact between beam and slab is as good as total contact, providing protection to the upper surface of the flange, whereas less than 85% leaves the flange 'exposed' (unless the voids are filled with appropriate material). Software from proprietary beam manufacturers recognises this, and uses an approach whereby temperatures in the top flange are 'adjusted' to allow for more than 15% voids. These higher temperatures must be addressed by using increased fire protection.

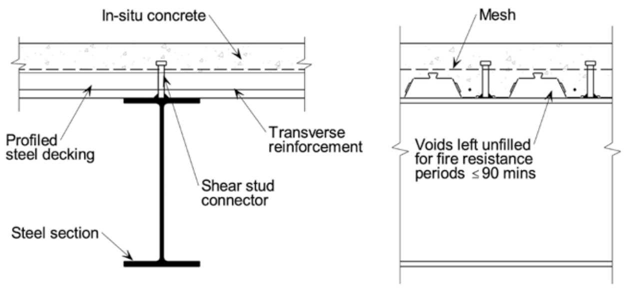

Figure 2 – Typical composite beam cross-section and elevation

Table 2 is reproduced from SCI's recently updated publication P300[5] and presents simplified rules for achieving the required level of fire protection when voids are unfilled, for different fire resistance periods. It shows that for 90 minutes a considerable increase in protection may be required. This could have significant and not obvious consequences on cost/programme if it resulted in a need for an additional coat of offsite intumescent, because of the requirements for drying time.

However, as noted the procedure described above is based on considering each part of the composite cross-section in isolation, and ignores the fact that some parts are more important than others. The upper steel flange of a composite beam is rarely important, post-construction stage, because the concrete slab carries most if not all of the compression. This is why composite beams often have asymmetric steel flanges. Moreover, when there are large web openings the design of the beam is likely to be governed by web-post buckling, not compression flange (steel and/or concrete) resistance. So why waste fire protection material to ensure a level of performance of a cross-section part that is not needed? The answer is ease of design, sometimes combined with a lack of understanding.

When sufficient fire protection is used, it ensures that ambient temperature design will govern, so elevated temperature structural design is not explicitly needed. This is achieved when the reduction in resistance with temperature is less than the reduction in load factors when fire is considered. However, even without explicit design at elevated temperatures, it might be possible to produce a more efficient design for the scenario described above. The presence of higher temperatures in the upper steel flange, sufficiently high for fire design to govern, could be modelled in an ambient design by using a smaller upper flange, rather than reduced material strength, to represent a loss in resistance. If this smaller beam still worked then the higher temperatures at the fire limit state would not be a problem, and some of the fire protection could be 'saved' to provide a more optimal design. This conclusion is based on the assumption that any reduction in shear connection resistance with temperature would not be relevant. This assumption is not unreasonable given the reduction factors for studs and associated concrete given in BS EN 1994-1-2 Clause 4.3.4.2.5.

Conclusions

The design, construction, use and ultimately removal of a building at end of life are each in themselves complicated, and taken together that complexity increases due to the interactions between these different phases. It then becomes extremely difficult to identify what is an optimal design. Traditionally optimisation has considered the simple metrics of design fees, and more significantly construction cost. More recently whole life (financial) cost, and 'carbon cost', have come into consideration. This is therefore an interesting time for designers, who will be able to use their skills and experience to come up with solutions that are more broadly optimal. ■

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3. Approved Document B: Fire safety. The Building Regulations 2010. UK Government, 2022
4. BS EN 1994-1-2:2005 Eurocode 4. Design of composite steel and concrete structures. General rules. Structural fire design. BSI
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Trapezoidal deck				
Beam type	Fire protection on beam	Fire resistance		
		Up to 60	90	Over 90
Composite	Insulating sprays and boards (assessed at 550°C)	No increase in thickness	Increase thickness by 10 % or assess thickness using *A/V* increased by 15%*	Fill voids
Composite	Intumescent coatings (assessed at 620°C)	Increase thickness by 20% or assess thickness using *A/V* increased by 30%*	Increase thickness by 30% or assess thickness using *A/V* increased by 50%	Fill voids
Non composite	All types	Fill voids		
Dovetail deck				
Beam type	Fire protection on beam	Fire resistance		
		Up to 60	90	Over 90
Any	All types	Voids may be left unfilled for all periods of fire resistance		

* The least onerous option may be used (*A/V*=heated surface area per unit volume of the steel section)

Table 2 – Recommendations for fire protection of voids between decking and beam

Atmospheric corrosivity classifications for weathering steel

Weathering steel has an increased resistance to atmospheric corrosion, compared to conventional steel, and a distinctive appearance which make it attractive for use in bridges. However, the decision to specify weathering steel in bridge projects often depends on the environmental conditions and more specifically on the proximity of a bridge to the coast. In the UK, corrosivity testing in accordance with BS EN ISO 9223 for a minimum of 12 months is mandatory at distances of less than 15 km from the coast. Typically, UK procurement routes do not give adequate time to undertake and assess the results of this testing. In other countries testing may be required in specific situations.

This article discusses the findings of a study commissioned by SCI's Steel Bridge Group and carried out by Arup[1]. The objective of the study leading to this article was to provide an evidenced based approach for the classification/assessment of environments and propose scientifically justified limitations on the use of weathering steel in bridges (or requirements for site specific corrosivity testing) within a certain distance from the coast for the UK.

Introduction

Weathering steel is a low-alloy steel that under normal atmospheric conditions gives an enhanced resistance to corrosion compared with that of ordinary carbon-manganese steel. Weathering steel is specified to BS EN 10025-5[2] and has similar mechanical properties to conventional steel. In the presence of moisture and air, a rust layer is formed that adheres to the base metal due to the specific alloying elements used in the manufacturing process. This rust 'patina', which develops under conditions of alternate wetting and drying, acts as a protective barrier, impeding further access of oxygen, moisture and contaminants and effectively reducing the rate the steel corrodes.

Weathering steel bridges are generally suitable for use in most locations. However, as with other forms of construction, there are certain environments which can lead to durability problems. The performance of weathering steel in such extreme environments as marine environments and/or environments with very high levels of atmospheric pollution may not be satisfactory, and this should be considered.

Exposure to high concentrations of chloride ions, originating from sea water spray, salt fogs or coastal airborne salts, is detrimental. The hygroscopic nature of salt adversely affects the 'patina' as it maintains a continuously damp environment on the metal surface.

The scope and outcomes of the investigation leading to this article are summarised below:

- UK and international standards and guidance on the use of weathering steel near the coast was assessed.
- Data on chloride deposition with distance from the coast including those from ISOCORRAG[3] program, EUR 7433 Report[4], site specific test data and other relevant sources were collated.
- Available site-specific data were reported on a map and data were plotted as curves for a range of UK locations.
- The overall corrosivity of UK sites (based on categorisation of BS EN ISO 9223[5]) at increasing distances from the coast was estimated.

The methodology, data sets, plots and recommendations are reported in the following paragraphs.

Review of standards and international guidance

Weathering steel is widely used for bridge fabrication in the UK, continental Europe, North America, Australia, and New Zealand. Highway and Rail authorities in these countries have standards and guidance relevant to the use of weathering steel. The standards and guidance cover all aspects of steel fabrication, but this article only considers those parts relevant to durability and the potential need for assessment when a bridge is located near the coast. A summary of the requirements and limitations on the use of weathering steel in bridges in the UK, Germany, France, North America, Australia, and New Zealand is presented in Table 1.

Country	Minimum distance from the coast	Additional requirements
UK[6]	No minimum distance but not permitted in a corrosivity category of C5	For locations less than 15 km inland from the coast the salinity shall be determined via monitoring/testing for 12 months, and if found to be S3 (to BS EN ISO 9223[5]), i.e. chloride deposition rate Sd > 300 mg/(m².day), weathering steel cannot be used.
Germany[7]	0.5 km	
France[7]	1 km to 2 km	1 km refers to coasts in the Mediterranean, 2 km from the North Sea, English Channel and Atlantic Ocean.
North America[8]	2 miles (3.2 km) to 4 miles (6.4 km)	Depends on individual State Department of Transport (DOT)
Australia[9]	2 km	Corrosivity category C3 or lower according to ISO 9223†
New Zealand[10]	5 km	Corrosivity category C3 or lower according to ISO 9223; <5 km from the coast permitted provided site-specific assessment is undertaken for a minimum of 1 year; 'Coastal' weathering steel with addition of 1% or 3% Nickel may be used in C4 and C5 environments, respectively.

†Corrosivity categories are to Australian Standard AS 4312 but they are consistent with ISO 9223

Table 1 – Limitations of use of weathering steel in bridges

Chloride deposition data in the UK

The presence of chloride ions on the surface of steels increases the risk of corrosion. Chlorides are deliquescent and will absorb moisture from the atmosphere. This increases the time of wetness on the surface and the total period when corrosive processes are active. Chloride ions are present in natural water sources (particularly the oceans) and are made airborne by a combination of wave action and the wind blowing over the water. Airborne chlorides are then transported to and can deposit on surfaces at distance from the source.

A research report published in 1981 by the European Commission (EUR 7433[4]), summarises the results of a joint research project to provide directives for selection of atmospheric corrosion test sites and collect environmental data on the maritime atmospheric environment. Among other data presented, the chloride deposition rates were measured at 36 sites across Europe up to 5 km inland from the coast, 18 of these sites were in the UK. The aim of this element of the research was to gain an understanding of how rapidly the airborne salinity decreases with increasing distance from the coast.

Airborne salinity was monitored monthly using the wet candle method (the method now used in BS EN ISO 9225[11]) over a period of two years. The data from the EUR 7433 report[4], plotted as a line graph, shows the measured values for the chloride deposition rates at increasing distance from the coast. Figure 1 and Figure 2 show the data for UK sites split into those that fall west and east, respectively, of a central line of the UK. Lines in blue and red show the distance at which the chloride deposition rate falls below 150 and 300 mg/(m^2.day), respectively.

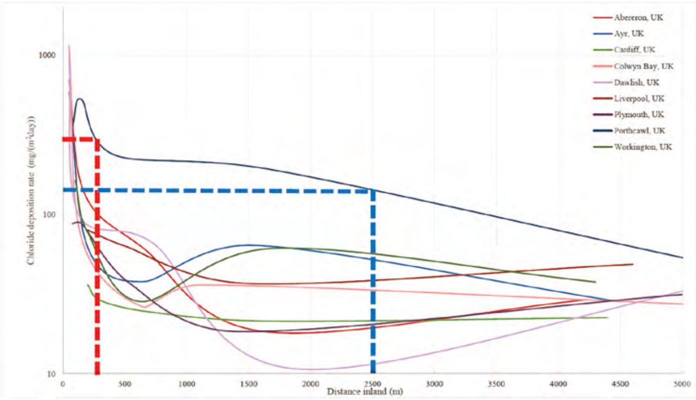

Figure 1 – Westerly UK site data collected in [4] of the chloride deposition rate with distance inland from the coast, plotted on a logarithmic scale

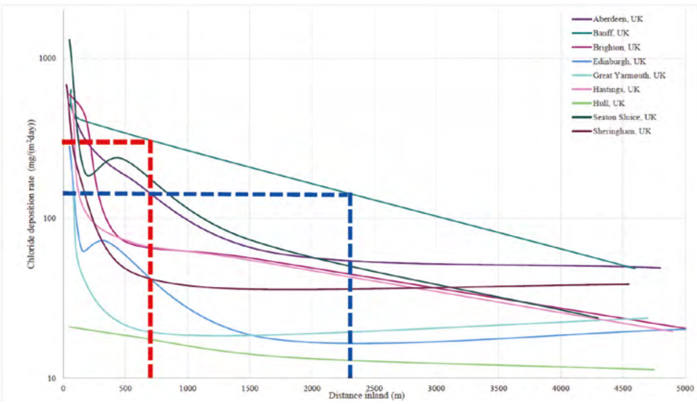

Figure 2 – Easterly UK site data collected in [4] of the chloride deposition rate with distance inland from the coast, plotted on a logarithmic scale

Some chloride deposition data points from EUR 7433[4] are higher/lower than expected at specific sites. These anomalies are caused by site specific conditions which affect the local (micro) environment at the measured distance. As an example, the chloride deposition may be lower than expected due to the monitoring site being in a sheltered location (by vegetation or adjacent buildings). Another example is the data presented in Figure 2 for Banff, which show that the chloride deposition varies linearly (as opposed to the exponential trend typically seen) between 150 and 4600m and this is because no site monitoring took place between these points for the specific case.

Further site-specific chloride deposition data has been obtained from different data sources which include:

- Transport and Road Research Laboratory (TRRL) report[12] on 'The corrosion performance of weathering steel in highway bridges' published in 1978, in which corrosivity assessments were carried out for weathering steel at various locations across the UK.
- ISOCORRAG 'International Atmosphere Exposure Program'[3], which formed the basis of BS EN ISO 9223 standard.
- Project specific data from previous Arup projects in the UK.

This data was found to sit within the extremes of the data reported in the EUR 7433 report[4] for the UK sites which was plotted in Figures 1 and 2.

The plots in Figure 1 and Figure 2 show that airborne chloride deposition decreases with distance from the coast:

1) At 200m inland the chloride deposition rates are less than the limit defined in BS EN ISO 9223 for an S3 category.
2) At 2500m inland the chloride deposition rate falls below 150mg/m^2.day.
3) Beyond 2500m from the coast the deposition rate continues to decay.

All the data demonstrate that chloride deposition rates rapidly decrease with increasing distance from the coast. At < 2500m inland, the chloride deposition rate is less than half that of the criteria given in DMRB CD 361[6].

Overall corrosivity of UK sites

Methodology

The atmospheric corrosion assessment for several UK sites was done according to BS EN ISO 9223[5], which uses data and statistical models developed as part of a global corrosion study to estimate corrosion rates. The ISOCORRAG study[3] used standardised samples and measurement methods to estimate corrosion rate over time and correlated those rates with environmental parameters. Corrosion rates were evaluated by weight loss at annual intervals. Samples were exposed at sites around the world.

The BS EN ISO 9223 standard is concerned with the classification of corrosivity of atmospheres based on the first-year corrosion rate for various metals. Such classification requires twelve-month exposure of relevant test specimens. The standard also includes other methodologies to estimate atmosphere classification, i.e. with\out a twelve-month exposure trial. In this report, estimation of corrosivity uses a semiquantitative interpretation of the BS EN ISO 9223 Dose Response Function (DRF) using environmental input parameters.

Four environmental parameters are used to assess likely corrosion rates:

- Average annual air temperature (T)
- Average annual relative humidity (RH)
- Average annual deposition rate for sulphur dioxide (P_d)
- Average annual deposition rate for chloride (S_d)

These parameters permit the estimation of a corrosion rate for the first year of exposure using a statistical model specified in BS EN ISO 9223. An additional environmental parameter to consider is wind direction. Wind influences the transport and deposition rate of sulphur dioxide and chlorides. Where site specific values for these are not known, the wind data can be used in predicting likely exposures.

The methodology provides an overall, or macro, assessment of corrosion risk based on the general corrosion rate i.e. uniform loss of section over the

1 Aberaevon
2 Aberdeen
3 Ayr
4 Banff
5 Brighton
6 Cardiff
7 Colwyn Bay
8 Dawlish
9 Edinburgh
10 Great Yarmouth
11 Hastings
12 Hull
13 Liverpool
14 Plymouth
15 Porthcawl
16 Seaton Sluice
17 Sheringham
18 Workington
19 Rye
20 Red Lodge
21 Kings Lynn
22 Carville
23 Bradbury
24 Ffridd Farm, Machnylleth
25 Fflat Stiwdio, Machnylleth
26 Bridge Street Overbridge, Newport
27 M4/A48M Interchange, Castleton
28 Liberty Steel, Newport Docks
29 M4 Junction 23A Westbound, Magor
30 Tintagel
31 A30 Intechange, Tolvaddon
32 Eastney
33 Iden bridge
34 Silverdale
35 Portishead
36 Northumberland

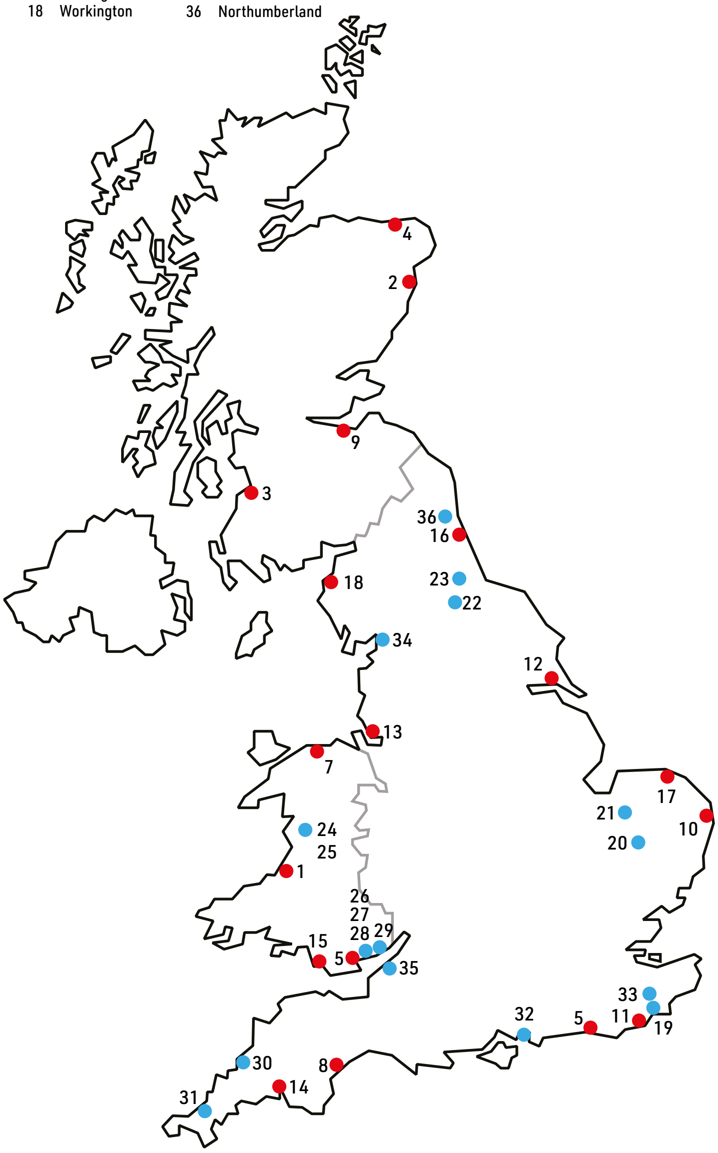

Figure 3 – Location of UK monitored sites

surface exposed to the natural environment. There is a degree of uncertainty associated with the estimation of atmospheric corrosivity, which BS EN ISO 9223 estimates as - 33% and +50% for zinc, carbon steel and copper, and -50% and +100% for aluminium. The corrosion rate calculated in accordance with BS EN ISO 9223 assumes uniform corrosion.

Site locations

The 36 sites considered are shown in Figure 3. Those in red are from EUR 7433[4] and those in blue are from the TRRL report[12], ISOCORRAG[3] and other project specific reports.

Average annual temperature and relative humidity

The BS EN ISO 9223 standard uses annual average temperature and humidity as its temperature and humidity parameter. The average annual temperature and relative humidity for the sites considered was obtained from UK Met Office[13] where climate averages over a rolling 30-year period with the most recent averaging period being 1991-2020 are reported.

Average annual deposition rate for sulphur dioxide

Historically, the most common source of SO_2 emissions were from coal fired power plants, refineries, heavy industry, vehicle exhaust emissions and shipping in ports. Following UK and European emission control legislation in the late 20th Century atmospheric SO_2 concentrations have dramatically declined in recent decades. DEFRA routinely collected sulphur dioxide concentrations in the air in various parts of the UK until circa 2005 when the concentrations had declined to concentrations that were nearly undetectable with the conventional measurement technique. It was recommended as a result that the measurements ceased.

SO_2 deposition rates were used from monitoring data where possible. When data was not available, sites were qualitatively assessed for any nearby SO_2 sources and SO_2 deposition was defined based on the relative distance, topology and wind direction. In general, for sites near ports, harbours and industrial facilities an upper bound urban atmosphere (P_1) was conservatively assumed. Sites located in urban areas with no nearby SO_2 sources, were classified on the boundary of a rural (P_0) and urban (P_1) atmosphere. Rural areas assumed a lower bound rural (P_0) atmosphere, the value of which is defined by the Dose Response Function equation. BS EN ISO 9223 groups pollution by sulphur dioxide into four categories (P_0 to P_3).

Average annual deposition rate for chlorides

The average annual chloride deposition for the assessed sites at distances of 300 to 5000m were obtained from the EUR 7433 data[4]. At 5000m and greater the chloride deposition has fallen to S1 category of BS EN ISO 9223.

Atmospheric corrosion assessment

BS EN ISO 9223 provides a model that uses the location and the previously mentioned environmental parameters (temperature/humidity and deposition rates for sulphur dioxide and chlorides) to estimate the corrosion rate at a site for the first year of exposure, by use of a statistical dose response function. The first-year corrosion rate also defines the site's corrosivity category.

BS EN ISO 9223 groups corrosion rates into a series of corrosivity categories that reflect the severity of the exposure environment, as shown in Table 2.

Corrosivity Category	First year corrosion rate (mm/year)
C1	<1.3
C2	1.3-25
C3	25-50
C4	50-80
C5	80-200

Table 2 – Corrosivity categories according to BS EN ISO 9223

Using the location and environmental data collected for each site, the corrosion rate for UK sites was estimated for distances of 300, 1000, 2500, 5000, 10000 and 15000m inland, with the corresponding corrosivity category. This is plotted in Figure 4, conservatively adding 50% uncertainty in the calculation.

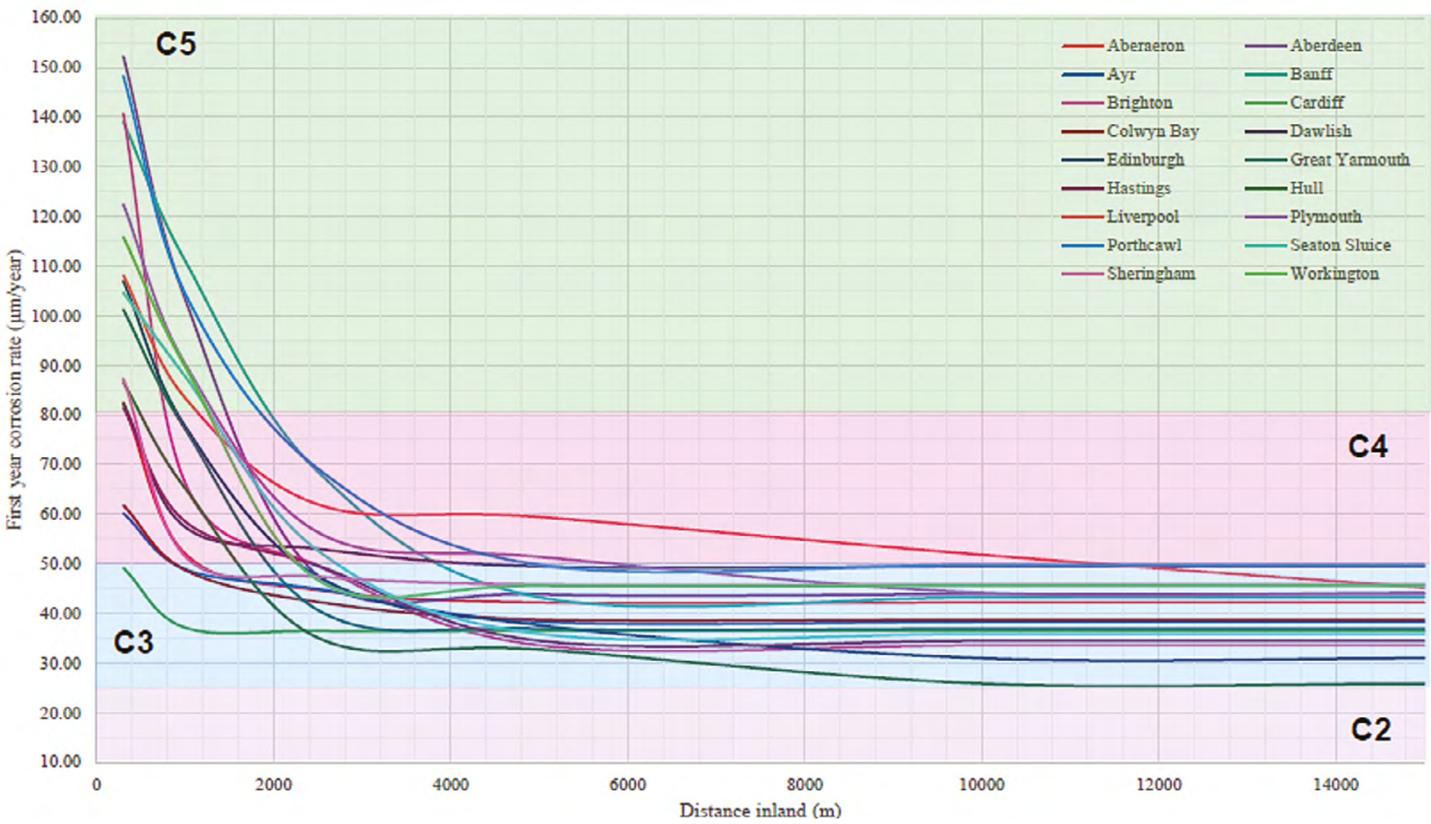

Figure 4 – Calculated first year corrosion rate (+50% uncertainty) with increasing distance from the coast for UK sites [4]

As shown in Figure 4, C5 corrosivity categories only occur at sites of very short distances from the coast (< 2 km, and in the majority of cases < 0.5 km to 1 km) and C4/C3 at > 2 km conservatively assuming 50% uncertainty in the calculations.

Conclusions

1) Review of international guidance on the use of weathering steel shows the UK approach to requirements for testing for both salinity and corrosivity is conservative.
2) All data indicates that airborne salinity, measured as dry deposition to BS EN ISO 9225, decays rapidly with distance from coast. At a distance of 2.5 km inland from the coast the deposition rate is less than half the S3 value. At distances greater than 2.5 km inland from the coast the chloride deposition continues to decay. These conclusions are supported by site specific data measured in accordance with BS EN ISO 9225 at various distances inland from the coast.
3) Estimation of corrosivity categories with distance from the coast, using the equation for carbon steel given in BS EN ISO 9223, show the highest corrosivity class (C5) only occurs very close to the coast (< 2 km) and that within a short distance, typically 1 km, the corrosivity category is generally C3 or in some cases C4, where the +50% uncertainty is included in the estimation of loss.
4) The data provides evidence that full corrosivity testing using coupons and salinity testing should be a mandatory requirement only if the proposed structure is less than 2.5 km from the coastline.

In addition to the findings and conclusions of the study reported above, the Steel Bridge Group has been gathering data from its members and from bridge owners on the performance of weathering steel bridges that have previously been constructed within 15 km of the UK coastline. This data is being added onto an online map[14]. This data suggests that existing weathering steel bridges are performing well, and where problems have been encountered they were not significant and they have been the direct result of poor detailing and specific faults such as leaking deck joints, rather than any general inadequacy in corrosion performance. ■

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Fire protection of steelwork

In this first of two technical articles, David Brown of the Steel Construction Institute gives some general background on fire protection and demonstrates how the guidance used by designers to specify fire protection for beams has been developed. Part 2 will consider the protection of columns.

Critical temperature

According to BS EN 1991-2[1], the temperature of a so-called standard fire rises rapidly and continues to increase with time. Unprotected steel begins to lose strength above 400°C and at 1006°C (the standard fire temperature at 90 minutes) has only 4% of its original strength. Apart from some specific cases, steel will generally need protection to limit the reduction in strength. This article assumes the protection is an intumescent coating.

Protection is specified to limit the steel temperature to a maximum value, known as the critical temperature. A higher critical temperature will mean less protection is required; a lower critical temperature will mean more protection is required. The critical temperature therefore has an important influence on cost and time, since more protection often means more coats and longer time to cure between coats.

The critical temperature can be calculated, but many designers appear to use the tabulated values published by ASFP[2]. Others may use the tabulated values provided in the UK NA to BS EN 1993-1-2[3]. Others appear to leave the specification entirely to the coating manufacturer. For the critical temperatures tabulated by ASFP, manufacturers provide tables of required protection thickness, for different periods of fire protection and for different values of A_m/V (equivalent to H_p/A). For temperatures not given in the ASFP tables, the manufacturer must be consulted.

The values of critical temperature published by ASFP and in the UK NA differ and are presented in different formats. The background to the tabulated values is opaque. The aim of this article is to explain how the values in both documents were calculated, demonstrate that the values are generally (but not always) conservative and encourage designers to take proper responsibility for this important aspect of design.

Both ASFP and the UK NA provide values of critical temperatures for beams and columns. Beams are (or should be) more straightforward since in both documents they are assumed to be restrained. This important limitation in scope is however not mentioned.

Utilisation

Utilisation is a measure of how hard the beam is working (strength, not deflection), which might be referring to the situation at ambient temperatures, or at elevated temperatures – it is essential to know!

At elevated temperatures, three factors influence the degree of utilisation. Firstly, the design value of actions are reduced, secondly a non-uniform temperature through the section can be of benefit and finally the member may not have been fully utilised at ambient temperatures – it has spare resistance which can be used in the fire design situation.

Reduced effects of actions in the fire limit state.

In the fire condition, the design values of forces and moments are reduced by applying a factor, η_{fi}. The reduction factor represents the characteristic permanent actions and a reduced value of the characteristic variable actions – effectively implying that not all the variable action will be applied in a fire, which seems sensible.

If the original design combination had been calculated using expression 6.10 of BS EN 1990, the factor η_{fi} is given by:

$\eta_{fi} = \frac{G_k + \Psi_{fi} Q_k}{\gamma_G G_k + \gamma_Q Q_k}$, which is expression 2.5 in BS EN 1993-1-2.

There are similar expressions (2.5a and 2.5b) if the load combinations had originally been determined using expressions 6.10a and 6.10b of BS EN 1990.

The UK NA to BS EN 1991-2 specifies that $\Psi_{fi} = \Psi_1$, which is to be taken from the UK NA to BS EN 1990. Typical values of Ψ_1 for different categories of loading are:

- For offices, $\Psi_1 = 0.5$
- For shopping areas, $\Psi_1 = 0.7$
- For storage, $\Psi_1 = 0.9$

Looking at the expression for η_{fi}, it is clear that the computed answer depends on the ratio Q_k:G_k and also on the value of Ψ_1. Designers might then observe that:

1. The ASFP document provides different limiting temperatures for offices, shopping and storage categories – but does not define Q_k:G_k
2. The UK NA to BS EN 1993-1-2 offers no categorisation of loading and no definition of Q_k:G_k

BS EN 1993-1-2 offers a helpful figure showing how η_{fi} varies with the ratio Q_k:G_k and the value of Ψ_1. For the three categories of loading and values of Ψ_1 = 0.5, 0.7 and 0.9, this relationship is shown in Figure 1.

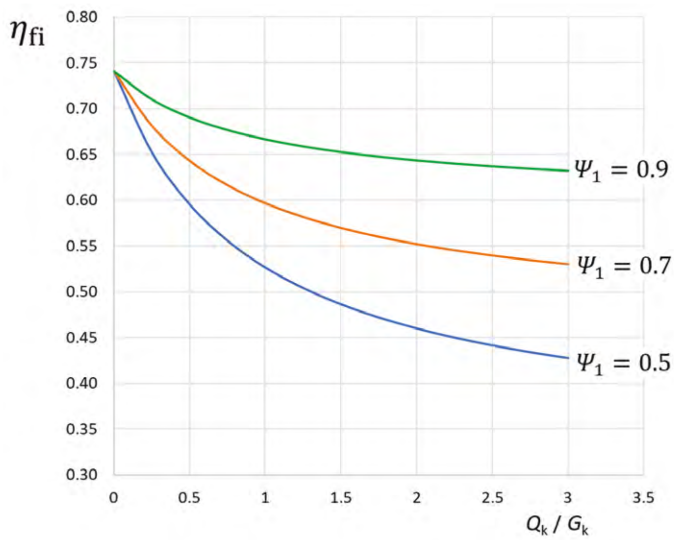

Table 1 – Reduction factor η_{fi}

NOTE 2 to Figure 2.1 in BS EN 1993-1-2 allows the use of η_{fi} = 0.65 except for storage. This conservative value should not be used as it will result in unnecessary protection being specified. For a typical Q_k:G_k ratio of 1:1 and Ψ_1 (office), the value of η_{fi} is 0.53.

Non-uniform temperature through the cross section.

If a beam supports a slab, the top flange is protected to some degree. BS EN 1993-1-2 allows for this by introducing an adaptation factor, κ_1. The values are:

- For a beam exposed on four sides (i.e. no slb), κ_1=1.0
- For an unprotected beam exposed on three sides and a slab on side four, κ_1 = 0.7
- For a protected beam exposed on three sides and a slab on side four, κ_1 = 0.85

An additional factor κ_2, will generally be 1.0.

In the fire condition, the moment resistance of a beam is the moment resistance at ambient temperature, divided by $\kappa_1\kappa_2$. When κ_1< 1, this produces an enhanced value of the moment resistance.

In Table NA.1 of the UK NA to BS EN 1993-1-2, the reason for three descriptions of beams should now be clear – the three categories reflect the three values of κ_1 above.

Confusingly, Table 16 of the ASFP guide has "non-composite beams carrying concrete floor slabs" and "composite beams supporting floor slabs", which both have a concrete slab. The difference between non-composite and composite in the ASFP table is discussed later.

Utilisation at ambient temperatures

Clearly, if a member has a surplus of resistance at ambient temperatures, those reserves of strength will be useful at elevated temperatures.

Calculation of the critical temperature

Reading BS EN 1993-1-2, designers might be tempted to use expression 4.22 to calculate the critical temperature (as it falls under the clause 4.2.4 "Critical temperature"). Once the utilisation μ_0 has been determined, the critical temperature $\theta_{a,cr}$ is given by:

$$\theta_{a,cr} = 39.19 \ln\left[\frac{1}{0.9674\mu_0^{3.833}} - 1\right] + 482$$

As an alternative, both the ASFP values and those in the UK NA are based on the necessary steel strength to carry the reduced design actions in the fire condition, which will of course be less than the nominal yield strength. Having determined the reduced strength required to carry the design loads, Table 3.1 of BS EN 1993-1-2 which shows reduced steel strength vs. temperature can be interrogated to determine at what elevated temperature the calculated reduction in steel strength occurs. This temperature is presented in the ASFP guide and in the UK NA as the critical temperature.

A comparison of the two alternatives is shown in Figure 2. As can be seen, the relationship between strength reduction and temperature is almost identical. If trying to reproduce the precise values in the ASFP document or the UK NA, it is important to note that the second process involving Table 3.1 is used.

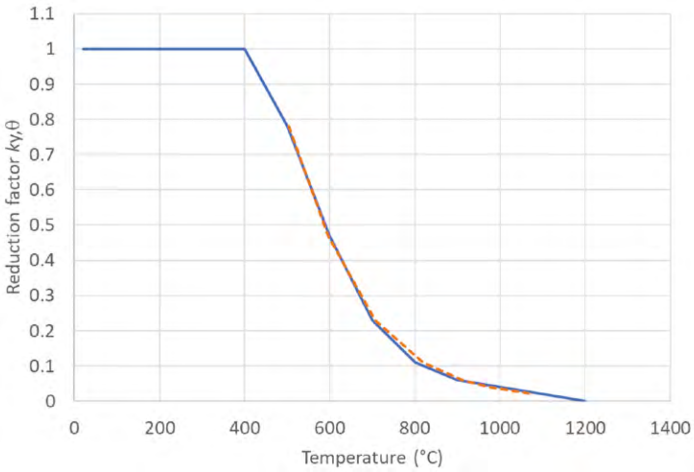

Figure 2: Reduction in steel yield strength

Beams in the UK NA to BS EN 1993-1-1

The relevant part of Table NA.1 is reproduced below. To help understand the tabulated temperatures, the value of κ_1 has been added to the relevant row.

Description of member		Critical temperature (°C) for utilisation factor μ_0					
	κ_1	0.7	0.6	0.5	0.4	0.3	0.2
Protected beams with slabs	0.85	558	587	619	654	690	750
Unprotected beams with slabs	0.7	594	621	650	670	717	775
Beams with no slab	1.0	526	558	590	629	671	725

Example 1: Protected beam with slab, $\mu_0 = 0.6$

In this example, κ_1 is 0.85 and enhances the moment resistance in the fire condition, which is equivalent to reducing the utilisation. The effective utilisation is therefore 0.6 × 0.85 = 0.51.

From Figure 2 it can be seen that the steel reaches 51% of its original strength at a temperature just below 600°C. The precise figure, obtained by linear interpolation from the values in Table 3.1, is 587°C, as tabulated above.

Example 1: Protected beam with slab

In this example, the value of μ_0 = 0.53, as calculated above for an office with Q_k:G_k = 1:1.

If it is assumed that for some reason, the beam is not fully utilised at ambient temperature, but is only utilised 90%, the effective utilisation becomes 0.53 × 0.85 × 0.9 = 0.41.

In this case, the critical temperature is 651°C, so the requirement for protection is reduced compared to example 1.

ASFP critical temperatures

The background to the ASFP critical temperatures is quite different to the approach in the UK NA. The UK NA requires the designer to calculate the utilisation, μ_0. In contrast, for different loading categories, ASFP have already calculated what is considered to be an appropriate value of μ_0 (although the value is not tabulated). The ASFP approach assumes that the beam is fully utilised at ambient temperatures – there is no opportunity to allow for any under-utilisation.

The relevant part of Table 16 from the ASFP Yellow Book is shown below (critical temperature in °C).

Building type	Non-composite beams carrying concrete floor slabs	Composite beams supporting concrete floor slabs
Office / Domestic	603	576
Storage	576	544
Shopping / Congregational / Car Park	583	553

ASFP utilisations

In the "office" loading category, ASFP assume Q_k:G_k = 1:1 and use expression 2.5b for η_{fi} (which includes ξ = 0.925).

The utilisation is therefore $\dfrac{1 + 0.5 \times 1}{0.925 \times 1.35 \times 1 + 1.5 \times 1} = 0.546$

If the beam is protected and has a slab, then κ_1 = 0.85 and the effective utilisation becomes 0.546 × 0.85 = 0.464.

From Figure 2, the critical temperature can be seen to be approximately 600°C. The precise value is 603°C, as tabulated above, under the heading "Non-composite beams carrying concrete floor slabs".

For composite beams, ASFP adopt the guidance in clause 4.3.4.2.3 of BS EN 1994-1-2, which indicates that the temperature in the steel section is assumed to be uniform, meaning that κ_1=1.0.

If κ_1 is set to 1.0, the tabulated value of 576°C is calculated.

The UK NA sees no need to discriminate between composite and non-composite beams. It does seem rather odd that in the ASFP guidance a beam designed compositely is considered a more onerous condition than a non-composite design when both are supporting a slab.

Designers should note the assumed value of Q_k:G_k = 1:1 for the "office category". If the ratio was, say 0.8:1, the critical temperature reduces from 603°C to 595°C. If the ratio changes in the opposite direction ($Q_k > G_k$), the value of 603°C is conservative. The ratio assumed for shopping areas is also Q_k:G_k = 1:1.

For storage, the assumed ratio is Q_k:G_k = 1:2 and the calculation for η_{fi} uses expression 2.5a from BS EN 1993-1-2, since this is more onerous than the result from expression 2.5b.

The utilisation is therefore $\dfrac{1 + 0.9 \times 2}{1.35 \times 1 + 1.5 + 1.0 \times 2} = 0.644$

(the utilisation according to expression 2.5b is 0.659)

If κ_1 = 0.85, the tabulated value of 575°C is calculated and if κ_1 = 1.0, the tabulated value of 544°C.

Conclusions from Part 1

Consider a composite beam in a multi-storey office building (a very typical example).

The ASFP guidance leads to a critical temperature of 576°C. This approach has the benefit of simplicity. As demonstrated above, the (unstated) utilisation is 0.546. The UK NA invites the designer to determine the utilisation. If the same utilisation is used, interpolation in Table NA.1 leads to a less onerous critical temperature of 602°C. The difference is because ASFP assume a uniform temperature through the cross section (κ_1 = 1.0) and the UK NA takes the benefit of a protected top flange (κ_1 = 0.85).

If the ratios Q_k:G_k assumed by ASFP reduce, the critical temperatures are not conservative.

Neither the ASFP nor UK NA values are appropriate for unrestrained beams.

Best practice is to calculate the actual utilisation – including any overdesign at ambient temperatures – and the critical temperature, which is not at all difficult. Alternatively, sufficient information must be provided so that the critical temperature can correctly determined by others. This must include the Q_k:G_k ratio, the loading category and the utilisation at ambient temperature. ■

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Critical temperatures for fire design: Part 2 – Columns

Part 1 of this article discussed the calculation of critical temperatures for beams, presented by ASFP and in the UK NA to BS EN 1993-1-2. In this second part, David Brown considers the information provided for columns.

Calculation process

The calculation of critical temperatures for columns is more involved than the process for beams, but is not so complicated that it should be avoided. According to BS EN 1993-1-2, clause 4.2.3.2, the resistance of a column at elevated temperature is given by:

$$N_{b,fi,t,Rd} = \frac{\chi_{fi} A k_{y,\theta} f_y}{\gamma_{M,fi}}$$

χ_{fi} is a reduction factor, which at least looks familiar to anyone who has designed a column.

$k_{y,\theta}$ is the reduction factor for yield strength, taken from Table 3.1 of BS EN 1993-1-2.

t relates to time – the temperature increases with time, so the value of $k_{y,\theta}$ reduces and therefore also the buckling resistance.

The expressions for χ_{fi} are very similar to those used at ambient temperature and presented in the same format:

$$\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^2 - \bar{\lambda}_\theta^2}}$$

and

$$\varphi_\theta = ½\,[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}_\theta^2] \text{ with } \alpha = 0.65\sqrt{235/f_y}$$

The final modification is that the non-dimensional slenderness is adjusted to reflect the fire condition:

$$\lambda_\theta = \lambda[k_{y,\theta}/k_{E,\theta}]^{0.5}$$

$k_{E,\theta}$ is an adjustment to the modulus of elasticity (Young's modulus), which changes with temperature and like $k_{y,\theta}$, is taken from Table 3.1 of BS EN 1993-1-2.

Buckling length

The most drama is associated with the buckling lengths to be assumed in the fire condition. BS EN 1993-1-2 specifies that for braced frames (the bracing could be a core, shear walls or bracing), the buckling lengths are to be taken as:

- 0.7L for the top storey
- 0.5L for all intermediate storeys

Identical guidance is given in BS EN 1994-1-2, but in that standard the buckling lengths are made a Nationally Determined Parameter. The UK NA to BS EN 1994-1-2 is more cautious than the code and specifies 0.85L for the top storey and 0.7L for all intermediate storeys. There is no opportunity for national choice in BS EN 1993-1-2 with respect to the buckling lengths, leading to an "interesting" difference in assumed behaviour between bare steel and composite columns – composite columns have a longer buckling length than their bare steel cousins.

Tabulated critical temperatures

Just like for beams, both ASFP and the UK NA to BS EN 1993-1-2 present critical temperatures for columns. The presentation is markedly different – the UK NA has a matrix of non-dimensional slenderness and utilisation, whilst the ASFP has values for UC sections and hollow sections in different building types. The ASFP table has no reference to slenderness or utilisation; the UK NA makes no distinction between section types. The following sections demonstrate how the tabulated temperatures have been determined.

ASFP critical temperatures

The relevant part of the ASFP table for Eurocode design is reproduced below (temperatures in °C).

Building type	Hot rolled H section columns in compression	Hot finished/formed structural hollow sections
Office/domestic	536	547
Storage	530	512
Shopping / congregational / car park	539	521

The ASFP temperatures are stated to be based on:

- 60% utilisation in fire (but this is not true);
- S275 steel;
- A "mid-range" UC section;
- Storey height of 3.5m;
- A top storey column.

The actual utilisations adopted by ASFP, together with the Q_k:G_k ratios are:

- For office loading, Q_k:G_k = 1:1 and $\eta_{fi} = 0.546$
- For storage loading, Q_k:G_k = 2:1 and $\eta_{fi} = 0.644$
- For shopping loading, Q_k:G_k = 1:1 and $\eta_{fi} = 0.618$

The analysis that led to the ASFP critical temperatures considered section sizes between 203 UC 46 and 305 UC 283, in S275 and S355, with some averaging of intermediate values. It was found that the lower steel grade was the more critical, which is the basis for the ASFP values.

The ASFP methodology always uses the more conservative reduced buckling length of 0.7L.

Due to the averaging of intermediate values, the quoted temperatures will not be correct for any particular situation, but should be conservative. The following example shows the calculation process.

254 UC 73, in S275, 3.5m long, in an office environment

At ambient temperature, $N_{b,Rd,z}$ = 1977 kN (quoted to three significant figures as 1980 kN in the Blue Book). The non-dimension slenderness is 0.622.

Assuming the column was fully utilised at ambient temperature, the reduction in design effects is due only to η_{fi}, given above as 0.546.

The critical temperature of 563°C will be satisfactory if the reduced resistance at this temperature is equal to or more than 0.546 of the "cold" resistance.

Interpolating Table 3.1 of BS EN 1993-1-2, for θ = 563°C, then:

$k_{y,\theta}$ = 0.585 and $k_{E,\theta}$=0.417

The modified slenderness, including the reduced buckling length of 0.7L, is given by:

$$\lambda_\theta = \lambda[k_{y,\theta}/k_{E,\theta}]^{0.5} = 0.7 \times 0.622 \times [0.585/0.417]^{0.5} = 0.516$$

$$\alpha = 0.65\sqrt{235/f_y} = 0.65 \times \sqrt{235/275} = 0.6$$

$$\varphi_\theta = ½\,[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}_\theta^2] = ½\,[1 + 0.6 \times 0.516 + 0.516^2] = 0.788$$

$$\chi_{fi} = \frac{1}{\varphi_0 + \sqrt{\varphi_0^2 - \bar{\lambda}_0^2}} = \frac{1}{0.788 + \sqrt{0.788^2 - 0.516^2}} = 0.723$$

$$N_{b,fi,t,Rd} = \frac{\chi_{fi} A k_{y,\theta} f_y}{\gamma_{M,fi}} = \frac{0.723 \times 9310 \times 0.585 \times 275}{1.0 \times 10^3} = 1083 \text{ kN}$$

1083/1977 = 0.548, so at 563°C the column has slightly more resistance that required – the critical temperature is satisfactory.

In S355, the ratio is 0.576, showing that ASFP values are conservative for S355 and S460. The ASFP temperatures are conservative for column lengths above 3.5m. If the column length is less than 3.5m, the values are not conservative, but only by a trivial amount.

As the ASFP temperatures are generally conservative, higher temperatures will be calculated if the *actual* design situation is assessed. The following are examples, all using the same 254 UC 73:

245 UC 73, S355, 4.5m long, Office loading, 100% utilised at ambient: $\theta_{a,cr}$ = 581°C

245 UC 73, S460, 4.5m long, Office loading, 100% utilised at ambient: $\theta_{a,cr}$ = 592°C

245 UC 73, S355, 4.5m long, Office loading, 80% utilised at ambient: $\theta_{a,cr}$ = 616°C

245 UC 73, S355, 4.5m long, Office loading, 60% utilised at ambient: $\theta_{a,cr}$ = 659°C

ASFP provide different temperatures for hollow sections. This is because at ambient temperatures the imperfection factor for UC sections was taken as 0.49 in all cases. For hollow sections the value was taken as 0.21.

180 × 180 × 8 SHS, in S355, 3.5m long, in an office environment

At ambient temperature, $N_{b,Rd,z}$ = 1676 kN (quoted to three significant figures as 1680 kN in the Blue Book). The non-dimension slenderness is 0.655

Assuming the column was fully utilised at ambient temperature, the reduction in design effects is due only to η_{fi}, given above as 0.546

The critical temperature of 547°C will be satisfactory if the reduced resistance at this temperature is equal to or more than 0.546 of the "cold" resistance.

Interpolating Table 3.1 of BS EN 1993-1-2, for θ = 547°C, then:

$k_{y,\theta}$ = 0.634 and $k_{E,\theta}$ = 0.464

The modified slenderness, including the reduced buckling length of 0.7L, is given by:

$$\lambda_\theta = \lambda[k_{y,\theta}/k_{E,\theta}]^{0.5} = 0.7 \times 0.655 \times [0.634/0.464]^{0.5} = 0.536$$

$$\alpha = 0.65\sqrt{235/f_y} = 0.65 \times \sqrt{235/355} = 0.529$$

$$\varphi_\theta = ½\,[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}_\theta^2] = ½\,[1 + 0.529 \times 0.561 + 0.536^2] = 0.786$$

$$\chi_{fi} = \frac{1}{\varphi_0 + \sqrt{\varphi_0^2 - \bar{\lambda}_0^2}} = \frac{1}{0.786 + \sqrt{0.786^2 - 0.536^2}} = 0.735$$

$$N_{b,fi,t,Rd} = \frac{\chi_{fi} A k_{y,\theta} f_y}{\gamma_{M,fi}} = \frac{0.735 \times 5440 \times 0.634 \times 355}{1.0 \times 10^3} = 900 \text{ kN}$$

900/1676 = 0.537 , so at 547°C the column has slightly lower resistance than required – the critical temperature is (just) unsatisfactory. The correct critical temperature is 544°C, which is not considered to be a significant difference.

UK NA Critical temperatures

The relevant part of the UK NA table is shown below.

In contrast to ASFP, the values of critical temperatures for columns in the UK NA are based on S355 steel and do not apply the reduction to the buckling length.

For a non-dimensional slenderness of 0.8, the reduction factor at ambient temperature can be calculated as 0.663.

Non-dimensional slenderness	Critical temperature (°C) for utilisation factor μ_0					
	0.7	0.6	0.5	0.4	0.3	0.2
λ = 0.4	485	526	562	598	646	694
λ = 0.6	470	548	554	590	637	686
λ = 0.8	451	510	546	583	627	678
λ = 1.0	434	505	541	577	619	672
λ = 1.2	422	502	538	573	614	668
λ = 1.4	415	500	536	572	611	666
λ = 1.6	411	500	535	571	620	665

For those interested, an alternative way to calculate the reduction factor without any reference to the section is to use the following expressions:

$$\chi = \frac{T_1 - T_2}{2\bar{\lambda}^2}$$

Where $T_1 = 2\phi$ and $T_2 = (T_1^2 - 4\bar{\lambda}^2)^{0.5}$

Using these expressions with λ = 0.8 and α = 0.49, then ϕ = 0.967

T_1 = 2 × 0.967 = 1.934

$T_2 = (1.934^2 - 4 \times 0.8^2)^{0.5}$ = 1.086

$$\chi = \frac{1.934 - 1.086}{2 \times 0.8^2} = 0.663 \text{ as above}$$

The buckling stress at ambient temperature is therefore 0.663 × 355 = 235 N/mm²

If the utilisation in the fire condition was 0.6, the buckling stress in the fire condition would be 0.6 × 235 = 141 N/mm²

The objective then is to determine at what temperature the buckling stress is 141 N/mm² – the UK NA states this to be 510°C. Following the same process as demonstrated for the ASFP values (but omitting the 0.7L reduction in buckling length), the steps are shown below.

Interpolating Table 3.1 of BS EN 1993-1-2, for θ = 510°C, then:

$k_{y,\theta}$ = 0.749 and $k_{E,\theta}$ = 0.571

The modified slenderness is given by:

$$\lambda_\theta = \lambda[k_{y,\theta}/k_{E,\theta}]^{0.5} = 0.8 \times [0.749/0.571]^{0.5} = 0.916$$

$$\alpha = 0.65\sqrt{235/f_y} = 0.65 \times \sqrt{235/355} = 0.529$$

$$\varphi_\theta = ½\,[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}_\theta^2] = ½\,[1 + 0.529 \times 0.916 + 0.916^2] = 1.162$$

T_1 = 2 × 1.162 = 2.324

$T_2 = (2.324^2 - 4 \times 0.916^2)^{0.5}$ = 1.430

$$k_{y,\theta}\chi = 0.749 \times \frac{2.324 - 1.430}{2 \times 0.916^2} = 0.399$$

The buckling stress at the temperature of 510°C is therefore 0.399 × 355 = 142 N/mm²

The UK NA is not conservative for columns in S275 steel. The largest difference is at highly utilised sections and large slenderness (for example μ_0 = 0.7; $\bar{\lambda}$ = 1.6, where the difference is about 6%)

Comparison between ASFP and UK NA for columns

The calculation process is identical, although the results are presented in quite different formats. The UK NA does not apply the 0.7L reduction in buckling length, so for a given utilisation will be more conservative. If the ASFP approach is applied to a 254 UC 73 in S355, 3.5m long, fully utilised at ambient temperatures, in an office loading condition, the critical temperature is 572°C. The UK NA approach would show a more onerous critical temperature of 534°C, simply because of the longer buckling length.

The UK NA has the advantage that actual utilisations can be calculated, including the 0.5L or 0.7L buckling length reduction and allowing for surplus resistance in the ambient condition. ■

Design of steel beams with large web openings

BS EN 1993-1-13: Beams with large web openings **has been published by BSI. It deals with the design of steel beams with circular, hexagonal, rectangular, elongated circular and sinusoidal openings and is a sister-document to the design of composite beams with large web openings that is being worked on currently under EN 1994-1-1. Mark Lawson of the SCI, who was a member of the Project Team, explains some of the technical aspects of this new Part.**

To a UK audience, Eurocode 3 Part 1–13 *Beams with Large Web Openings* follows the SCI publication P-355 relatively closely and it includes the following information:

- Application for steel grades up to S460.
- Rules for different opening shapes.
- Beams with relative slender webs depths, $h_w \leq 121\, t_w\, \varepsilon$, where t_w is the web thickness.
- Limits on web opening sizes for both unstiffened and stiffened openings.
- Two methods for Vierendeel bending checks at circular openings, which are a simplified equivalent rectangle method and a radial stress method.
- Web-post buckling rules, now using buckling curve 'a' to BS EN 1993-1-1 and extended to include hexagonal openings.
- Rules for the buckling resistance of the compressed top Tee at long openings.
- Rules for end-post buckling based on an adaptation of the web-post buckling rules.
- Rules for asymmetric steel sections taking account of an in-plane web-post moment required for re-distribution of shear forces between the Tees.
- Simplified rules for the additional deflection due to large web-openings.
- Lateral torsional buckling verifications based on the section properties at the centre-line of the openings.

This article covers the principles of design of large web openings in steel beams and a second article will summarise the rules for end-posts based on recent tests at City, University of London.

The limits on maximum opening sizes for unstiffened openings are presented in Table 1, below. These are Nationally Determined Parameters so could be modified for use in the UK. In this table, the effective opening length of elongated circular openings is taken as $a_{eff} = a_o - 0.3h_o$, where a_o is the opening length. The corresponding limiting dimensions for longitudinally stiffened openings are given in a further table.

Verifications at large web openings

The verifications that should be made at large web openings in steel beams are:

- Pure shear check based on the reduced depth of the web.
- Bending resistance at the centre-line of the opening.
- Vierendeel bending of the web-flange Tees due to transfer of shear across the opening.
- Web buckling next to isolated openings.
- Web-post shear and buckling between closely spaced openings.
- End-post shear and buckling next to the connections.
- Combined compression and bending of slender top Tees in regions of high moment.
- Calculations of the relative deflection across large web openings, where this impairs the serviceability performance.

The resistance to Vierendeel bending of the Tees at large rectangular openings can be increased by welding horizontal stiffeners on one or both sides of the beam that project at least 150mm past the ends of the opening to act as an 'anchorage length'.

For Vierendeel bending at circular, elongated circular and hexagonal openings, the equivalent rectangular opening width, a_{eq}, defines the double curvature moment that is developed in the web-flange Tees. The critical angle for Vierendeel bending around a circular opening is at approximately 26° to the vertical and so the equivalent rectangular opening width for this verification is given as $a_{eq} = 0.45\, h_o$, where h_o is the opening diameter.

Web-post buckling between openings

For closely spaced openings, web-post buckling may occur due to the transfer of horizontal shear which leads to 'strut and tie' action in the web-post. The method for web-post buckling uses an effective length of the equivalent strut, which is illustrated in Figure 1 for adjacent circular openings. The buckling strength is obtained using buckling curve 'a' to EN 1993-1-1, which is justified by correlation with tests and by the additional restraints to plate buckling in comparison to an equivalent strut.

For this verification, the compressive force acting on the web-post, $N_{wp,Ed}$ should be taken equal to the horizontal shear force in the web-post and it is required that the buckling resistance of the web-post exceeds this force.

The non-dimensional slenderness of the web-post is defined as follows for the different opening shapes, where so is the edge to edge spacing of the openings.

Table 1 : Limiting dimensions for different shapes of unstiffened openings

Shape of opening	Maximum opening height, h_o	Maximum opening length, a_o	Minimum edge to edge spacing, s_o	Minimum depth of Tee: Tee in compression	Minimum depth of Tee: Tee in tension
Circular	0.8 h	-	0.1 h_o	Max($t_f + r$ + 10mm; t_f + 30mm)	Max($t_f + r$ + 10mm; t_f +30mm)
Hexagonal	0.75 h	1.5 h_o	0.25 h_o		
Rectangular	0.75 h	2.5 h_o	Max(0.5a_o ;h_o)	Max(a_o / 12; 0.1h)	0.1h
Elongated	0.8 h	3 h_o	Max(0.25a_{eff};0.5h_o)	a_{eff} / 12	Max($t_f + r$ + 10mm; t_f +30mm)
Sinusoidal	0.8 h	5 h_o	0.25 h_o	a_{eff} / 12	

a_{eff} = effective opening length; h = beam depth; t_f = flange thickness; r = root radius

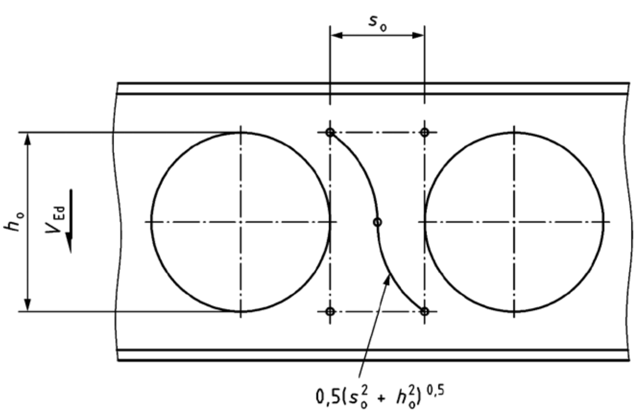

Figure 1 - Illustration of the effective length due to web-post buckling between circular openings

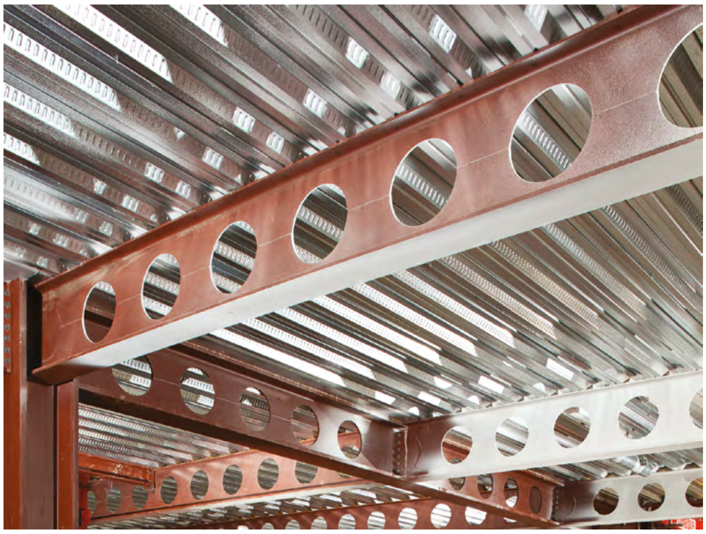

Figure 2 - Typical slender beam with infills (courtesy of Kloeckner UK Metals Westok)

For circular openings and elongated circular openings:

$$\bar{\lambda}_{wp} = \frac{1.75\sqrt{s_o^2 + h_o^2}}{t_w}\frac{1}{\lambda_1} \text{ but } \bar{\lambda}_{wp} \leq \frac{2.4\, h_o^2}{t_w}\frac{1}{\lambda_1}$$

For cellular beams with unequal web thickness in the two parts, t_w may be taken as the average web thickness in this formula.

For rectangular openings:

$$\bar{\lambda}_{wp} = \frac{2.5\sqrt{s_o^2 + h_o^2}}{t_w}\frac{1}{\lambda_1} \text{ but } \bar{\lambda}_{wp} \leq \frac{3.5\, h_o^2}{t_w}\frac{1}{\lambda_1}$$

The web-post buckling resistance should be taken as:

$N_{wp,Rd} = \chi_{wp}\, s_o \min\{t_{w,tT} f_{y,tT}\,;\, t_{w,bT} f_{y,bT}\} / \gamma_{M1}$

Where χ_{wp} is determined to buckling curve 'a' using λ_{wp}

$t_{w,tT} f_{y,tT}$ and $t_{w,bT} f_{y,bT}$ are the multiples of the top or bottom web thickness and the steel strengths for these parts.

γ_{M1} is taken as 1.0.

Buckling Resistance of a Compressed Top Tee at a Large Rectangular Opening

A new method is presented for the stability of slender Tees at long web openings. The definition of a 'long opening' is given in clause 8.3.2 (1). The combination of compression, Vierendeel bending and bending from local applied loads acting on a slender is determined as follows:

$$\frac{N_{T,Ed}}{N_{b,Rd}} + \frac{0.4M_{T,Ed} + M_{add,Ed}}{M_{T,Rd}} \leq 1.0$$

where

$$M_{T,Ed} = \frac{V_{Ed}a_{eff}}{4} + N_{T,Ed}w_{vier,add}$$

where

$N_{T,Ed}$ is the compression force in the top Tee resulting from global bending

$N_{b,Rd}$ is the buckling resistance of the top Tee

$M_{T,Ed}$ is the moment in the Tees due to Vierendeel bending combined with an eccentricity due to the relative deflection across the opening

$M_{T,Rd}$ is the bending resistance of the Tee

$M_{add,Ed}$ is the moment due to the loading applied over the opening

$w_{vier,add}$ is the relative deflection across the opening in Vierendeel bending, which is calculated at serviceability. In the limit, $w_{vier,add} \leq a_o/200$.

Note: It is assumed that under factored loads, the relative deflection across the opening is $2w_{vier,add}$ and so the additional eccentric moment acting on each Tee due to the relative deflection across the opening is, $N_{T,Ed}\, 2w_{vier,add} / 2 = N_{T,Ed}w_{vier,add}$.

Example for a Slender Tee

As an example of the use of this check for combined actions on the compressed Tee, consider a large rectangular opening in a 13m long beam at x = 5m from one support for the following data:

a_o = 1000mm (a_o/h_o = 2.22 < 2.5)

h_o = 450mm and h = 650mm (h_o/h = 0.69 < 0.75)

A_f = 220mm × 20mm and t_w =10mm

Q_{Ed} = 30 kN/m; M_{Ed} = 619 kNm and V_{Ed} =45 kN and at x = 5m

Cross-sectional area of Tee, A_T= 220 × 20 + 80 × 10 = 5200mm²

Depth of elastic neutral axis from top of section, z_e = 18mm

Axial resistance of the Tee (for f_y=345 N/mm²) = 5200 × 345 × 10^{-6} = 1794 kN

Bending resistance at the opening, $M_{o,Rd}$ = 1794 × (650 − 2 × 18) × 10^{-3} = 1102 kNm > 619 kNm

Compression force in top Tee, $M_{T,Ed} = \dfrac{619 \times 10^3}{650 - 2 \times 18} = 1008$ kN

Inertia of Tee in vertical direction, I_T = 2265 × 10³mm⁴

Radius of gyration of Tee, $i_{zz} = \left(\dfrac{2265 \times 10^3}{5200}\right)^{0.5} = 20.9$mm

Slenderness of Tee, $\lambda_T = \dfrac{0.5 \times 1000}{20.9} = 24$

Non-dimensional slenderness, $\bar{\lambda}_t$ = 24/77 = 0.31

For buckling curve (c) considering the Tee as a strut, its buckling resistance is obtained as χ_T= 0.93.

Buckling resistance of Tee, $N_{b,Rd}$ = 0.93 × 1794 × 10^{-3} = 1668 kN > 1008 kN

Vierendeel bending moment acting on a Tee, $M_{T,Ed}$ = 45 x 1.0/4 = 11.3 kNm

Plastic bending resistance of Tee, $M_{T,Rd}$= 21.1 kNm

Additional end moment due to local load on Tee, $M_{add,Ed}$ = 30 × 1.0²/12 = 2.5 kNm

Eccentricity of the axial force due to the deflection across the opening for V_{ser} = 31.6 kN:

$$w_{vier,add} = \frac{31.6 \times 1.0^3 \times 10^9}{24 \times 210 \times 2265 \times 10^3} = 2.8\text{mm } (=a_0 / 355)$$

This satisfies the deflection limit of $a_o/200$ across the opening at serviceability.

The additional eccentric moment due to the relative deflection across the opening is,

$N_{T,Ed}\, w_{Vier,add}$ = 1008 × 2.8 × 10^{-3} = 2.8 kNm

Verification of the combined buckling and bending resistance of the top Tee:

$$\frac{1008}{1668} + \frac{0.4 \times (11.3 + 2.8) + 2.5}{21.1} = 0.60 + 0.39 = 0.99 < 1.0 \text{ just OK}$$

This shows that the 100mm deep top Tee is stable under combined loads for a 1m long opening.

Checks on End-Posts

The design of end-posts next to connections is an aspect not properly covered by SCI P355 and is now addressed in BS EN 1993-1-13. This will be covered in a subsequent article in *New Steel Construction*, based on tests on cellular beams at City University of London. The minimum width of an end-post in a cellular beam with circular openings is given as 0.25 h_o, and for rectangular openings, the minimum width increases to 0.5 a_o. ■

Design of End-Posts to BS EN 1993-1-13: Beams with Large Web Openings

In a first article last month, Mark Lawson of the Steel Construction Institute presented an outline of the new BS EN 1993-1-13. This second article presents results of tests on end-posts in cellular beams at City, University of London.

In the recently published BS EN 1993-1-13 Beams with large web openings, a new method is given for the design of end-posts, which is the part of the web next to an end connection. This was missing in previous guidance to SCI P355. Often, it is necessary to introduce a half or full infill plate next to the connection to satisfy the dimensional limits and to achieve the required design shear resistance.

Two generic connection types may be considered:

- Bolted shear connections to the beam web either by fin plates or angles.
- Welded end-plate connections in which the end-plate is either connected only to the beam web (partial depth end-plate) or also to the flanges (full depth end-plate).

For end-plate connections, the end-plate strengthens the end-post in horizontal shear and bending, and also partly stabilises the end-post against buckling. Conversely, bolted fin-plate or angle connections lead to a reduction in the shear and bending resistance at the line of the bolt holes and may provide less restraint to end-post buckling.

The design method for end-post buckling given in BS EN 1993-1-13 was compared to the results of tests on cellular beams at City, University of London reported by Tsavdaridis et al. (2024). The tests were on symmetric cellular beam sections with various end-post details and the two connection types noted above.

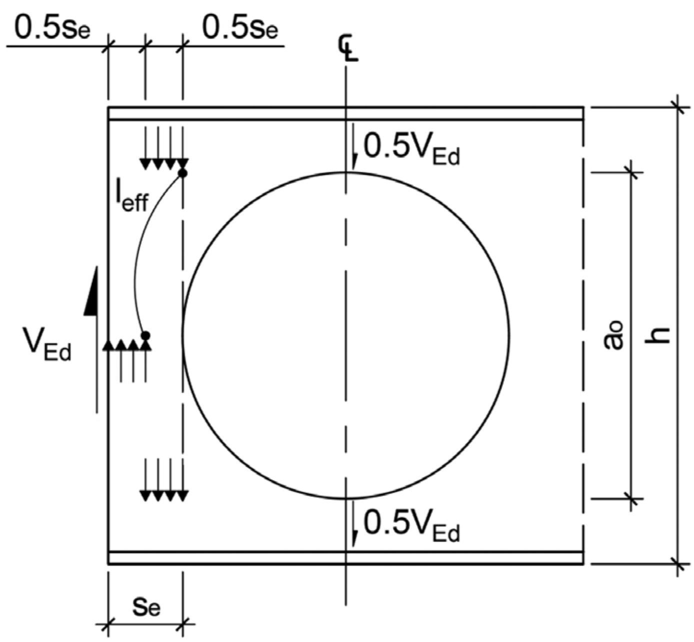

Figure 1 – Illustration of strut buckling model for an end-post in EN 1993-1-13

Buckling of the end-post to BS EN1993-1-13

The design method for buckling of end-posts in EN1993-1-13 is based on an adaptation of the web-post buckling model. This strut action in the end-post is shown in Figure 1. The compression force, $N_{ep,Ed}$, acting on the strut is taken as equal to the shear force in the top Tee, which is $N_{ep,Ed} = 0.5V_{Ed}$ for a symmetric section, and the effective width of the equivalent strut is taken as $b_{eff} = 0.5s_e$, where s_e is the end-post width.

The minimum width of the end-post is given as $s_e \le 0.25a_o$ in the case of an adjacent circular opening of diameter, a_o, and $s_e \le 0.5a_o$ for an adjacent rectangular opening of length, a_o.

For an end-post next to a circular opening, the effective length of the equivalent strut is taken as the diagonal distance over half of the end-post width and half of the opening depth. The end-post relative slenderness is:

$$\bar{\lambda}_{ep} = 1.75\frac{(s_e^2 + a_o^2)^{0.5}}{t_w\lambda_1} \le \frac{2.45a_o}{t_w\lambda_1}$$

where $\lambda_1 = 3.14\ (E/f_y)^{0.5}$

The buckling resistance of the end-post is obtained from buckling curve 'a' to EN1993-1-1.

Modifications to this equation are given an end-post partly stabilised by a full depth end-plate connection, and for an end-post with notches. No guidance is given in EN 1993-1-13 for the use of half or full infill plates to form part of the end-post, although in principle the same theory may be used by replacing t_w by t_i where t_i is the thickness of the infill plate if this is thinner.

The buckling resistance of the end-post should exceed the compression force transferred from shear in the top Tee, which for a symmetric section is given by:

$$N_{ep,b,Rd} = \chi_{ep}0.5s_et_wf_y \ge 0.5\ V_{Ed}$$

Where χ_{ep} is the reduction factor due to buckling of the end-post using the relative slenderness in the above equation.

Test details	Connection type	Failure shear in test	End-post buckling to BS EN 1993-1-13	Mode of failure
90mm wide end-post	Fin plate	325 kN	188 kN	End-post bending
	Full depth end-plate	331 kN (+2%)	197 kN	Vierendeel bending at opening
90mm wide end-post with 90x60mm notches	Fin plate	279 kN	178 kN	Buckling at notch
	Partial depth end-plate	298 kN (+7%)	193 kN	Lateral movement of flange at notch
Half infill plate (200mm wide) with 90x60mm notches	Fin plate	398 kN	263 kN	Buckling of half infill plate
	Partial depth end-plate	417 kN (+5%)	279 kN	

Table 1 - Test shear failure loads of end-posts and comparison with the design predictions using the measured steel strength

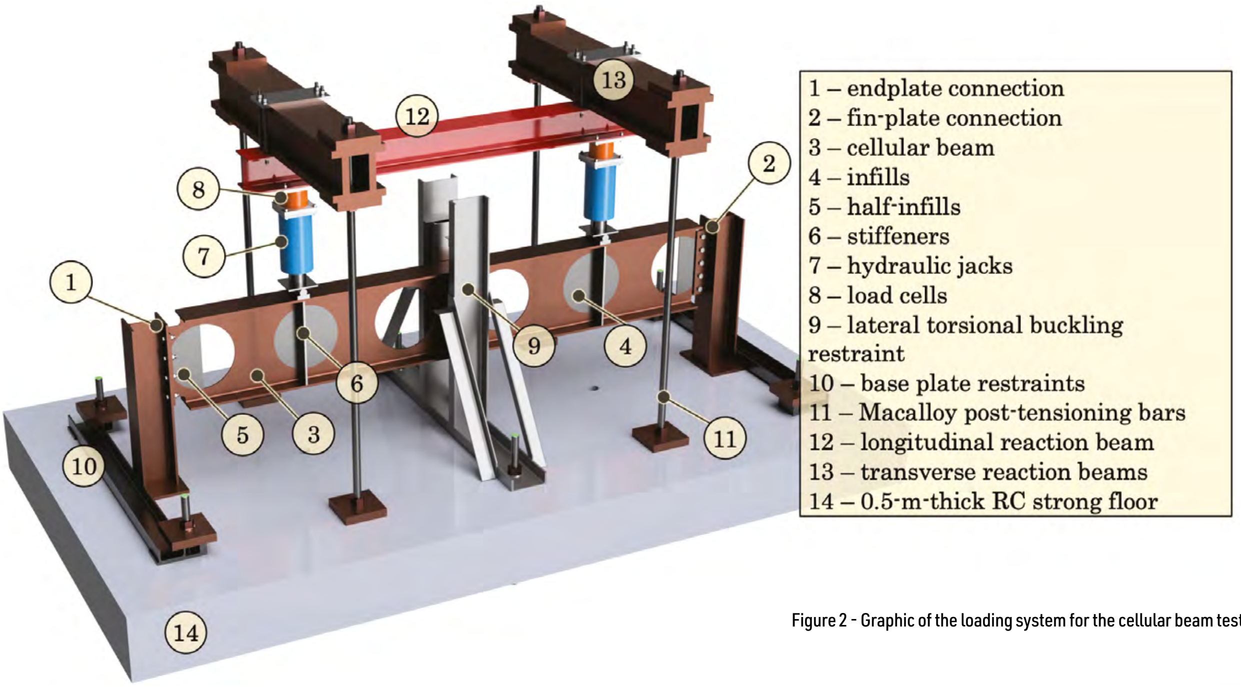

Figure 2 - Graphic of the loading system for the cellular beam tests (Tsavdaridis et al, 2024)

Example for a partial depth fin plate connection with s_e = 100mm and a_o = 400mm; t_w = 9.0mm; f_y = 355 N/mm²:

$\ell_{eff} = 0.5 \times (100^2+400^2)^{0.5} = 206$mm
$\lambda_{ep} = 3.46 \times 206/ 9.0 = 79$
$\lambda_1 = 3.14 \times (210 \times 10^3/355)^{0.5} = 76$
$\lambda_{ep} = 79/76 = 1.04$
$\phi = 0.5 \times (1 + 0.21 \times (1.04 - 0.2) + 1.04^2) = 1.13$
$\chi_{ep} = [1.13 + (1.13^2 - 1.04^2)^{0.5}]^{-1} = 0.63$
Buckling resistance, $N_{b,Rd} = 0.63 \times 0.5 s_e t_w f_y = 0.63 \times 50 \times 9.0 \times 355 \times 10^{-3} =$ 100.6 kN

For a symmetric section, it is required that $0.5V_{Ed} \leq N_{ep,b,Rd}$, and so the maximum end shear force that may act at the connection is $V_{Ed} \leq 201$ kN.

Comparison with tests on end-posts in cellular beams

A series of 3 cellular beams, each with two types of connections, was tested to compare with the design method for end-posts and these tests were reported by Tsavdaridis (2024). The test configuration is shown in Figure 2 and the details of the tests were:

- Cellular beams of h = 560mm depth using 406 × 178 × 67 kg/m UB sections.
- Opening diameter, a_o = 400mm (a_o = 0.71h).
- Beam span, L = 3.63m with jack loads applied at 0.82m from the supports.
- S355 nominal steel grade (measured as f_y = 393 N/mm²).
- Columns, 203 × 203 × 60 kg/m UC sections (1m high).

The two connection types were:

- End plate connections using a 12mm thick end plate with 2 × 4 no. M20 bolts to the column flange.
- Fin plate connection using a 12mm thick projecting welded plate of 440mm depth with 5 no M20 bolts to the beam web.

The three forms of end-post combined with the two connection types were:

- Narrow end-post of 90mm width ($s_e = 0.225a_o$).
- Narrow end-post with 90mm wide × 60mm deep notches to both flanges with a 20mm radius corner of the notch.
- End-post formed by 200mm wide half infill of 9mm measured thickness.

Full infills and web stiffeners were used at the loading positions.

The mode of failure of the narrow end-post next to the notched flange at a shear force of 298 kN is shown in Figure 3. The same beam with a half infill plate shown in Figure 4 failed at a shear force of 398 kN, in this case by buckling of the infill plate.

The test shear failure loads are presented in Table 1 in comparison to the prediction of the design to BS EN 1993-1-13 using measured material strengths. The ratio of the test failure shear to the design prediction was in the range of 1.54 to 1.73 for the four tests with narrow end posts.

This shows that the proposed method is conservative, probably because of redistribution of shear forces from the compressed top Tee to the bottom Tee in tension after buckling at the notch had occurred.

For the test with half infill plates, the ratio of the failure load to the design prediction was 1.49 and 1.51 and shows that the model for buckling of the infill plate is reasonably accurate.

It is concluded that the design method for end-posts to BS EN 1993-1-13 is relatively conservative when applied to symmetric cellular beams and some improvements could be made based on a parametric study of various end-plate geometries. Based on the test results, the minimum width of an end-post in cellular beams may be potentially reduced to $0.2a_o$ as a Nationally Determined Parameter. ■

Acknowledgements

The tests were performed in the Heavy Structures Laboratory at City, University of London and the test work was sponsored by ASD Westok Ltd who also fabricated the test beams, columns and their connections.

References

Tsavdaridis, K.D., McKinley, B., Corfar, D-A, Lawson, R.M. (2024) *Cellular Beam End-posts with Two Connection Types, End Notches and Infill Plates. Journal of Constructional Steel Research*, 215, article number 108547.

Figure 3 - Buckling at narrow end post for the notched cellular beam

Figure 4 - Buckling of the half infill plate due to the transfer of shear from the top Tee

Minimum degree of shear connection in composite beams, according to Eurocode 4 and other guidance

In this article Graham Couchman of SCI and former chairman of CEN/TC250/SC4 (the Eurocode 4 committee), considers why we have rules for minimum degree of shear connection, and what implications the upcoming changes to Eurocode 4 have for current UK practice. The rules are very much related to the resistance of shear connectors (hereafter simply referred to as studs), which are also changing – or maybe not?

Introduction

The shear connection between a steel beam and the slab it supports ensures that force can be transferred from the steel to the concrete. That is how a humble steel beam can become twice as strong and three times as stiff when connected structurally to the floor slab (the slab would have been present anyway, even if not structurally connected).

Each stud can transfer an indicative 50kN to 80kN, depending on the slab details (presence of deck etc), with its resistance readily determined using relevant guidance. Full shear connection is achieved when the number of studs is sufficient to transfer enough force to either fully utilise the concrete in compression, or the steel beam in tension. In buildings, the former normally governs. If fewer studs are used the degree of shear connection is the sum of their resistances divided by this maximum force, and the beam resistance is determined using this lower level of force. However, studs have a limited range of slip (movement) over which they can transfer their assumed resistance, from about 1mm to in excess of 6mm, again depending on the details of the slab (Figure 1). They also have a finite stiffness, with an upper bound of about 100kN per mm in a solid slab. This stiffness is not readily calculable, or indeed quantifiable in any other practical way by a designer. The purpose of a minimum degree of shear connection is to ensure that the collective stiffness of the studs on a given beam is enough to ensure that the greatest slip, which occurs at the beam ends (for a beam subject to UDL we have a symmetric situation with zero slip at mid-span), does not exceed the slip at which the stud resistance starts to drop off. Because the stiffness of the studs is not known by designers, the rules allow them to make this verification indirectly by considering the resistance of the studs, which is known (i.e. there is an assumed relationship between stud stiffness and strength).

Generation 2 EN 1994-1-1[1] (hereafter referred to as prEN 1994-1-1 to differentiate from the current EN 1994-1-1) makes this all clear in 8.6.3.3(1) of the 2023 draft by including wording that states that 'either the maximum calculated slip should not exceed the capacity….., or the degree of shear connection shall comply [with the appropriate minimum value]'. This is something SCI has been doing for years, using numerical modelling to predict the level of slip in beams that failed to comply with the minimum degree rules, often because somebody on site left out half the studs!

It is worth noting that for beams with transverse decking the minimum degree is often a critical check. It is not physically possible to place studs closer than in every trough (so at about 300mm centres). Adding more than two per trough is not permissible according to Eurocode 4 and indeed will add very little in typical cases where a concrete failure surface passing over the studs governs (Figure 1). So, if two studs per trough will not transfer enough force to satisfy the limit the design will not just have a lower resistance, it simply cannot be made to satisfy the code.

The evolution of codified limits and other guidance

BS 5950-3.1[2] included very simple rules for determining both stud resistance and minimum degree of shear connection. The rules only considered the influence of a limited number of variables. EN 1994-1-1[3] includes rules for minimum degree that is only assumed to vary as a function of the steel grade and span of the beam, plus any asymmetry of the steel cross-section, despite the fact that other variables may well be relevant, most obviously:

Figure 1: Deformation (slip) of a 19mm diameter shear stud in transverse decking at failure (courtesy University of Luxembourg)

- the slip capacity of the studs (tests have shown that in excess of 10mm can be achieved when studs are placed in decking with ribs running transversely with respect to the beam).
- whether the beam is propped or unpropped during construction.
- how much the beam is utilised under design loading (once the steel starts to plastify and loose stiffness the amount of slip, which is related to the curvature of the cross-section, increases disproportionately, so if a beam is not required to be fully utilised the number of studs needed to control slip drops significantly).

EN 1994-1-1 also imposes an absolute minimum of 40% connection, although the reasons why are not obvious or indeed logical (a limit on maximum spacing of studs ensures other effects, such as vertical separation of the slab and steel, will not occur, and effectively sets a minimum level of shear connection). The example below is taken from 6.6.1.2(1) to illustrate the current EN 1994-1-1 approach for symmetric beams:

- For an effective span L_e not exceeding 25m:

  $$\eta \geq 1 - \left(\frac{355}{f_y}\right)(0.75 - 0.03L_e)$$

  But $\eta \geq 0.4$

  Where f_y is the yield strength of the steel
- For spans in excess of 25m:

  $\eta \geq 1.0$

Given the often critical and 'show stopping' nature of the rules for minimum degree of connection, as well as their obvious simplicity in terms of variables considered, some time ago at SCI we undertook an extensive range of numerical modelling to see how many studs were needed to limit the slip for different beams. This led to the rules presented in SCI's publication P405[4]. For the first time all the relevant variables, including those noted above, were taken into account. The presentation is a series of equations of a form similar to those presented in EN 1994-1-1, but with different values to cover different design situations. This more comprehensive and explicit approach allowed massive reductions in the minimum degree limits, as can be seen from Figure 2 (these curves are for a symmetric section, and show the influence of slip capacity and whether the beam is 80% or 100% utilised in bending). The P405 rules are incorporated in many current examples of design software, highlighting just how important this work was in facilitating economic beam design.

The rules that are currently included in prEN 1994-1-1 (due for publication by BSI in the next year or so) move the previous code approach very much in the direction of P405, although the rules for minimum degree are simplified and more conservative. This is for two reasons as explained below.

Firstly, P405 uses many pages to give different equations to cover different situations (propped vs unpropped etc), whereas it would not be possible for a design code to treat a single subject in this way. As a result, the rules are more succinctly presented in prEN 1994-1-1, possibly making them appear more complicated but at the same time including some simplification (and simplification is almost always achieved at the price of conservatism).

Secondly, rules given in international standards like the Eurocodes must always satisfy everybody involved, and whilst one might tolerate some excess conservatism, acceptance of something considered to be unconservative will not happen. As experts from different nations have different traditions and views on the benefits of economy vs safety, it is inevitable that the results will appear excessively conservative for some.

The extracts below are taken from the 2023 draft to illustrate the prEN 1994-1-1 approach (clause numbers are unlikely to change in the version to be published by BSI). The previous Eurocode equations are reproduced in 8.6.3.3(3) for symmetric and asymmetric sections, with the exception that the previous variable η is now defined as η_0.

In 8.6.3.3(2) various modification factors are applied to this 'basic' minimum degree η_0, that adjust the value to take into account the effects of part utilisation (ρ_m) and method of construction (k_{up}). As can be seen from the definitions below, both these variables have absolute limits that, similar to the traditional 40% lower bound, appear illogical given they relate to physical phenomena that have no discontinuities (there is no physical change in behaviour at certain spans, as the curves in Figure 2 suggest – the kinks are due to these absolute limits). Whilst they will ensure stud spacings are reasonable, they have the effect of limiting the benefits to be had from considering these variables. Extracts from 8.6.3.3(2) are given below:

$$\eta \geq \eta_0 \rho_m^2 k_{up} \geq \eta_{min}$$

$$\rho_m = \frac{M_{Ed}}{(0.95 M_{Rd(\eta)})}$$

But $0.8 \leq \rho_m \leq 1.0$

Where:

M_{Ed} is the applied moment

$M_{Rd(\eta)}$ is the moment resistance of the composite section with degree of shear connection η

When the steel section is propped during construction $k_{up} = 1.0$

When unpropped:

$$k_{up} = (1 - \rho_{up})$$

$$\rho_{up} = \frac{M_{a,Ed}}{M_{Pl,Rd}} \leq 0.15 \text{ when } \frac{M_{Ed}}{M_{Rd(\eta)}} \leq 0.95 \text{ otherwise } \rho_{up} = 0.0$$

$M_{a,Ed}$ is the moment applied to the steel section

$M_{Pl,Ed}$ is the moment resistance of the composite section assuming 100% shear connection

Absolute limits are also given in this clause:

$\eta_{min} = 0.4$ for studs in Ductility Category D2

$\eta_{min} = 0.3$ for studs in Ductility Category D3

The references to Ductility Category relate to studs with 6mm slip capacity (D2) and 10mm (D3). Studs in transverse trapezoidal decking will be D3, which relates this work to the P405 rules.

All very fascinating you may say, but what does it mean for design in the UK? SCI's recommendation would be that you carry on using the rules in P405, so no change! The P405 rules may reasonably be considered to be non-conflicting and complementary (NCCI), looking at the subject (or at least presenting outcomes) in more detail, which is why more accurate results can be obtained. The fact that Eurocode 4 has gone in a similar direction validates the work done by SCI a decade ago. The P405 guidance also addresses the prEN 1994-1-1 allowance to ignore the code rules for minimum degree if control of slip has been demonstrated in some other way (it's just that the designer is not doing this explicitly for themselves, SCI did it for them), which is part of a general Eurocode philosophy that the code rules should not prevent an expert designer doing something better.

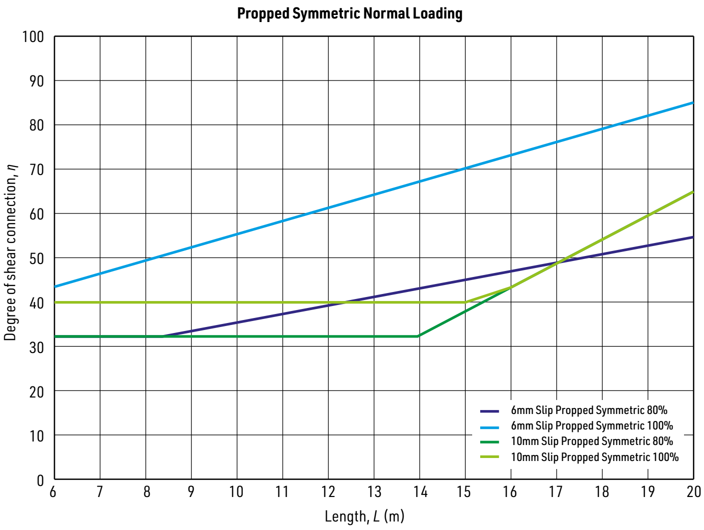

Figure 2: Minimum degree of shear connection vs span, according to the rules given in P405 (EN 1994-1-1 would only consider 6mm slip and 100% utilisation, so is represented by the upper curve)

And what of stud shear resistance?

As noted above, although they are used to limit slip (which is a function of stud stiffness) the rules for verifying minimum degree of shear connection rely on stud resistance as an input. It is therefore worth noting that prEN 1994-1-1 provides different ways of determining stud resistance, partly related to scope of application. When decking is present the traditional approach of reducing the resistance in a solid slab using a k factor can still be used, but the scope of slabs for which this can be applied has now been revised and will exclude much typical UK practice (for example requiring an extra embedment length for the stud above the deck). An alternative approach is given in an informative annex, but as well as scope issues the method is complicated and more conservative than traditional UK practice.

It is worth noting that the reasons for current UK practice now being excluded and/or appearing (relatively) unconservative are because the new work is based on a mechanical model, which has been shown to be excessively conservative. This was the price paid for developing a model that deals with a multitude of variables but needed to be simple enough to understand. SCI alumnus and current Eurocode 4 Chairman Prof Stephen Hicks at the University of Warwick has given further comment on this approach, including noting that it is 'unreliable'[5]. It is highly likely that when a UK National Annex for prEN 1994-1-1 is produced it will state this annex should not be used. Thankfully testing can be used as an alternative, and SCI's position is to exploit this rule to justify carrying on using the values we have been using for the past decade (and potentially use better values coming from the Warwick work in due course).

Conclusions

The second generation of Eurocodes will bring widespread change to the current rules. However, for composite beams the rules related to shear connection effectively endorse previous guidance produced by SCI, which provide more accurate results and it is recommended can still be used as NCCI, so there is no need to change current practice.

References

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3. BE EN 1994-1-1:2004 (incorporating corrigendum April 2009) *Eurocode 4: Design of composite steel and concrete structures –Part 1-1: General rules and rules for buildings*. BSI
4. Couchman G *Minimum degree of shear connection rules for UK construction to Eurocode 4*. P405. SCI, 2015
5. Degtyarev VV, Hicks SJ *Shear resistance of welded studs in deck slab ribs transverse to beams*, 2023, Engineering Structures, 294

Open car parks in fire

The behaviour of all structures in fire has come under close scrutiny in recent years, and since the unfortunate event at Luton Airport last year steel framed car parks are receiving particular attention, with different clients asking for different levels of resistance. Most structures are designed for a period of resistance, given in minutes, when subject to a standard ISO fire curve which defines the temperature at a given time. Whilst standardisation allows direct comparisons, the relationship between a 'standard fire' and a 'real fire' is not at all obvious. In this article Dr Yigit Ozcelik of SCI compares the flexural buckling resistance of columns exposed to fire, considering prescriptive and performance-based fire design approaches.

Introduction

The unfortunate fire incidents at the Luton Airport car park in 2023 and the Kings Dock car park in Liverpool in 2017 have raised fundamental questions around the fire safety of open sided car parks. Whether or not the recent fires show there is a problem with current designs is open to question (did these structures do what they were intended to do or not?), but the fact that vehicles have changed since the current guidance was written is undeniable and it is quite likely that design rules and regulations will be modified in the near future in order to improve the perceived safety of such car parks.

Approved Document B[1], providing guidance on the fire safety buildings in England, requires the structural frames of open car parks less than 30m in height to have 15 minutes fire resistance. (Note the AD is one way of meeting the requirements of the Building Regulations; however, there may be other ways to comply with the Building Regulations than following the guidance provided by the AD.[1]) At the time the AD was produced this period was deemed enough to allow evacuation should a fire take hold. However, various stakeholders are currently asking themselves if 15 minutes is adequate when modern vehicles are considered. For example, the IStructE Car Park Design Guide[2] suggests that such a low fire rating should be used with caution.

Modern vehicles are typically larger than older vehicles and they make greater use of plastic and synthetic materials. As a result, they have a larger fire load, and when plastic fuel tanks rupture there may be a greater tendency for a fire to spread between vehicles. Modern vehicles with alternative power sources such as LPG and lithium-ion batteries have different fire loads when compared to older vehicles[2]. A recent CROSS report[3] on fire risks states a 15-minutes fire resistance may not satisfy the functional requirement of the building regulations in multi-storey car parks occupied by modern vehicles. However, whilst a longer period of resistance, remembering this is resistance when exposed to a standardised 'fictional' fire, would undoubtedly increase fire capability, before concluding this is the correct way to go, we must remember that carbon and financial costs must also be taken into account in order to achieve the best overall solution. Adding fire protection that is not needed would not be a sensible thing to do.

Most of steel open sections used in the UK have 15 minutes inherent fire resistance[4]. Hence, currently, fire protection is rarely needed or used for steel members in open car parks. To satisfy a more stringent fire resistance period would almost certainly require protection, with a resulting increase in embodied carbon as well as construction and potentially maintenance costs.

An alternative approach to prescriptive fire design (considering a resistance period when subject to a standard fire) is so-called performance-based fire design. By following this design approach, it is possible to calculate temperatures and thereafter the resistance of steel members in fire more accurately. The example below shows that with this greater accuracy the fire protection that would be needed according to a prescriptive approach for a fire resistance period longer than 15 minutes might be eliminated or reduced. In other words, using more elaborate engineering could help meet budget and reduce embodied carbon.

The SCI recently undertook a preliminary study aiming to show that performance-based fire design is a promising tool that could be used to justify reduced or eliminated levels of fire protection for open sided car parks. Pursuant to this goal, an unprotected steel column of an open car park was examined considering distinct fire scenarios to quantify how its buckling resistance varies when different fire design approaches are adopted.

Fire design approaches and fire curves

The prescriptive approach for fire design typically used in the UK considers the ISO standard curve included in BS EN 1991-1-2[5], which is intended to model gas temperature in a fully developed compartment fire. Notably, this curve does not consider the decaying phase of a fire. Use of the ISO standard curve is reasonable for relatively small compartments where fire load is distributed uniformly. For large compartments or cases where the fire load is restricted to a relatively small area, use of the ISO standard curve is generally conservative. In cases such as fires in open car parks, use of a localised fire curve is more suitable to estimate steel temperatures. The publication SCI P423 *Design of columns subject to localised fire*[6] presents a method for determining the temperature of a column subject to a localised fire. This method adopts the software OZone [7]. In the current study, both the ISO standard curve and a localised fire curve based on a realistic scenario, described below, were used to compare their effect on column buckling resistance.

Fire scenario and temperature analysis

P423[6] includes a worked example considering an open car park with a length of 60m and a width of 45m. The ceiling height is 3.5m. The standard dimensions of the parking bays are 2.5m × 5m. The fire scenario considers three large cars and a van parked around a column and the fire starts from the car in the South-West direction of the column then spreads to the van in the North-West direction and the car in the South-East direction after 12 minutes. After another 12 minutes, the fire propagates to the car in the North-East direction. Figure 1 shows the heat release rates (HRRs) of the cars and the van.

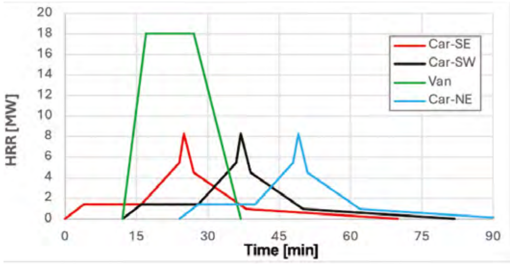

Figure 1: Heat release rate vs time

In addition to the fire scenario that assumed the ISO standard curve, the fire scenario in P423[6] was used in this study to develop localised fire curves in OZone[7]. The column section was assumed to be UC305×305×97, the most similar UK section to the European section HEA300 considered in P423[6]. As the localised fire model in OZone[7] can estimate the steel temperatures along the

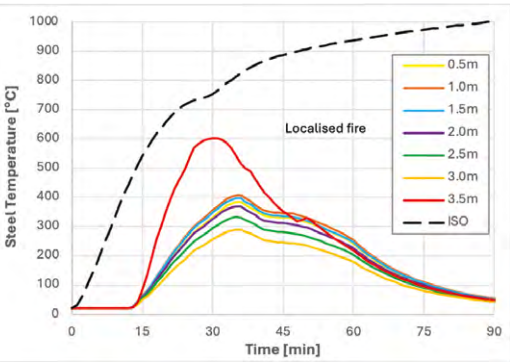

Figure 2: Steel temperature vs. time

column height, the steel temperatures were recorded at 0.5m intervals. The steel temperatures for the ISO standard curve and the localised fire curve are shown in Figure 2.

The steel temperature for the ISO standard curve is the same for the entire member. The steel temperatures at 30 minutes and 60 minutes were estimated as 750°C and 935°C, respectively. For the localised fire scenario, the maximum temperature recorded for each segment is between 300°C and 400°C except for the top segment whose maximum temperature is about 600°C. The main reason for this is a hot zone forms under the ceiling with a depth of approximately 0.5m. Note that the steel temperature decreases after reaching the peak value at approximately 30 minutes while it increases continuously when the ISO standard curve is considered. This suggests that the ISO standard curve not only overestimates the steel temperature but also fails to predict the shape of the steel temperature curve.

Flexural buckling resistance of column at elevated temperatures

Table 3.1 of BS EN 1993-1-2[8] tabulates the reduction factors for the stress-strain relationship of steel at elevated temperatures. To determine the flexural buckling resistance of the column, the reduction factors for yield stress (f_y) and modulus of elasticity (E) are required. Clause 4.2.3.2 of BS EN 1993-1-2[8] outlines a method for compression members; however, the method considers a uniform temperature for the member, which might disguise the full benefit of adopting a localised fire curve.

In this study, an isolated column model was considered to determine the flexural buckling resistance of the column in lieu of clause 4.2.3.2 of BS EN 1993-1-2[8]. To accurately represent the flexural buckling behaviour, the column was divided into several elements with an initial bow imperfection. The imperfection was perpendicular to the minor axis and assumed to be represented by a sine curve with an amplitude of $L/300$, where L is the length of the column. The properties f_y and E of each column segment were calculated considering the reduction factors given in BS EN 1993-1-2[8] per the steel temperatures shown in Figure 2. An axial compressive force was applied until the column buckled. The analysis model is shown in Figure 3.

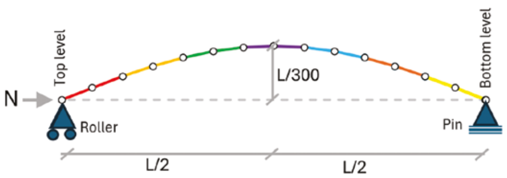

Figure 3: Structural model (colour coding is the same as the one in Figure 2)

For the flexural buckling analysis, three cases were considered:

- Case (1): Material properties determined for the maximum steel temperature per the ISO standard curve
- Case (2): Material properties determined for the maximum steel temperature along the column height per the localised fire scenario (i.e., the steel temperature of the onerous segment is considered.)
- Case (3): Material properties determined for the maximum steel temperature for each column segment separately per the localised fire scenario (i.e., seven reduction factors calculated for each material property, namely, f_y and E.)

The flexural buckling resistances of the UC section in S355 are given in Table 1. As can be seen from these results, use of a realistic localised fire instead of the ISO standard curve has a substantial impact on flexural buckling resistance. When the more conservative of the performance-based designs (Case (2)) is adopted, the flexural buckling resistance triples for 30 minutes and it is 8 times as much for 60 minutes fire resistance compared to Case (1). Similarly, considering the temperature variation along the column length further positively affects the buckling resistance (Case (3)), leading to an approximately 25% further increase in flexural buckling resistance. Another observation is that because the maximum steel temperature for the localised fire scenario occurs at approximately 30 minutes, the flexural buckling resistance remains almost the same for 30 minutes and 60 minutes fire resistances. On the other hand, when the prescriptive approach is adopted, the buckling resistance decreases significantly with the increase in temperature as the ISO standard curve does not acknowledge the decaying phase of fire. It can be concluded use of performance-based design in lieu of prescriptive design will have a more significant advantage for longer fire resistance periods.

Table 1 – Comparison of flexural buckling resistances for different fire design approaches

Fire resistance	Case (1)	Case (2)	Case (3)
30′	534 kN	1507 kN	1900 kN
60′	186 kN	1507 kN	1900 kN

Finally, the results shown in Table 1 were used to determine the maximum utilisation ratios for the ULS combination at ambient temperature beyond which fire protection would be needed to prevent the fire case design governing the member size. The flexural buckling resistance of a UC305×305×97 for a buckling length of 3.5m is given as 3440 kN in the Bluebook[9]. Assuming a reduction factor of 0.7 for the design load level for the fire situation, the factored design load for the fire situation, beyond which protection would be needed, is 0.7 × 3440 kN = 2408 kN for 100% utilisation at ambient. When the flexural buckling resistances given in Table 1 are divided by 2408 kN, the allowable utilisation ratios for ULS ambient design can be calculated. The results are given in Table 2.

Table 2 – Allowable utilisation ratios for ULS

Fire resistance	Case (1)	Case (2)	Case (3)
30′	0.22	0.63	0.79
60′	0.08	0.63	0.79

The values in Table 2 show that if a prescriptive approach was used, namely one considering exposure to the standard ISO fire curve, any column with a utilisation ratio at ambient temperature greater than 22% would need protecting to achieve 30 minutes, and any one with utilisation greater than 8% would need protecting for 60 minutes. In other words, they would all need protection.

However, using a localised fire scenario only columns that were more than 79% utilised at ambient would need protection. A small increase in section size could eliminate this need, or indeed a more accurate analysis might increase the utilisation limit.

Further research

It is recognised that because the study reported above is based on an SCI publication that was completed several years ago, the crucial aspect of modern vehicles has not yet been addressed. Its aim was simply to show the potential of this approach. Work is on-going at SCI to consider modern vehicles and extend the study to cover beams and perimeter columns. We will then have fact based quantified evidence, rather than a simple view that modern vehicles must be a significant problem for fire in car parks.

Conclusion

A preliminary study has been undertaken to quantify the advantages of performance-based fire design. Based on the results, it is believed performance-based fire design is a viable option to help prove unprotected columns have adequate buckling resistance during a fire in an open car park if a fire resistance period longer than 15 minutes is required. Consequently, fire protection that would come at significant cost and with embodied carbon implications could be avoided. Further research is ongoing to justify these initial findings. ■

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Preparing the way for 2028 changes in steel design

With the publication of BS EN 1993-1-1:2022, work can start developing "Generation 2" (Gen 2) resources in time for 2028 when the revisions will be implemented. David Brown of the SCI investigates some of the finer details.

The Blue Book

Despite the widespread use of software, the Blue Book remains an important part of the steel designer's tool kit – in paper form it is the SCI's best-selling publication and in electronic format on various sites is the most used resource. Work has commenced to revise the software routines used to generate the values presented in the tables – requiring some detailed investigation of the revised code and its implications. This article is probably the first in a series – it seems likely that more "interesting" (used here as a euphemism!) features will emerge as the work proceeds.

Section classification

Although classification limits for outstand flanges have not changed, the Gen 2 limiting values for webs have become more conservative. For members in compression the most important limit is when members become Class 4 (since the resistance calculations for Classes 1, 2 and 3 all use the gross area). The Gen 2 limit for $^{c}/_{t}$ reduces from 42ε to 38ε. Perhaps not terribly significant, but in S355, 10 additional sections become Class 4 in pure compression.

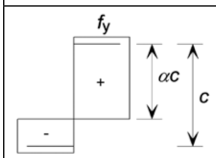
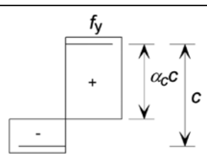
"*n*" limits

The Blue Book presents a column of "n limits" in the combined axial force and bending tables, where $n = N_{Ed}/A_g f_y$. As the axial compression N_{Ed} in a member is increased, a section which was Class 1 might become Class 2, and as the axial force is further increased, might become Class 3 and then Class 4. The "n limits" tell a designer the value of axial load when this change in Class happens.

The Class 2 limit is the increased level of axial force when a section becomes Class 3. There is no need to know when a Class 1 section becomes Class 2, since both Class 1 and Class 2 use the same section properties (gross area and plastic modulus) in the resistance calculations. The interest is when a section becomes Class 3 and the elastic section modulus must be used in the resistance calculations. In turn, the Class 3 limit is the increased level of axial compression when a section becomes Class 4 and effective properties must be used in the resistance calculations. For designers undertaking manual calculations, the "n limits" are a really useful way to quickly determine the section classification.

Extracts from the two standards showing the Class 2 limit are shown in Table 1, together with the formula for the Class 2 limiting value of n.

Table 1: Current and proposed Class 2 limiting values

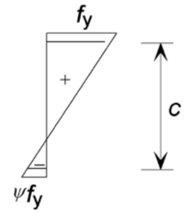
EN 1993-1-1:2005	EN 1993-1-1:2022 (Gen 2)
	
$n = \dfrac{2c_w t_w(\alpha - 0.5)}{A_g}$	$n = \dfrac{2c_w t_w(\alpha_c - 0.5)}{A_g}$
when $\alpha > 0.5$: $c/t \leq \dfrac{456\varepsilon}{13\alpha\text{-}1}$	when $\alpha_c > 0.5$: $c/t \leq \dfrac{188\varepsilon}{6.53\alpha_c\text{-}1}$
$n = \dfrac{c_w t_w}{6.5A_g}(4.56\varepsilon\, ^{t_w}/_{c_w} - 5.5)$	$n = \dfrac{c_w t_w}{3.265A_g}(188\varepsilon\, ^{t_w}/_{c_w} - 2.265)$

The initial expression for n in both Table 1 and Table 2 is determined from the stress blocks shown, given that $n = N_{Ed}/A_g f_y$. Rearranging the second expressions in terms of α (Table 1) and ψ (Table 2) and substituting in the expressions for n leads to the final formulae.

The Gen 2 Class 2 limit in Table 1 is 82.3% of the current limit, which put another way means that sections will become Class 3 at a significantly lower level of axial compression.

The comparison for the Class 3 limit is shown in Table 2.

Table 2: Current and proposed Class 3 limiting values

EN 1993-1-1:2005	EN 1993-1-1:2022 (Gen 2)
	
$\psi = 2n - 1$	
when $\psi > -1$: $c/t \leq \dfrac{42\varepsilon}{0.67+0.33\psi}$	when $\psi > -1$: $c/t \leq \dfrac{38\varepsilon}{0.608+0.343\psi+0.0.49\psi^2}$
$n = 63\, ^{t_w}/_{c_w} - 0.5$	n is the positive root of $0.196n^2 + 0.49n + (0.314 - 38\varepsilon\, ^{t_w}/_{c_w})$

Apart from the obvious additional complexity of the Gen 2 expression to calculate n, the Class 3 Gen 2 limit is again more demanding than the current code – or put another way, sections will become Class 4 at lower levels of axial compression. As c_w/t_w decreases (i.e. the heavier weights in a serial size), the difference between the current code and the Gen 2 version increases. The axial compression when a section becomes Class 4 can be around 10% lower under the Gen 2 rules.

Variable f_M

This new variable appears in the Gen 2 expressions for lateral torsional buckling – which have been covered previously in June 2021. Three years later, the detailed rules turn out not to be as simple as had been imagined. Factor f_M appears in Table 8.6 in the Gen 2 standard. The factor may conservatively be taken as 1.0 or calculated for certain shapes of bending moment diagram.

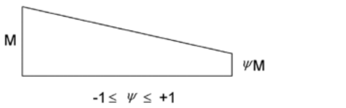
The Blue Book presents resistance values for certain values of C_1, which is itself a factor reflecting the shape of the bending moment diagram. A designer using the Blue Book must first determine the appropriate value of C_1 based on their shape of bending moment diagram. The Blue Book tabulates resistance values for C_1 values of 1.0, 1.13, 1.35, 1.5, 1.77, 2.0 and 2.5. The values in red are for "standard" bending moment diagram shapes – the other values are to allow interpolation. The "standard" cases also have specific values of f_M in Table 8.6 of the Gen 2 standard.

Since C_1 relates to the shape of the bending moment diagram, and f_M is also presented for different shapes of bending moment diagram, it seemed logical that there would be a general expression that linked C_1 to f_M. The factor C_1 can readily be calculated, so with a general expression to calculate f_M the lateral torsional buckling resistance could be computed. However, even in 2021 there was the hint of a difficulty. The "known points" connecting C_1 to f_M were plotted and an "inconstancy" identified.

The hunt for a general expression for f_M

The 2024 investigation began by using one of the simpler parts of Table 8.6, with a linear bending moment diagram, as shown in Table 3.

Table 3: Extract from Table 8.6 of BS EN 1993-1-1:2022 – linear BMD

Bending moment diagram	f_M
	$1.25 - 0.1\psi - 0.15\psi^2$

M_{cr} for a range of linear being moment diagrams was computed, and C_1 calculated using the expression given in the UK National Annex. The selected beam was a 254 × 146 × 31, 6m long. For a uniform BMD, M_{cr} = 53 kNm. The following table lists some of the data points calculated.

ψ	M_{cr} (kNm)	C_1	f_M
0.45	72.06	1.36	1.17
0.60	65.83	1.24	1.14
0.0	97.03	1.83 (1.77 anticipated)	1.25
-0.2	111.33	2.1	1.26
-0.5	134.36	2.54	1.26
-1	146.68	2.76	1.2

The full relationship is shown in Figure 1. The circled data points are two of the "standard" values from Table 8.6 of the Gen 2 code, for a UDL and for a central point load, both with pinned ends. The circled points are an early indication that f_M is not related directly to C_1.

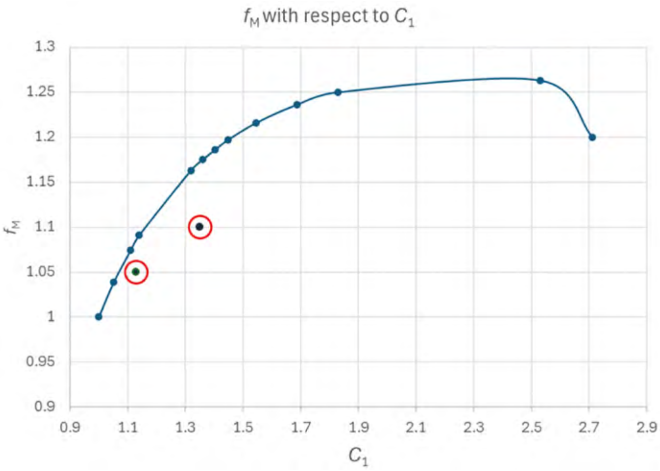
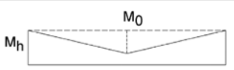
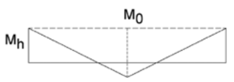


Figure 1: Relationship between C_1 and f_M for a linear BMD

The next step was to investigate other rows in Table 8.6, starting with a member with end moments and a central point load, as shown in Table 4.

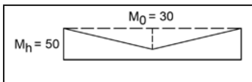
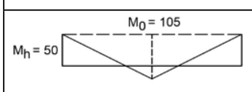
Table 4: Extract from Table 8.6 of BS EN 1993-1-1:2022 – end moment and point load

Form of bending moment diagram	f_M	
	For $0 \le \frac{M_0}{M_h} < 2.0$	$1.0 + 1.25\frac{M_0}{M_h} - 0.3\left(\frac{M_0}{M_h}\right)^3$
	For $\frac{M_0}{M_h} > 2.0$	1.1

The form of the function for f_M is shown in Figure 2, which suggests a simple relationship with C_1 is unlikely.

The problem becomes apparent when a range of bending moment diagrams are considered and C_1 calculated. Two examples are shown in Table 5, for the same 254 × 146 × 31 UB.

Table 5: Examples of C_1 and f_M – end moments and point load

	$\frac{M_0}{M_h} = 0.6$	$f_M = 1.69$	M_{cr} = 89.3 kNm	$C_1 = 1.69$
	$\frac{M_0}{M_h} = 2.1$	$f_M = 1.10$	M_{cr} = 90.2 kNm	$C_1 = 1.70$

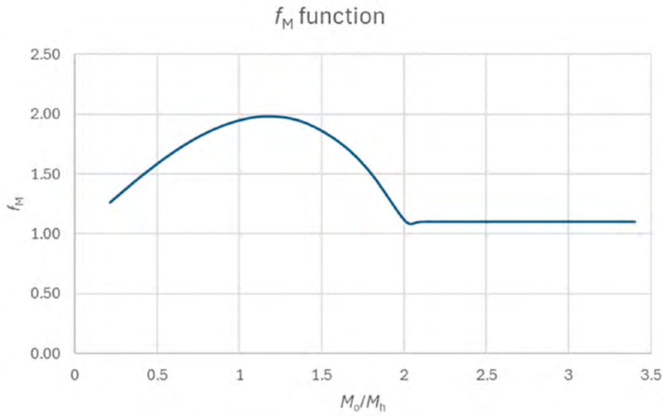
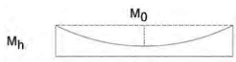
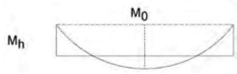
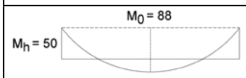


Figure 2: Relationship between M_0 / M_h and f_M - end moments and point load

Two bending moment diagrams which both lead to the same value of C_1 produce a quite different value of f_M. From Figure 1, the value of f_M at C_1 = 1.7 is 1.24, so different again.

A single example with a distributed load and end moments is shown in Table 6, for the same 254 × 146 × 31 UB.

Table 6: Examples of C_1 and f_M – end moments and distributed load

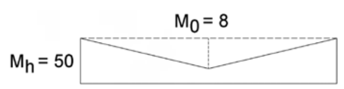
Form of bending moment diagram	f_M			
	For $0 \le \frac{M_0}{M_h} < 2.0$	$1.0 + 1.35\frac{M_0}{M_h} - 0.33\left(\frac{M_0}{M_h}\right)^3$		
	For $\frac{M_0}{M_h} > 2.0$	1.05		
	$\frac{M_0}{M_h} = 1.76$	f_M = 1.97	M_{cr} = 89.2 kNm	$C_1 = 1.68$

For the same value of C_1 four values of f_M have now been calculated: 1.1, 1.24, 1.69 and 1.97. The inescapable conclusion is that there is no direct link between C_1 and f_M.

The way forward for the Blue Book

One might conclude that we at least know the value of f_M for certain values of C_1, for example when C_1 = 1.13, we know that f_M = 1.05, but even this is not correct. We only know that f_M = 1.05 for a parabolic bending moment diagram with zero end moments. This shape of this particular bending moment diagram does lead to C_1 = 1.13, but we could arrive at C_1 = 1.13 from a variety of bending moment diagram shapes, when it would not be correct to assume f_M = 1.05, as the above investigation demonstrates. Two examples when C_1 = 1.13 are shown in Table 7.

Table 7: Examples of C_1 and f_M – different shapes of BMD

Bending moment diagram	C_1	f_M
	1.13	1.05
	1.13	1.21

One option would be to assume that f_M = 1, which is a conservative choice. The Gen 2 resistances are generally lower than currently, so to lose even more resistance by setting f_M = 1 is not an attractive option. The impact of setting f_M = 1 varies but can reduce the resistance by around 10 - 12 % at some spans.

The *current* proposed solution is to include the benefit of f_M where it is known, so for C_1 values of 1.0, 1.13, 1.35 and 1.77. A health warning will have to be made that in the Gen 2 Blue Book, the values of 1.0, 1.13, 1.35 and 1.77 indicate the *shapes* of the bending moment diagram, not the calculated value of C_1. Designers will no longer be able to determine C_1 for any shape of bending moment diagram and interpolate within the table. Multiple shapes of bending moment diagram can lead to C_1 = 1.5, 2.0 and 2.5, meaning the benefit of f_M cannot be incorporated in the resistance values, so the values for those rows will be conservative.

The current proposal results in inconsistent inclusion of f_M, meaning some values will be conservative, and others will be accurate. As C_1 increases in value (going down the table, one row to the next), the LTB resistance should increase. This will not always be the case if the row with the higher C_1 value sets f_M = 1, which can result in a lower resistance despite the increase in C_1.

The variable f_M could be set to 1.0 for all values of C_1. The resistance tables would then appear logical and consistent, but the published resistances would be conservative. Feedback from users on the preferred way forward would be helpful.

Conclusions

This article has examined just two of the changes presented in the Gen 2 version of BS EN 1993-1-1. The effect of these changes has significant repercussions for familiar design resources and will also impact design software. Commercial software design packages for Gen 2 will face the same question – how is f_M to be determined for a general case? ■

Dealing with multiple point loads on a composite slab

Whilst placing concentrated loads on composite slabs is not particularly recommended, we are seeing more and more situations where multiple, sometimes quite significant, loads are placed on a slab. Dr Graham Couchman from SCI discusses how composite slabs support concentrated loads, how software designs these slabs, and highlights issues to be taken into account when numerous such loads are present on a given area of slab. He also reminds designers about the need to explicitly consider the transverse reinforcement, normally mesh (fabric), that is needed to distribute a load, and the confusion that has arisen over the definition of a load that is sufficiently small to not warrant this check given in EN 1994-1-1[1].

Why are concentrated loads a concern?

A composite slab supports loads because it has resistance to moment, shear etc. The moment resistance is achieved by the axial force in some of the concrete in compression balancing the axial force in the steel decking in tension, forming a couple. The axial force in the decking is governed by the mechanical shear interaction between the steel and concrete, up to the point at which the steel yields. For typical deck geometries, steel grades and spans the deck does not reach yield and shear interaction dictates the moment resistance.

Shear interaction is achieved through the embossments that are rolled into the deck, and the shape of the re-entrant parts of its profile. This means that the force that can be transferred between steel and concrete is a function of the contact area between the two, therefore for a given width force transfer increases with distance into the span. The moment resistance of the slab increases to a maximum at mid-span. This is analogous to a composite beam, except in the latter case force transferred increases as each stud is 'passed', working away from the support. With an off-centre concentrated load the maximum applied moment, which may also be off-centre depending on relative sizes of coincident concentrated and uniform loading, must be compared with the resistance at the same point in the span (not the maximum resistance). Near a support this resistance could be considerably less than the maximum resistance at mid-span.

A second concern when considering concentrated loads, although this may be more theoretical than practical, is that shear interaction values are determined from tests, and these tests only ever consider unform loading. It is assumed that the interaction that can be achieved per unit contact area will not vary as a function of the type of loading.

How are concentrated loads supported?

The first thing to remember when considering their behaviour is that composite slabs are assumed to be one-way spanning. This is not an unreasonable assumption given the ribs run in one direction only. However, they do still clearly have some stiffness in the orthogonal direction. When a concentrated load is placed on a slab it is assumed to distribute laterally over a width that comprises the stiff bearing width, plus the width achieved by 45 degree distribution through the concrete, plus an additional width due to the transverse stiffness of the slab (see Figure 1, which is a reproduction of EN 1994-1-1 Figure 9.4). The later depends on where in the span the load is placed. All this is quantified in EN 1994-1-1 clause 9.4.3.

For bending and longituinal shear, for simple spans (EN 1994-1-1 Eq. 9.2):

$$b_{em} = b_m + 2L_p\left(1 - \frac{L_p}{L}\right) \leq \textit{slab width}$$

For vertical shear (EN 1994-1-1 Eq. 9.4):

$$b_{em} = b_m + L_p\left(1 - \frac{L_p}{L}\right) \leq \textit{slab width}$$

L_p is the distance of the centre of the load from the nearest support.
L is the span length.
b_m is the stiff bearing width.
b_{em} is the effective width of the longitudinal strip carrying the load.

Not stated, but the effective widths defined in EN 1994 are maximum values – the load should not be considered to be supported by a greater width when verifying the various resistances (bending, longitudinal shear and vertical shear). This is what a designer would normally want, as it places the least demand on the slab in the direction of span. The rules also assume a certain (unstated but typical) transverse stiffness for the slab. If the slab had less transverse stiffness than this unstated value, the effective widths defined by EN 1994-1-1 could not be achieved. In the extreme one can imagine a slab that had no transverse stiffness and therefore could not resist transverse bending – the load would simply be carried on a longitudinal strip of width defined by the stiff bearing width plus 45 degree distribution through the concrete.

A method for determining the magnitude of transverse moment present in the longitudinal strip, and therefore how much transverse reinforcement is needed to support a given load, is defined in AD450[2]. AD477[3] takes this further, and introduces the idea of reducing the width of a longitudinal strip down to the minimum that will still support the load spanning between end supports, in order to reduce transverse demands. The principle is easy to understand, but the implementation can get complex so SCI has produced a Tedds module[4] that does it for you. It should also be noted that the EN 1994 allowance to assume nominal mesh is sufficient when concentrated loads do not exceed certain limits (9.4.3(5)) has long been misunderstood (including by SCI, with an incorrect explanation given in P359[5]) and should not be relied upon. Recent investigations into the origin of this rule revealed it only applies to slabs that are far different from many designed in the UK (for example the mesh is assumed to be laid directly on the decking, and only one concentrated load may be present in what may be rather a large area of slab).

What does design software do?

It is important to understand that composite slab design software relies on the one-way spanning characteristic to simply design a 1m strip of slab. A clue is the fact that input values do not ask for the 'width' of slab, only the span. If there are concentrated loads present on a floor plate, a designer will consider the region(s) where those loads are applied. For its 1m strip, the software will take into account any uniform load that is present, plus whatever proportion of the concentrated load is acting on the strip (so although the input may define a concentrated load P as present, if the EN 1994 rules distribute that load over say 2m then only 0.5 P acts on the 1m strip designed by the software).

What about overlapping loads?

The fact that a given concentrated load may be carried by a longitudinal strip that has a width in excess of the 1m designed by software gets complicated if you have adjacent – side-by-side - concentrated loads. Although not present on the line of slab the designer assumed to be most critical they could still affect it (Figure 2 b). For a typical slab spanning 3m, the EN 1994 rules tell us that a concentrated load placed at mid-span will be carried by a strip of width around 2m. If you had two adjacent loads 1m apart, the critical 1m strip would be centred about the mid-point between the loads (not about a line on which one of the loads was present), and subject to both loads. Failure to take this into account, and instead design a strip that one of the loads was directly applied to, could result in a significantly under designed slab. To avoid this some side calculations may be needed to increase the level of load used as a software input.

We can also envisage more complex situations where adjacent loads well into the span (so with significant effective width) overlap on a strip between their points of application, but there are other loads, on the same adjacent lines, near the support that will not overlap because they have a smaller effective width. It then becomes less easy to predict which is the most critical strip, and more than one case may need to be designed.

Conclusions

It seems that composite slabs are more-and-more being used in situations where there are numerous concentrated loads present. It is therefore more important than ever that designers using slab design software have a good understanding of how composite slabs behave, particularly the way they support concentrated loads. Before deciding what concentrated loads to include as inputs, designers should ensure there are no adjacent loads that could also affect a given area of slab. ■

References

1. BS EN 1994-1-1:2005, *Eurocode 4 - Design of composite steel and concrete structures - Part 1-1: General rules and rules for buildings*, BSI, 2005.
2. AD450 *Resistance of composite slabs to concentrated loads*, SCI
3. AD477 *Transverse bending of composite slabs subjected to point loads*, SCI
4. *https://steel-sci.com/sci-tedds-modules-for-specialist-steel-design.html*
5. P359 *Composite design of steel framed buildings*, SCI, 2011

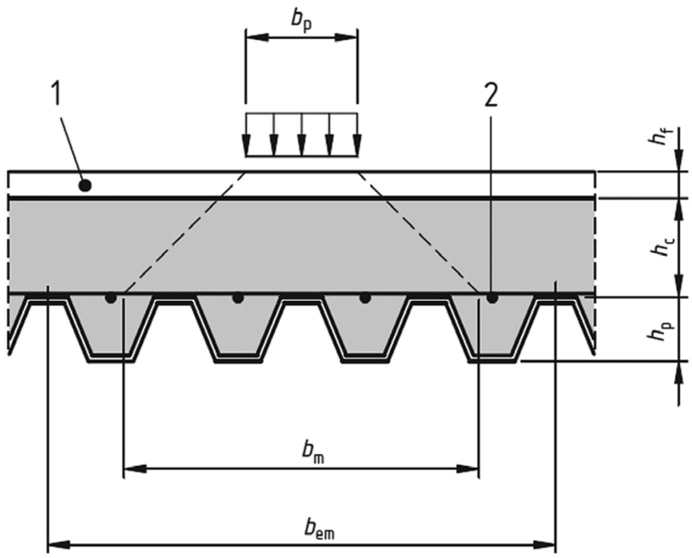


Figure 1: Distribution of concentrated load

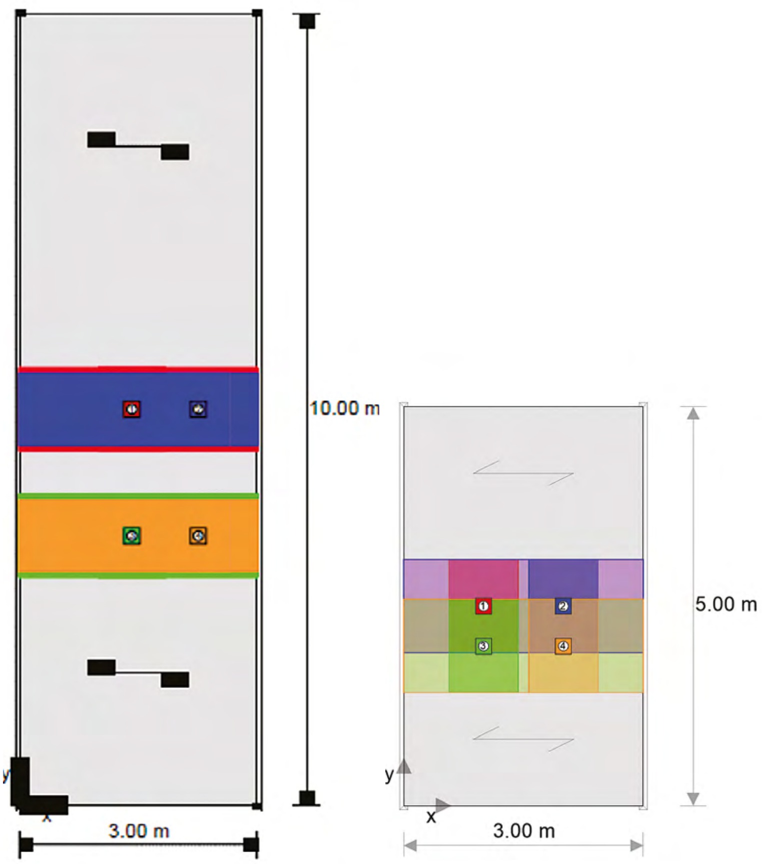


Figure 2: Tedds output showing effective widths supporting lines of loads with a) no overlap b) overlap (of green and pink longitudinal strips)

Advisory Desk 2024

AD 519: Equivalent horizontal forces and combinations of actions

Recent questions to SCI's Advisory Desk have shown that some designers are unclear whether equivalent horizontal forces (EHFs) should be applied to a structural frame in serviceability and accidental load combinations.

EHFs are intended to allow for frame imperfections such as a lack of verticality (the frame is out of plumb), which is a geometrical imperfection as described in BS EN 1993-1-1:2005 clauses 5.3.1 and 5.3.2. A lack of verticality of the structural frame produces lateral effects, since the vertical loads are eccentric to the base positions. Frame imperfections lead to increased design forces in a stability bracing system, or increased sway moments in a continuous frame. EHFs must be included in all ultimate limit state verifications, unless the condition discussed below is satisfied.

Serviceability load cases are intended to determine the effects of variable actions on the users of a building and on brittle finishes. EHFs do not have to be included in serviceability load combinations.

Accidental combinations of actions are an ultimate limit state verification and therefore the EHFs should be included.

EHF do not need to be included in ultimate limit state verifications when the externally applied lateral loads are relatively high compared to the vertical loads. As noted in BS EN 1993-1-1 clause 5.3.2 (4), EHFs need not be included when:

$H_{ED} \geq 0.15\ V_{Ed}$

where:

H_{ED} is the design value of the horizontal loads,

V_{Ed} is the design value of the vertical loads.

Contact: **Liam Dougherty**
Telephone: **01344 636555**
Email: **advisory@steel-sci.com**

AD 520: Amendment to Table 8 in the MPS-Bolts (issue 14)

The BCSA has reviewed the requirements for *impact test for machined test pieces* in Table 8 of the *Model specification for the purchase of structural bolting assemblies and holding down bolts* (MPS-Bolts) following discussions with bolt suppliers. Currently Table 8 requires a sample size of three bolts, per manufacturing lot, for property classes up to and including 8.8, and five bolts for property class 10.9, see table below.

Property (see Table 3 of BS EN ISO 898-1:2013)		Test method	Reference	Sample size	
No.				≤8.8	10.9
18	Impact strength, K_v	Impact test for machined test pieces	Clause 9.14 of BS EN ISO 898-1:2013	3	5

Table 8 – Inspection and testing requirements for bolts of property class 4.6 in all diameters and property class 8.8 and 10.9 up to M39

For bolts used in normal UK temperatures (T), (i.e. minimum -15°C), a sample size of one bolt, per manufacturing lot, for any property class, was found to be more appropriate. This is in line with Table 9 of the MPS-Bolts.

For bolts used in temperatures below -15°C but above -50°C, a sample size of three bolts, per manufacturing lot, for all property classes should be specified. For bolts used below -50°C, the purchaser should discuss the sample size with a bolt metallurgist and consider increasing the sample size. The new sample size should be communicated to the bolt supplier on the purchase order.

It is the responsibility of the purchaser to confirm the sample size at the time of order, otherwise it will be assumed that the fasteners will not be used at service temperatures below -15°C.

The revised Table 8 for evaluation of the impact strength should read as the table below:

Property (see Table 3 of BS EN ISO 898-1:2013)		Test method	Reference	Sample size		
No.				T (°C)	≤8.8	10.9
18	Impact strength, K_v	Impact test for machined test pieces	Clause 9.14 of BS EN ISO 898-1:2013	-15	1	1
				-50<T -15	3	3
				≤ -50	Seek advice	

Table 8 –Inspection and testing requirements for bolts of property class 4.6 in all diameters and property class 8.8 and 10.9 up to M39 (revised)

Contact: **Ana M. Girão Coelho**
Telephone: **020 7747 8127**
Email: **ana.girao-coelho@bcsa.org.uk**

AD 524: Composite slabs and minimum reinforcement limits

Reinforcement in composite slabs is not only required to provide resistance to the slab but also to prevent cracking of concrete. Codified limits to the minimum amount of reinforcement often exist to avoid more complex calculation (of limiting crack widths for example). In this AD note, the reinforcement requirements for composite slabs and the reasons behind current codified minimum reinforcement limits are discussed.

Purpose of reinforcement in composite slabs

In addition to the external reinforcement provided by the profiled steel decking, composite slabs require some sort of 'internal' reinforcement (whether it is mesh, bars or fibres) to be provided for a number of reasons. The most obvious one is control of cracking under negative (hogging) bending of the slab at the supports. More reinforcement is required in propped construction because the slab is also subject to additional loads on removal of the props. The resistance of the slab in the event of fire also influences the required reinforcement, which may include additional bars in the slab ribs. During fire, the steel deck will lose most of its strength due to the development of high temperatures, and therefore tension reinforcement embedded in the concrete is required to enable the slab to maintain part of its load-carrying capacity at the fire limit state. For this purpose, either mesh or fibre reinforcement over internal supports is used to provide hogging moment resistance to compensate for the reduced sagging bending resistance. In some cases, additional bars are placed inside the troughs of the steel deck to enhance the sagging resistance, particularly when the slab is single spanning. The reinforcement remains at a lower temperature than the steel decking and so is more effective in fire.

Reinforcement is also required locally over supporting composite beams to provide resistance to the longitudinal shear force transferred into the slab via the shear connection (headed studs). In that case, reinforcement resists splitting or shearing of the concrete flange along any of the potential surfaces shear failure - see clause 6.6.6 (4) of BS EN 1994-1-1:2004 (Figure 6.16). Reinforcement also contributes to the vertical shear resistance of the slab. In cases where the steel decking is discontinuous across the top flange of a supporting beam and the steel deck is not connected to the beam with thru-deck welded studs, the use of properly anchored bars or fibre reinforcement is essential.

Other purposes served by reinforcement include the enhancement of the slab resistance in bending and shear under high locally applied loads (including those due to walls placed on the slab and plant loads), tying action for structural integrity and the distribution of strains due to concrete shrinkage and creep under sustained loads.

Codified minimum reinforcement limits

Clause 9.8.1(2) of BS EN 1994-1-1:2004 [1] requires that, for continuous slabs designed as simply supported, the 'anti-crack' reinforcement above the ribs has a minimum area of 0.2% (for unpropped construction, 0.4% for propped construction) of the area of concrete above the ribs. Clause 7.4.1(4) of BS EN 1994-1-1:2004[1] also requires that, for slabs that are continuous over beams that are designed as simply supported, the 'longitudinal' reinforcement provided over the effective width of the concrete flange has a minimum area of 0.2% (for unpropped construction, 0.4% for propped construction) of the area of concrete above the ribs. A greater reinforcement area may be required when it is necessary to control cracking.

Both the above limits refer to slabs that are physically continuous, even though slabs are designed as simply supported at ambient temperature, which is typically the case in steel framed buildings. However, there are other applications where the slab may be discontinuous over the supports (simply supported). Such applications are typically found in light-gauge steel framed construction where panel head members split the slab over light steel walls. For such applications, the limits specified in these clauses do not need to be satisfied. However, there may be other reasons that could lead to a similar or even higher amount of reinforcement.

Cases when a higher amount of reinforcement may be required

When a composite slab needs to accommodate concentrated loads of significant magnitude, a higher amount of reinforcement (higher than that required for other purposes) will normally be required, particularly in the transverse direction to the orientation of the ribs. clause 9.4.3 (5) of BS EN 1994-1-1:2004 suggests a nominal transverse reinforcement with an area of at least 0.2% the area of the concrete above the ribs may be used (without further analysis/calculation), for cases where the concentrated loads are of a certain magnitude. However, this clause is misleading because it fails to provide sufficient context. SCI has published more recent guidance [2,3] on concentrated loads, also interpreting the specific clause. It is clear that cases with concentrated loads will often require higher amounts of reinforcement (mesh or fibres).

Furthermore, a higher amount of reinforcement may be required when a slab is more vulnerable in terms of cracking and deflections due to combined shrinkage and creep of concrete. For example, slabs carrying permanent loads, such as these in storage applications, in plant rooms or heavy superimposed dead loads, would need more reinforcement, particularly at the supports. Clearly exposure conditions also influence the need to control cracking.

Conclusions

The purposes the reinforcement serves in a composite slab are many. Current codified minimum reinforcement limits are there to control cracking in continuous slabs, which are typically designed as simply supported for the normal stage, or to avoid complex calculations when a slab is loaded by moderate magnitude concentrated loads. These limits are not always applicable, as is the case with discontinuous, simply supported slabs used typically in light gauge steel framed construction. However, significant amounts of reinforcement (similar or higher to the limits) may still be required when high concentrated loads act on the slab or when high permanent loads are applied to the slab.

References

[1] BS EN 1994-1-1:2004, Eurocode 4: Design of composite steel and concrete structures. General rules and rules for buildings. BSI, 2005.
[2] AD 450: Resistance of composite slabs to concentrated loads, SCI, October 2020.
[3] AD 477: Transverse bending of composite slabs subjected to point loads, SCI, February 2022.

Contact: **Eleftherios Aggelopoulos**
Telephone: **01344 636555**
Email: **advisory@steel-sci.com**

AD 527:
Hybrid connections with bolts and welds

SCI's Advisory service occasionally receives questions about connections where load is to be shared between bolts and welds. This AD explains the code requirements and why hybrid connections are generally not recommended.

So-called hybrid connections, where load might be shared between bolts and welds are covered by clause 3.9.3 of BS EN 1993-1-8. The clause permits Category C bolts (non-slip at ULS) to share load with welds, provided the final tightening of the bolts is carried out after welding is complete.

Non-preloaded bolts transfer load in shear and bearing. The bearing deformation and the movement in clearance holes mean that if this category of fixing were used in a hybrid connection, all the load would actually be carried by the welds, since the welds prevent movement. The same principle applies for hybrid connections using Category B bolts, as these are assumed to slip between SLS and ULS.

Using Category C bolts, preloaded after completion of the welding, precludes slip, so it can be assumed that the welds and Category C bolts share the load.

SCI is not aware of any guidance on how the force might be shared between the bolts and welds. The situation is unlikely to be simple as it will depend on the stress distribution through the connection, which will be affected by the arrangement of bolts and welds. Owens and Cheal[1] point out that the strength and stiffness of fillet welds vary substantially with the direction of the applied load. A second comment is that an elastic analysis based on a single value of weld stiffness cannot be accurate; the limited ductility of the weld precludes the use of simple plastic analysis. It may be possible to undertake a finite element analysis (FEA) of a hybrid connection, though SCI's experience is that FEA is often not straightforward.

The guidance on hybrid connections is not new – identical guidance is given in clause 6.1.1 of BS 5950, but SCI's advice is that hybrid connections should not adopted without very careful consideration of the force distribution within the connection.

[1] Owens, Graham W.; and Cheal, Brian D. 1989. *Structural steelwork connections*, Butterworth & Co. Ltd, London, UK

Contact: **Advisory Desk**
Telephone: **01344 636555**
Email: **advisory@steel-sci.com**

AD 528:
Lateral restraint forces for beams

SCI's Advisory Desk has received queries from designers as to what restraint forces should be used to restrain the compression flange of a beam. This AD Note compares the lateral restraint force requirements for BS EN 1993-1-1[1] and BS 5950-1[2].

To use a steel beam economically, the compression flange needs to be restrained laterally against buckling and two requirements may be identified for all restraint systems[3,4]:

1. The restraint should have sufficient stiffness to increase the buckling load of the restrained member to the desired level by limiting the buckling deformations.

2. The restraint should have sufficient strength to resist the loads transmitted as a result of restricting the buckling deformations.

The relationship between stiffness and strength is such that the greater the stiffness of the restraint, the smaller its required strength. Despite the importance of both strength and stiffness, many structural design codes provide only strength requirements (e.g. BS 5950-1) and it is assumed that a member of such strength will also possess sufficient stiffness. Long span structures will develop large restraint forces and additional checks may be required.

In BS 5950-1, the restraint force required is straightforward, in BS EN 1993-1-1, the approach is more detailed.

BS EN 1993-1-1

In BS EN 1993-1-1, restraint is dealt with by assuming an initial geometric imperfection. The initial geometric imperfection may be replaced by an equivalent stabilising force q_d, defined by Equation 5.13 of BS EN 1993-1-1, which is applied as a uniformly distributed load on the member to be resisted by a bracing system.

The equivalent stabilising force q_d is defined in clause 5.3.3(2) of BS EN 1993-1-1 as:

$$q_d = \sum N_{Ed} 8 \frac{e_0 + \delta_q}{L^2}$$

Where:

N_{Ed} is the axial force in the compression flange of the beam, taken as:

$$N_{Ed} = \frac{M_{Ed}}{h}$$

M_{Ed} is the maximum moment in the beam

h is the overall beam height

Where a beam is subjected to external compression N_{Ed}, it should include the part of the compression force carried by the flange.

e_0 is the member imperfection defined by Equation 5.12 of BS EN 1993-1-1 as:

$e_0 = \alpha_m L/500$

α_m is a reduction factor when multiple beams are being restrained by a bracing system, given in clause 5.3.3(1) of BS EN 1993-1-1 as:

$$\alpha_m = 0.5\left(1 + \frac{1}{m}\right)$$

m is the number of members to be restrained

L is the length of the beam

δ_q is the in-plane deflection of the bracing system under q_d plus any external loads. The in-plane deflection of the bracing system could have a significant impact on the stabilising force. SCI's P360 suggests that the deflections of typical bracing systems in buildings are unlikely to exceed $L/2000$ and a useful approach is to assume initially (and subsequently confirm) that the deflection of the bracing system δ_q will be less than this conservative value. The total resulting equivalent stabilising force ($q_d L$) is then 2% of N_{Ed}.

Where two or more intermediate lateral restraints are provided, P360 suggests that each restraint should be capable of resisting a force of not less than $5q_d L/8$. Provided that the actual deflection of the bracing system δ_q is less than the L/2000, the restraint force equals 1.25% of N_{Ed}.

The restraints should also be capable of resisting any additional forces due to external actions and it must be ensured that sum of the restraint forces for the individual beams are transferred to some 'stiff' point in the structure, for example, to in-plane bracing or concrete core walls.

BS 5950-1

BS 5950-1, clause 4.2.2 says that full lateral restraint may be assumed to exist if the frictional or positive connection of a floor (or other) construction to the compression flange of the member is capable of resisting a lateral force of not less than 2.5% of the maximum force in the compression flange of the member.

Similarly, clause 4.3.2.2 says that where intermediate lateral restraint is required at intervals within the length of a beam, the intermediate lateral restraints should be capable of resisting a total force of not less than 2.5% of the maximum value of the factored force in the compression flange within the relevant span, divided between the intermediate lateral restraints in proportion to their spacing.

Where three or more intermediate lateral restraints are provided, each intermediate lateral restraint should be capable of resisting a force of not less than 1% of the maximum value of the factored force in the compression flange within the relevant span.

The intermediate lateral restraints should either be connected to an appropriate system of bracing capable of transferring the restraint forces to the effective points of support of the member, or else connected to an independent robust part of the structure capable of fulfilling a similar function.

The bracing system should be capable of resisting each of the following alternatives:

a) the 1% restraint force considered as acting at only one point at a time and
b) the 2.5% restraint force divided between the intermediate lateral restraints in proportion to their spacing

Clause 4.3.2.2.3 requires that bracing systems that supply intermediate lateral restraint to more than one member should be designed to resist the sum of the lateral restraint forces from each member that they restrain, reduced by the factor k_r obtained from:

$k_r = (0.2 + 1/N_r)^{0.5}$

N_r is the number of parallel members restrained.

Conclusion

Both BS EN 1993-1-1 and BS 5950-1 result in similar lateral restraint forces, for full lateral restraint a force equal to 2% and 2.5% of the axial force in the compression flange respectively and for intermediate lateral restraints a force equal to 1.25% and 1% of the axial force in the compression flange respectively. However, in BS EN 1993-1-1 the determination of restraint forces is an iterative process, due to the dependence of the forces on the level of deflection of the bracing system.

Both approaches include a reduction factor on the restraint forces to bracing systems when multiple beams are being restrained.

Long span structures will develop large restraint forces and additional checks may be required.

1 BS EN 1993-1-1:2005 *Eurocode 3 - Design of steel structures - General rules and rules for buildings*, BSI
2 BS 5950-1:2000 *Structural use of steelwork in building. Code of practice for design. Rolled and welded sections*
3 Nethercot, D.A. and Lawson, R.M. *Lateral stability of steel beams and columns (P093)*, SCI, 1992
4 Gardner, L. *Stability of steel beams and columns (P360)*, SCI, 2011

Contact: **Liam Dougherty**
Telephone: **01344 636555**
Email: **advisory@steel-sci.com**

AD 530: Countersunk head bolts

Countersunk head bolts may be manufactured with a slot (for a screwdriver), see Fig. 1, or a hexagonal socket (for an allen key driver), see Fig. 2. This AD is to advise that countersunk bolts with a hexagonal socket may have reduced tensile resistance due to the reduced section at the head-to-shank location and should be used with care.

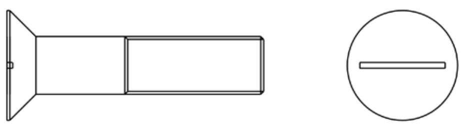


Figure 1: Countersunk headbolts with a slot

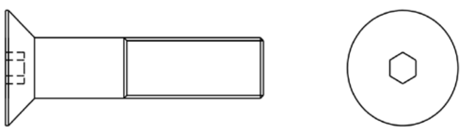


Figure 2: Countersunk headbolts with a hexagonal socket

This note only applies to non-preloaded countersunk head bolts. Countersunk head bolts used as a preloaded assembly are manufactured to BS EN 14399-7, have a screwdriver type slot and have full loadability. Preloaded countersunk head bolts to BS EN 14399-7 may be used without preloading.

Requirements for non-preloaded countersunk head bolts are specified in BS 4933. The Note to Table 8 of BS 4933:2010 permits the forming of a feature to prevent rotation at the choice of the manufacturer. The note goes on to state that the feature should not reduce the "loadability" of the fixing when subject to an axial tensile force. The note provides a forward reference to BS EN ISO 898 for further guidance.

BCSA has produced a Model specification for the purchase of structural bolting assemblies and holding down bolts which currently (14th edition) requires that countersunk bolt assemblies subject to tensile loads, or combined shear and tensile loads should only be supplied with a screwdriver slot, unless an alternative can be demonstrated to not adversely affect the bolt loadability.

BS EN ISO 898-1:2013 specifies mechanical properties for fasteners in carbon and alloy steel. The scope recognises that certain fasteners might not fulfil the tensile requirements because of the geometry of the heads, including those with a countersunk head. Clause 8.2 of BS EN ISO 898-1:2013 identifies the geometric reasons why a fixing might have reduced loadability, including a countersunk head with an internal driving feature (a hexagonal socket).

Despite the possible inference in the scope that reduced loadability fasteners are not covered, BS EN ISO 898-1:2013 specifies testing requirements for reduced loadability fasteners in Table 10, requiring that (among other things) the fastener achieves at least the minimum ultimate tensile load in the product standard.

The product standard for countersunk fasteners with socket heads is BS EN ISO 10642:2019, including hexagonal socket countersunk head screws with reduced loadability, up to M20. The minimum ultimate resistance is given as 80% of the value for fixings with full loadability.

Clause 5 of BS EN ISO 898-1:2013 requires fasteners with reduced loadability to be marked with a zero preceding the normal property class designation. A reduced loadability property class 8.8 fastener becomes 08.8.

Countersunk bolts with a hexagonal socket head may also be specified to DIN 7991. This is a withdrawn standard and does not specify a loadability test. It is recommended that countersunk head bolts are not specified to this standard.

BCSA's Model specification for the purchase of structural bolting assemblies will be reviewed later this year (to be issued as 15th edition) to omit a reference to DIN 7991 and to reflect the advice in this AD about the use of bolts with reduced loadability.

Design tension resistance of countersunk head bolts with a hexagon socket

If countersunk head bolts with a hexagonal socket are to be used, their design tension resistance should be reduced by applying a 0.8 factor to the calculated resistance given in BS EN 1993-1-8:2005. It should be noted that the values given in the "Blue Book" and similar resources are applicable only to countersunk heads with a screwdriver slot.

In addition when specifying or using countersunk heads with a hexagonal socket:

- The fixings should be specified in accordance with BS EN ISO 10642:2019,
- The mechanical properties should meet the requirements of BS EN ISO 898-1:2013,
- The fixing should be correctly marked with a zero preceding the normal property class.

It should be clear that if a joint design using countersunk head bolts has been based on the design resistances calculated in accordance with Table 3.4 of BS EN 1993-1-8:2005, only countersunk head bolts with a screwdriver slot should be used, unless the design is verified for reduced loadability socket head fasteners.

If a design is completed using the reduced value of tensile resistance, countersunk head bolts to either BS 4933:2010 (with slots) or BS EN ISO 10642:2019 (with sockets) may be used.

Contact: **David Brown**
Telephone: **01344 636555**
Email: **advisory@steel-sci.com**

AD 531: Heavy power floats on composite slabs during construction

It has come to SCI's attention that composite slabs are being loaded with large power floats weighing up to 1 tonne, that have not been accounted for in the design. The purpose of this note is to remind designers of construction stage loading used in the design of decking profiles and provide guidance in situations where these loads are exceeded.

Power floating is carried out within 2-3 hours of casting after the concrete has sufficiently hardened but prior to it gaining full strength and should therefore be accounted for in the design of the decking profile or slab. As noted in SCI publication P300 'Composite Slabs and Beams using Steel Decking', where construction equipment such as a power float is required to be used, it is recommended that this additional loading does not exceed the allowable temporary construction loading of 1.5 kN/m² over the 3m×3m 'working area'. Therefore, if the power floating loads are no more than the construction load used in the design of the decking, the slab is not overloaded (provided there is no additional, unforeseen load due to 'ponding'). It is worth noting that when a slab design is not governed by the ability of the decking to carry construction stage loading, there will be some spare capacity and so the potential to resist higher loads than defined above.

For loads greater than this, it might be possible to rely on the concrete strength to consider the resistance of the composite slab rather than the decking alone. Both the strength and stiffness of the concrete are relevant, with stiffness potentially affecting the shear connection with the decking. However, reference to Section 3.1.3 of EN 1992-1-1 shows that the stiffness of immature concrete that has reached a certain strength will be similar to the stiffness of mature concrete of a similar strength. This means that strength can be used as a single variable to define the relevant concrete performance. Using cube test values taken prior to using the power float, the slab design could be re-run with the lower concrete strength and correct loading. However, whilst theoretically possible, this may present practical difficulties as software typically only allows standard grades of concrete to be considered. This practical limitation could mean that floating cannot take place until the concrete has reached a level of strength that can be designed for, or the size of power float must be restricted.

Where more than one power float is used, it must be ensured that either the combined weight of the power floats does not exceed the allowable temporary construction loading, there is good site control ensuring that no more than one power float is being used at a time in a bay, or the combined weight has been accounted for in some other way.

Contact: **Liam Dougherty**
Telephone: **01344 636555**
Email: **advisory@steel-sci.com**

AD 532: Integral bracing and diaphragm action of light steel framed walls

The purpose of this advisory desk note is to highlight that guidance provided in SCI P437 supersedes guidance previously provided in SCI ED002 in relation to the design of light steel framed walls with integral bracing and light steel framed wall panels designed as diaphragms.

ED002 was published in 2003 and primarily presents guidance related to lightweight steel and timber composite solutions. P437 was published in 2024 and provides guidance on the design for stability of light steel framed buildings with vertical stability provided by X-braced wall panels, integral bracing or diaphragm action of sheathing boards.

The following parts of ED002 are superseded by guidance provided in P437:

- Section 4 which provides information on the structural performance of light steel framed walls with integral bracing and wall panels designed as diaphragms.
- Appendix A.1 which provides guidance on recommended structural testing procedures for light steel framed wall panels.
- Appendix G which provides information on the structural performance of various types of light steel framed walls.

This advisory desk note will be of particular interest to designers of light steel framed buildings who may have used information from ED002 in their design procedures. For example, Section 4.2 of ED002 gives a serviceability limit state resistance of 4.5 kN for a wall panel with 1 bay of integral bracing. This value has often been quoted in design procedures. Designers must now calculate the design resistance of wall panels with integral bracing following the guidance given in P437 rather than using the generic value from ED002.

Contact: **Andrew Way**
Telephone: **01344 636555**
Email: **advisory@steel-sci.com**

AD 534:
Anchorage length of horizontal stiffeners in steel and composite beams with large web openings

In SCI publication P355 *Design of composite beams with large web openings*, some typographical mistakes have been found in the equations for the anchorage length of horizontal stiffeners. The corrected equations are given below. Given the age of the publication, and that rules will shortly appear in the second-generation Eurocodes for beams with large web openings, it is timely to also provide some background information on the design of horizontal stiffeners at large web openings.

In steel and composite beams with large web openings, horizontal stiffeners are formed from rectangular plates, welded to the top and/or the bottom edges of the openings, and they may be welded on one or both sides subject to certain geometric limits. They have the effect of:

- Increasing the local resistance to Vierendeel bending.
- Preventing local buckling of the web of the Tee.

The maximum size of stiffener is generally controlled by the ability of the web to resist the local anchorage forces, which are transferred by the welds between the stiffener and the web at the ends of the opening. A suitable anchorage length is required in order to develop the full axial resistance of the stiffener. Although not explicably stated in P355, for non-rectangular openings, such as elongated circular openings, the anchorage length is taken from the end of the equivalent rectangular opening.

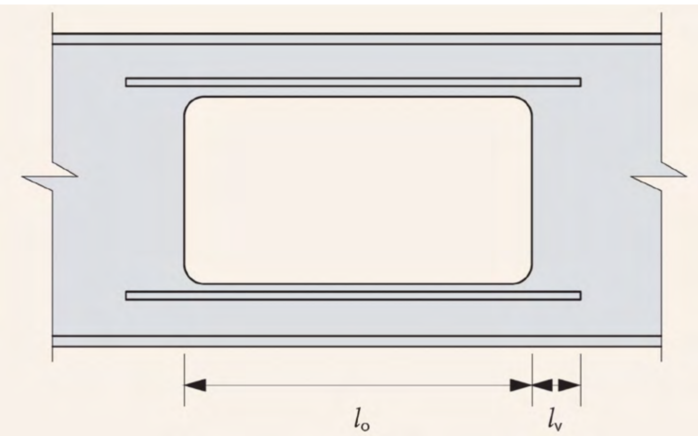


Figure 1: Anchorage length, l_v of horizontal stiffeners beyond each end of the opening

Clause 5.2.2 of P355 provides equations to calculate the anchorage length, l_v (as shown in Figure 1). Unfortunately, there are some typographical mistakes in the shear resistance of the web (equation c) and the design force in the stiffeners, F_r, both of which are corrected below.

In addition to the calculated values, SCI P355 states that the anchorage length, lv of the stiffener beyond each end of the opening should generally be taken as not less than 0.25lo or 2br, with a minimum of 150mm. These minimum values are good practice but should not be used in the absence of suitable calculations. The minimum offset distance from the edge of the opening should be at least 8mm, to allow for at least a 5mm leg length fillet weld.

Where:

l_o is the length of a rectangular opening (or may be taken as le = lo – 0.55 ho for an elongated circular opening)
h_o is the depth of the opening
b_r is the width of the stiffener

The anchorage length of the stiffener should satisfy the following criteria (the corresponding correct equations from P355 are identified for information only):

a) For the design resistance of the longitudinal fillet welds (P355 no different):

$$l_v \geq \frac{F_r}{2naf_{vw,d}}$$

b) For the shear resistance of the stiffeners (P355 no different):

$$l_v \geq \frac{F_r}{nt_r f_y/(\gamma_{M0}\sqrt{3})}$$

c) For the shear resistance of the web (P355 incorrectly included n in the denominator and assumed the web material was the same steel grade as the stiffeners):

$$l_v \geq \frac{F_r}{2t_w f_y/(\gamma_{M0}\sqrt{3})}$$

where:

F_r is the design axial force in the stiffener(s), which may be taken as:

$$F_r = F_{r,Rd} = \frac{nA_r f_{yr}}{\gamma_{M0}}$$

n is 1 for a single-sided stiffener; and 2 for double-sided stiffeners (P355 did not include the variable n)
A_r is the cross-sectional area of a stiffener, or effective area of a Class 3 stiffener
f_{yr} is the yield strength of the stiffener
M_0 is the partial factor for resistance of steel cross sections
a is the throat thickness of the fillet weld
$f_{vw,d}$ is the design shear strength of a fillet weld, given in clause 4.5.3.3 (3) of BS EN 1993-1-8
t_r is the thickness of the stiffener
t_w is the thickness of the web
f_y is the yield strength of steel beam

In addition, P355 gives limits for the relative thickness of the stiffener and web to avoid transverse shear and bending effects in the web. For stiffeners on one side of the web, the web should be relatively stocky so that minor-axis bending of the web due to the eccentric stiffener force can be resisted. This applies for webs with depth, $h_w \leq 70t_w$, where $\varepsilon = (235/f_{yr})0.5$. Double sided stiffeners should be used for more slender webs.

For stiffeners on both sides of the web, the thickness of the stiffeners should satisfy the following limit:

$$\frac{t_r}{t_w} \leq 1.2\left(\frac{l_v}{2b_r}\right)$$

Therefore, for the minimum case of $l_v = 2b_r$, $t_r \leq 1.2\, t_w$.

For single-sided stiffeners the thickness of the stiffener should satisfy the following limit:

$$\frac{t_r}{t_w} \leq 0.96\left(\frac{l_v}{2b_r}\right) \text{ but } \leq 1.0$$

Therefore, for the minimum case of $l_v = 2b_r$, $t_r \leq t_w$ is a reasonable limit.

The rules given in BS EN 1993-1-13:2024 are not as explicit as SCI P355 and therefore the designer should refer to P355 for comprehensive guidance.

Contact: **Liam Dougherty**
Tel: **01344 636555**
Email: **advisory@steel-sci.com**

AD 535: Horizontal lateral loading on internal load-bearing walls in residential buildings

There are two sources of horizontal lateral loading on internal walls in residential buildings that should be considered in their design. These are loads caused by:

- Building occupants (e.g. crowd loads).
- Differential internal air pressure.

These loads should be considered as a leading or accompanying variable actions in combinations determined in accordance with BS EN 1990[1] e.g. using expression 6.10 or expressions 6.10a and 6.10b.

Horizontal loads caused by building occupants

Values of horizontal loads acting on internal walls due to building occupants are defined in Table NA.8 of the UK National Annex to BS EN 1991-1-1:2002[2]. These values should be used in place of those given in Table 6.12 of BS EN 1991-1-1:2002[3]. The loads are specified for categories and sub-categories of loaded areas based on their specific use. Some examples are given in Table 1.

The characteristic horizontal load acting on walls (q_k) should be applied as a line load at a height of 1.20m above the floor level.

BSi published document PD 6688-1-1:2011[4] specifies values for uniformly distributed and concentrated loads applicable to infills for walls and parapets in Table 2. These are specifically for the infill panels within a wall or parapet acting as a barrier (e.g. glass panel) and should not be used for the design of the primary elements of the wall or parapet. The loads given in Table NA.8 of the UK National Annex to BS EN 1991-1-1:2002 and those given in Table 2 of PD 6688-1-1:2011 are not additive and should be considered as three separate load cases.

Horizontal loads caused by differential internal air pressures

Horizontal loads caused by differential internal air pressures lead to a bending moment on the walls that act in combination with the applied vertical loads. Additional moments due to eccentricity of vertical loads from unequal floor spans or unequal loading should also be considered in the design.

Guidance given in SCI publication P394[5] states that for multi-storey buildings the internal wind pressure coefficient (c_{pi}) is "commonly taken as the more onerous of +0.2 and −0.3". This approach is adopted on the basis that the probability of a dominant opening occurring during a severe storm is considered negligible. Adopting the more onerous case for c_{pi} of +0.2 and −0.3 in adjoining compartments results in an overall pressure coefficient for the internal walls of 0.5.

Clause 2.6.1.2 and Table 16 of the now withdrawn BS 6399-2[6] state that for buildings in which the four façade walls are equally permeable, the internal pressure coefficient may be taken as -0.3. It is also states that the maximum net pressure across internal walls should be taken as 0.5.

Modern residential buildings are designed with high levels of airtightness for effective thermal insulation and therefore their external walls are relatively impermeable. Internal separating or compartment walls in multi-occupancy residential type buildings are also likely to be of low permeability which can lead to significant differences in pressures on either side of these walls.

The value of c_{pi} can be estimated by iterative calculation of balancing the inward and outward flow through the various faces, as described in Section 6.2.1 of P394 and Appendix C of SCI-P286. The flow balance is sensitive to the relative permeability of walls and therefore to variations in build quality. Designers may therefore judge it prudent to use values +0.2 and −0.3 and an overall pressure coefficient 0.5 in preference to a more refined calculation. Further guidance on pressures on internal walls and an example of the iterative airflow calculation is provided in BRE Digest 346 Part 8[7].

SCI has conducted a limited number of airflow calculations for a building with a regular arrangement of units positioned either side of a central corridor. Calculations were conducted for various wall permeabilities and the resulting overall pressure coefficients varied from zero to 0.5 depending on the wall permeabilities used. Wall permeabilities based on guidance given in References [8] and [9] were used.

The balance of airflow does not occur instantly. The size effect factor C_a of the standard method in BS 6399-2 accounts for the non-simultaneous action of gusts across an external surface and for the response of internal pressures. As suggested in Reference 8, the size effect factor C_a may be used to account for the response time of the balance of airflow and reduce the resultant internal pressures. Values of the size effect factor are given in Figure 4 of BS 6399-2 and are dependent on the site exposure and the diagonal dimension a. For exposure category B and a diagonal dimension a of 40m, the size effect factor is 0.85.

Using load combinations given in BS EN 1990 expression 6.10 or expressions 6.10a and 6.10b requires variable actions to be assigned as leading or accompanying.

When the imposed floor loading is taken as the leading variable action and the internal wind pressure is taken as the accompanying variable action, there is an additional factor ($\psi_{0,i}$) of 0.5 to be applied to the wind as the accompanying variable action. When the internal wind pressure is taken as the leading variable action there is no additional partial factor to be applied to the wind.

In many cases, for loadbearing light steel frame walls in multi-storey buildings the critical design case will be when imposed floor loading is taken as the leading variable action and the internal wind pressure is taken as the accompanying variable action.

In the absence of a detailed airflow calculation, it is recommended that an overall pressure coefficient of 0.5 is used for the internal loadbearing walls of residential buildings. The factors discussed above can be used to reduce the design actions of the internal wind pressure.

For double leaf light steel framed walls, the lateral loading due to differential air pressures on either side of the wall can be assumed to be resisted equally by each leaf of the wall.

For internal walls within a dwelling it is not necessary to carry out airflow calculations to determine the air pressure on each side of the wall as there will be significant air leakage between rooms due to gaps around doors and in many cases doors being open.

Large windows or doors (which would be classified as dominant openings) being open during high winds should be treated as an accidental load combination which has lower partial factors applied to the loads.

Contact: **Andrew Way**
Telephone: **01344 636555**
Email: **advisory@steel-sci.com**

[1] BS EN 1990. *Eurocode - Basis of structural design*. BSI, 2010.
[2] BS EN 1991-1-1:2002 UK NA. *UK National Annex to Eurocode 1. Actions on structures - General actions. Densities, self-weight, imposed loads for buildings*. BSI, 2019.
[3] BS EN 1991-1-1:2002. *Eurocode 1. Actions on structures - General actions. Densities, self-weight, imposed loads for buildings*. BSI, 2002
[4] PD 6688-1-1:2011. *Recommendations for the design of structures to BS EN 1991-1-1*. BSI, 2011.
[5] SCI P394 - *Wind actions to BS EN 1991-1-4*. SCI, 2014.
[6] BS 6399-2:1997. *Loading for buildings - Code of practice for wind loads*. BSI, 2010.
[7] BRE Digest 346 Part 8. *The assessment of wind loads. Part 8: internal pressures*. BRE, 1990.
[8] *Wind loading: a practical guide to BS 6399-2*. N. Cook. Thomas Telford, 1999
[9] BS EN 1991-1-4:2005+A1:2010 UK NA. *UK National Annex to Eurocode 1. Actions on structures - General actions - Wind actions*. BSI, 2008.

Category	Specific use	Sub-category	Description	Characteristic load (q_k)
A	Areas for domestic and residential activities	(i)	All areas within or serving exclusively one dwelling including stairs, landings etc.	0.36 kN/m
		(ii)	Residential areas not covered by (i).	0.74 kN/m

Table 1: Examples of horizontal loads on internal walls due to building occupants
Note: For full definitions and descriptions see Table NA.8 of the UK National Annex to BS EN 1991-1-1:2002.

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